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PROCEEDINGS OF ROADBED STABILIZATION LIME INJECTION CONFERENCE

Edited by

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16. Abstract

Proceedings of Roadbed Stabilization Lime Injection Conference includes twelve technical papers that were presented at the conference on August 21 and 22, 1975 in Little Rock, Arkansas. The papers document the state of knowledge and related subjects on lime pressure injection stabilization of problem railroad subsoils. The papers were prepared and presented by academic researchers, injection contractors, soils engineers, a railroad engineer and a research geophysicist. The related papers are on electro-chemical stabilization, finite element analysis of roadbeds and nondestructive testing of roadbed soils. These proceedings are the first to be published on the subject of lime pressure injection soil stabilization.

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PREFACE

One of the major problems facing the American railroads is the rising cost of continued maintenance required on existing problem roadbeds. One method to combat the rising cost of track maintenance is the lime injection method of roadbed stabilization. Chemical stabilization of subgrade soils with modern hy-rail high performance injection equipment is relatively new to the railroad industry. In 1972 Southern Railroad began a roadbed maintenance program utilizing this method. Since that time over a dozen railroads have used the method and between 200 and 300 miles of track roadbed have been treated to depths ranging from 7 feet to 20 feet below the top of the track.

The Graduate Institute of Technology (GIT) was awarded a Federal Railroad Administration research contract in 1974 to examine the ability of the lime injection stabilization technique to improve in-place, subgrade soils of railroad trackways. The GIT has thoroughly studied, evaluated, and documented data generated by the several rail lines that have used lime injection over the past few years. The preliminary indications are that the lime injection stabilization technique is a valuable method for substantially reducing the maintenance cost on problem trackway sections.

The Roadbed Stabilization Lime Injection Conference brought together experienced railroad track engineers, research engineers, and injection contractors for the purpose of interchanging information. The papers presented covered technical data oriented to railroad track stabilization, maintenance design criteria and new construction. Those with specialities in the major areas of lime injection roadbed stabilization were invited to attend and participate in the conference. A complete set of the prepared conference papers was provided to each conference attendee. Copies of these same papers are included in this volume of proceedings. Dr. James L. Eades did not present a paper; all other speakers presented papers. These papers can serve as a guide for the immediate utilization of the lime injection method.

The Conference Director and FRA DOT Project Monitor wish to take this opportunity to express their appreciation to those who worked to make the conference a success, especially to the conference speakers who prepared and presented the excellent slide presentations and written papers and to the University of Arkansas and the DOT for sponsoring the conference.

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INTRODUCTION

The purpose of the lime injection conference was to bring people together for consolidation, discussion, interaction and transfer of state-of-the-art lime injection roadbed stabilization technical knowledge. The conference was conducted in four sessions on August 21 and 22, 1975 at the Camelot Inn in Little Rock, Arkansas. The conference was jointly sponsored by the University of Arkansas Graduate Institute of Technology and the Federal Railroad Administration of the Department of Transportation. See Figure 1-1 for a copy of the actual program. This was the first engineering speciality conference on the lime slurry pressure injection method of soil stabilization. This method, although in use for the past fifteen years by the civil building industry, has seldom been studied or documented. It is appropriate that at this time when the railroad industry is re-emerging as a viable industry, it should once again assume a leadership position in the development and expansion of technical knowledge in the field of civil engineering.

The conference was attended by engineers and technical employees from many areas of the United States and Canada. A list of attendees is presented in Appendix A. The attendees were treated to a concentrated program of sixteen hours of excellent technical instruction by the thirteen principal speakers and the two luncheon speakers. Two discussion sessions were held for the purpose of informal discussion and interaction. The professional cross-section of the speakers was distributed between railroad personnel, academic researchers, contractors, materials suppliers, and soils engineers. Mr. J. B. Farris of the Southern Railroad was the first conference speaker. He related his considerable experience with lime injection on the Southern Railroad. The Southern was the first railroad to utilize the current method of lime injection stabilization. Representatives of both railroad lime injection contractors, Roadway Stabilization, Inc. and Woodbine Corporation, were present. Mr. Paul Wright of Woodbine Corporation presented a paper explaining the contractor's viewpoint, equipment and railroad injection techniques.

Dr. James L. Eades of the University of Florida presented an extensive lecture with many colored slides on the subject of Lime-Soil Reaction. Dr. Eades is a foremost recognized expert in the lime stabilization field although he professed little actual experience in lime pressure injection. His attendance was especially meaningful to the conferees as he was able to form a link between the current conventional lime stabilization base of knowledge and the new developing lime pressure injection technology. Because of the length of Dr. Eades' presentation and the importance of the colored slides to the understanding of his explanation of the soil-lime crystalline structure, Dr. Eades was not able to present a written paper.

Three papers not directly related to lime stabilization were presented. Dr. Charles E. O'Bannon of Arizona State University presented a paper on the Electro-Chemical Stabilization method used on the Arizona highways which possibly has some application in the railroad industry. Mr. S. S. Cooper of the Waterways Experiment Station presented a paper on non-destructive testing of roadbed soils recounting experiences from the Kansas

LIME INJECTION CONFERENCE
Camelot Inn — Little Rock

PROGRAM

Thursday, August 21
7:30 a.m. – 4:30 p.m.

REGISTRATION

GREETINGS – *Jay W. Fredrickson, Director, Graduate Institute of Technology, University of Arkansas*

KEYNOTE ADDRESS – *William B. O'Sullivan, Chief, Improved Track System Research Division, Federal Railroad Administration*

LIME INJECTION ROAD BED STABILIZATION PROJECT EXPERIENCES – *J.B. Farris, Engineer, Geotechnical, Southern Railroad*

COFFEE BREAK

LIME–SOIL REACTIONS – *James L. Eades, Chairman, Department of Geology, University of Florida*

SOIL EXPLORATION ON RAILROAD TRACKS– WHY NECESSARY? – *C. Ronald Rone, President, Rone Engineering, Inc.*

LUNCHEON: CHANGING CAPITAL NEED AND REGULATORY REFORMS IN RAILROAD TRANSPORTATION – *Grant M. Davis, Oren Harris Professor of Transportation, University of Arkansas*

FIELD EVALUATION OF PRESSURE INJECTION LIME TREATMENT FOR STRENGTHENING SUBGRADE SOILS – *Marshall R. Thompson, Professor of Civil Engineering, University of Illinois*

ALTERATION OF SOIL PROPERTIES AS EFFECTED BY VARIOUS LIME TREATMENT PROCEDURES – *Quentin L. Robnett, Associate Professor of Civil Engineering, University of Illinois*

COFFEE BREAK

ELECTRO-CHEMICAL STABILIZATION – *Charles E. O'Bannon, Associate Professor of Civil Engineering, Arizona State University*

INFORMAL DISCUSSION SESSION

Friday, August 22
8:30 a.m. – 4:00 p.m.

ENVIRONMENTAL CONSIDERATIONS – *Albert F. Vickers, Assistant Professor of Environmental Engineering, Graduate Institute of Technology*

SELECTED CASE STUDIES OF LIME INJECTED RAILROAD TRACK – *David F. Sheaff, Project Engineer, McClelland Engineers, Inc.*

COFFEE BREAK

EXPLORATION AND TESTING – *Robert C. Welch, Associate Professor of Civil Engineering, University of Arkansas*

LIME INJECTION PRODUCTION EQUIPMENT AND TECHNIQUES – *Paul J. Wright, Vice President, Woodbine Corporation*

LUNCHEON: THE LIME OUTLOOK – *Robert S. Boynton, Executive Director, National Lime Association*

ANALYSIS OF LIME INJECTED ROADBEDS– *James R. Blacklock, Program Director of Railroad Research, Graduate Institute of Technology*

REPEATED LOAD TESTING ON FOUR LIME STABILIZED OKLAHOMA SHALES – *Subodh Kumar, Assistant Professor of Engineering, University of Arkansas at Little Rock*

COFFEE BREAK

NONDESTRUCTIVE TESTING OF ROADBED SOILS – *S.S. Cooper, Research Geophysicist, U.S. Army Engineer Waterways Experiment Station*

Speakers can be identified by white ribbons attached to their name tags.

Hosts will have red ribbons attached to their name tags.

Sponsors

UNIVERSITY OF ARKANSAS
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FEDERAL RAILROAD ADMINISTRATION

Figure 1-1 Actual Conference Program

test track. Dr. James R. Blacklock of the Graduate Institute of Technology presented a progress report on finite element research for structural analysis of track structural systems.

Two excellent luncheon speakers, Dr. Grant M. Davis of the University of Arkansas and Mr. Robert S. Boynton, Executive Director of the National Lime Association, and the keynote speaker, Mr. William B. O'Sullivan, Chief of the Improved Track System Research Division of the FRA, contributed substantially to the overall success of the conference. Mr. Boynton and Dr. Davis prepared papers and copies are included in these proceedings.

The results of a recent Air Force sponsored in-place soil stabilization research program to examine promising methods for rehabilitation of worn overloaded paved runways were presented by Dr. Marshall Thompson of the University of Illinois. Dr. Quentin L. Robnett of the Georgia Institute of Technology presented data on lime-soil reaction from the University of Illinois lime stabilization laboratory research programs.

Mr. C. Ronnie Rone of Rone Engineering, Inc. recounted his original experience as a soils engineer involved in the first lime injection project conducted in Texas in 1967 and emphasized the importance of the soils test laboratory in the lime injection soil stabilization process.

Dr. Albert F. Vickers, David F. Sheaff, Dr. Robert C. Welch and Dr. Subodh Kumar, members of the University of Arkansas Railroad Research Team, presented results from their lime stabilization research. Dr. Vickers addressed the environmental problem associated with lime injection, and Dr. Welch and Dr. Kumar discussed laboratory testing methods and results. Mr. Sheaff, who has been working on the GIT's FRA case history study program for lime injection maintenance of railroad subgrades, presented a progress report on the accomplishments of that portion of the FRA-funded railroad research.

LIME INJECTION ROADBED STABILIZATION

PROJECT EXPERIENCES

By

J. B. Farris
Engineer, Geotechnical, Southern Railroad
Atlanta, Georgia

ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

Little Rock, Arkansas
August 21 and 22, 1975

It is a privilege to appear before this group this morning to discuss some of our project experiences with Lime Injection Roadbed Stabilization on Southern. I believe I will neglect to dwell on those ulcer producing logistical problems this morning such as lost lime trucks and pick out a few of the locations we have treated to discuss somewhat in detail. First a bit of background.

The use of lime to stabilize problem clay soils is always attractive and simple when we build a new grade. We first used lime on Southern in 1959 on a new extension of the Louisiana Southern. We were very happy with the results until a hurricane came along and removed a large section of the grade. That which remains is a superior roadbed to that which was built under emergency conditions after the hurricane.

On our Mobile Division in the late 1960's, we had moderate success with post hole lime application. About 1970, reports from the southwest concerning lime injection for foundations of buildings caught our attention and when a contractor developed a truck rig with retractable rail wheels we decided to give the equipment a try. We planned to use this equipment to treat subgrade failures referred to by our maintenance people as "squeezes" because these failures manifest themselves by extruding failed material up between the crossties or to the sides of the track. I will refer to these as "squeezes" in this discussion. The squeezes are formed most frequently at switches, rail joints, points of unequal tie support, (that is, where one good tie is located between several soft ties), where drainage has not been maintained and where inadequate ballasting existed.

Methods which we formerly used to treat squeezes were sand piles, wood piles, poles or rail driven into the squeeze to arrest movement of the subgrade, cement grouting and pulling on ballast. Pile driving was very expensive, pulling on ballast was usually ineffective or temporary and cement grouting formed concrete with the ballast as the course aggregate, resulting in very expensive work when time came to replace the ties and surface the track. With the lime injection system offered by contractors, the unit price was right <u>if</u> it did the work.

The occurrence of the squeezes were most frequent in clay soil areas. When these locations were spotted on soil maps, the frequency of occurrence related to the soil. In this manner we try to anticipate where trouble might show up when tonnage is increased on a line. Some of the areas where squeezes are most prevalent in our territory are the prairie soils of Alabama, the Eden Shale soils of Northern Kentucky, the residual soils in the Tuscaloosa formation, the clay deposits on tops of ridges running through the Piedmont areas of North Carolina toward the Atlantic and in the high organic soils along the coast.

After a few trial locations were treated, we developed a program of stabilization for the line between Greensboro and Sanford, North Carolina in 1972. This line had carried light traffic for many years. With the increase of traffic and axle loading in recent years, the old squeezes began to reactivate and new ones were formed. The line had been timbered and surfaced two years before. The most successful treatment of the rapidly forming squeezes consisted of digging out the failed material and replacing with dense graded aggregate, a very expensive remedy. A number of locations had been dug out two or more times. Unstable areas were forming faster than they could be corrected. A small smoothing gang kept the track passable. The soils here were yellow silt clays, yellow and red clays. The yellow clays had a L.L. of 60 and P.I. of 30. The red clays had a L.L. of over 100 and a P.I. in the 70's. Lime applied at the rate of 6% lowered the P.I. 6 to 8 points in the yellow clay and 3 to 5 points in the red clay. Recognizing that we might not effect a high percentage of corrections, lime was applied to the failed areas of roadbed from March through June 1972. 37,000 lineal feet of track was treated with lime slurry. The mixture was slightly thinner than that currently recommended. The average cost was about $2.40 per lin. ft. of track treated including the material.

The lime application was followed three months later by T&S work across the first 22,000 feet of treated line so that timber condition in the track was good, the ballast section was adequate and the line and surface was in good shape. This 22,000 feet of track has resisted formation of new squeezes and has the old ones here remained inactive. The remaining 15,000 feet of track was not timbered and surfaced. Half of the squeezes in this area remained active. The area was treated with lime again in 1974 and track conditions corrected. The cost of this program, with costs averaged against effective work was $3.40 per foot of track corrected. The benefits were quickly apparent from the work. The small gang was able to be placed on more productive work and Slow Orders were reduced.

Sometimes individual squeezes offer the greatest challenge. There was one location in the Alabama prairie about 20 miles west of Selma, where we had tried all the measures we knew for several years to try to stabilize the squeeze. The squeeze was about 200 feet long, just east of a road crossing. The right of way was narrow and the land was very flat. The ditch was contained in the squeeze so that water was continuously ponded permitting cat tails to grow. We had tried subdrains, sand piles, grouting and replacing ballast without success. The soil was classified as Eutaw clay on agriculture mapping. The P.I. of this clay varies from 25 to 30 with lime at 7% reducing the P.I. by 8 to 10 points. Lime was applied twice to this area by the post hole method with no result. Lime was applied by injection twice, the last time in June 1974.

An inspection in November 1974 indicated no betterment of the condition. In June 1975, the track was holding line and surface, effective drainage had been restored and no cat tails were in sight.

We have had several locations react in this manner. Some several months have elapsed before the lime treatment appeared effective.

In Devon Industrial Park, about 12 miles south of Cincinnati in Northern Kentucky, there were severe squeezes on an industrial lead. The soil is residual clay in the area, from the Richmond-Maysville formation. The L.L. of the soils runs in the 40's. Soil samples indicated reaction to lime. Lime was injected during the summer of 1974. We estimate these results at 50% effective.

It is not unusual to fail to get the results we want even though the soil appears to react to lime. The failures are usually scattered through the area being treated. A second treatment will usually pick up some of these. Perhaps a third round will wipe them all out. We haven't reached that plateau yet.

There are certain things I don't believe lime will do even though at times it takes on the aspect of Dr. Smith's cure-all fresh from the medicine wagon.

I don't believe it will make the ballast deeper.
I don't believe it will make good crossties out of bad ones.
I don't believe it will straighten bent rail.

Programming the lime treatment is a dead serious business with our maintenance people. The unit cost is extremely attractive, but it takes a heavy toll of available funds to afford the mass production effort required to achieve the low unit cost. We have to produce actual dollar savings in our stabilization work. All of us realize that roadbed strengthening is badly needed to eliminate repetitious smoothing, to increase tie life, to increase the time period between maintenance cycles and to keep our plant in shape to take care of the added traffic which may be coming our way.

SOILS EXPLORATION ON RAILROAD
TRACKS - WHY NECESSARY?

By

C. Ronald Rone, P.E.
Rone Engineering, Inc.
Soils Engineering & Laboratory Testing
Arlington, Texas

ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

August 21 and 22, 1975
Little Rock, Arkansas

INTRODUCTION

Documented evidence shows that expansive clays exist world-wide and creates billions of dollars worth of damages each year to foundations. In the United States, the cost of damages caused by expansive clays, such as slope failures, pavement failures, and building foundation failures, alone is $2.3 billion annually. This $2.3 billion figure does not include damages caused by clays and other type soils to railroad tracks, as these damage figures are not available. This unbelievable figure is more than twice the damage caused from floods, hurricanes, tornadoes, and earthquakes. In the Dallas/Fort Worth area alone, there is over $30 million each year spent on repairing just house and apartment foundation failures caused by swelling clays.

With such a massive amount of money spent each year on just swelling clay damages, something is wrong. Why is so much money spent on this seemingly simple problem? We could very easily point to others, such as architects, structural engineers, railroad design engineers, contractors, universities, industries, etc., and say it is their fault the problem hasn't been solved. But, did you ever notice that when you point your finger at someone, there is always three fingers pointing back at you. In other words, it's not your fault, it's mine and other soils engineers like me. We, as soils engineers, have just begun to solve these expansive and technically oriented soil problems. In addition, soils engineers haven't been able to sell industry or you, that we really can solve or alleviate a number of your soil related problems. That is why I've been asked to speak to you today. I'm going to introduce you to soils engineering, the work we do, and the tests we can perform on related railroad problems.

LIME INJECTION TEST SECTION

In order to achieve my goal, since this conference is about lime injection, I'll discuss our experiences with this process in the North Texas, Dallas/Fort Worth area.

In late 1967, a lime injection company from Louisiana called me to discuss the lime injection process. About this time my father's house had undergone severe foundation distress. Accordingly, it was decided to use his house as a test section, since the lime injection process had not been used in Texas prior to this time. The soils underlying my father's house consisted of sands and clays from the Woodbine Geological Formation. It was decided to lime inject, at close intervals, around the perimeter to create a moisture barrier. I had invited over 100 clients to witness this new process. The process worked and the foundation stopped moving.

LIME INJECTION - PACKER SYSTEM & STRAIGHT-PIPE

One of the visitors to the aforementioned lime injection project was Mr. Joe Teague, now President of Woodbine Corporation. He and his partners, Mr. Gene Cain and Mr. Paul Wright (Mr. Wright will be one of our speakers tomorrow), were so impressed with the potential of the product that they started their company in 1968. As you know, they do an extensive amount of lime-slurry work for the railroads at the present time.

We performed a series of test sections for Woodbine Corporation back in 1968 and the findings were exciting. One method of injection, called the "inflatable packer method," seemed to have potential. However, the test section proved it to be not as effective as the "straight-pipe method."

The "inflatable packer" was designed to reduce loss of lime slurry back-up through the injection hole. However, it normally found only one zone of weakness in the soil to whatever depth you were injecting; whereas, the "straight-pipe" arrangement (see slide) proved to find a zone of weakness at each interval of injection. We normally specify the injection intervals to be approximately every one (1) foot. In so doing, our test section proved that one or more lime seams existed for each interval of injection down to the bottom of the seven (7) foot injection.

LIME INJECTION SUCCESSES

Due to many following successful lime injection projects, this new process overnight became the talk of the town and the solution to everyone's foundation problems. For instance, in the last six years in North Texas alone, over 50 million ft^2 of building foundations have been lime slurry injected. Some of the reasons why the process was accepted so easily and without question in some cases, were as follows:

1) Scarification of hydrated lime had been used as a very successful subgrade treatment for roads and parking lots in Texas and other states since 1934. To the layman, the lime injection process appeared to do the same thing as surface lime scarification did for roadbed construction.

2) In most cases, the only other alternative, other than a structurally suspended floor slab, was the costly over-excavating and backfilling with sand method. This method sometimes caused foundation problems itself.

3) The process was glamorous and technically complicated to both the engineer and user.

4) The results of the lime injection process on near surface soils allowed the contractor to have a working platform during the rainy season.

5) The process didn't take as much time as other methods.

6) The process was economical and very competitive in price compared to other construction techniques.

MISUSES OF LIME INJECTION

With time, the process started to be abused and often used incorrectly. Consequently, the process is now being closely scrutinized by soils engineers, architects, structural engineers, and builders. Relying on our experiences since 1967 with the lime injection process, we have compiled a list of reasons why the process is unsuccessful in some cases.

1) Using the process in clay soils, where hydrated lime will not chemically react.

2) Not having a qualified soils engineer drill and test the soils to determine how deep to inject or if the process will work at all.

3) Unqualified soils engineers performing the wrong soil tests prior to and after completion of the injection on a project.

4) Not having a set of lime injection specifications prepared for the project by a qualified soils engineer prior to injection or having carte blanche lime injection specifications for all sites.

5) Assuming that one injection is all that is ever needed.

6) Not re-injecting the soils when the soils engineer specified that they should be.

7) Not testing the soils after completion of injection.

8) Not understanding that in some clay foundations, that lime injection cannot and will not penetrate certain clay strata.

9) Using the process when other construction techniques would have been more positive and cheaper.

10) Allowing any lime injection company, without proper qualifications and experience, to inject a site.

11) Allowing any type of injection nozzle or equipment to be used for injecting.

12) Allowing waste lime to be used instead of hydrated lime.

13) No surfactant, or allowing the wrong kind of surfactant to be used.

14) No laboratory control of mixing of hydrated lime, surfactant, and water. Improper agitation in slurry tanks and no control of injection pressures.

SOIL SAMPLING USING A HY-RAIL DRILL RIG

Utilizing our prior experiences with the lime injection process and our knowledge that the process will work in certain soils if properly injected, we were naturally very excited when Dr. Jim Blacklock, Graduate Institute of Technology, came to visit our firm and asked us to work with G.I.T. on the Federal Railroad Administration Grant.

Because we know that a large portion of your railroad problems are caused by the following formula, we naturally will be orientating our analysis and recommendations to alleviate these conditions:

1) Clay + rain + dynamic train traffic = quagmire

2) Fill + freezing weather = unstable surface upon thawing

3) Improperly compacted fill + rain = settlement

4) Too steep a cut + rain = landslide

Since our first meeting, we have installed hy-rails to our drilling equipment in order that we may take soil samples on any railroad track in the United States.

When drilling a site, cohesive soil samples are obtained when possible using thin-walled, seamless, Shelby tube samplers. The soils are carefully extruded from Shelby tubes so as not to disturb them. For protection from loss of moisture, each sample is wrapped in aluminum foil and placed in a jar or core box. This is important, as the Shelby tube samples indicate where the potential problem might be. For instance, here is a sample (see slide) where a thin layer of fine gravel was piping water to the clays, thus causing the loss of shear strength in the soils.

This site which had a subsurface gravel layer was successfully injected and stabilized. Other sites which had subsurface sand layers also have been injected and successfully stabilized (see slide).

A gravel or sand layer cannot be observed with auger borings, thus requiring undisturbed sampling techniques. Since the identification of the different sand and gravel layers along with their respective depths is a necessity, the use of an experienced logger is imperative. The logger provides the soils engineer with pertinent information which will enable the engineer to determine what method is needed to correct a soil problem.

LABORATORY TESTS

All soil samples are classified in accordance with the Unified Soil Classification System and tested in accordance with the American Standard of Testing Materials (A.S.T.M.) and other acceptable test methods in our Arlington laboratory. Some of the laboratory tests, recommendations, and conclusions relevant to specific projects are as follows:

1) Classification

   a. Sieve Analysis
   b. Atterberg Limits
   c. Hydrometer
   d. Moisture Content
   e. Pin Hole Tests

2) Lime Reactivity - tests with varying lime content

   a. Atterberg Limits
   b. Bar Shrinkage
   c. Pin Hole Tests (dispersant clays)
   d. Eades Quick Test (pH)

3) Physical Properties - (undisturbed samples)

   a. Compressive Strength (Unconfined, Triaxial, Pocket Penetrometer)
   b. Permeability
   c. Swell or Compression Characteristics
   d. Proctor (compaction)

4) Durability or Endurance - time effects, tests with varying lime content

   a. Soak and dry test on cured remolded samples.
   b. Compressive strength of cured remolded samples.

5) Chemical Characteristics

   a. Effect of salt, fly ash, or other chemical stabilizers and additives.
   b. Harmful contaminants - sulfates, organic matter, etc.
   c. Chemical Analysis (as required on selected soil samples)

6) Recommendations and Conclusions

   a. Describe track and site conditions
   b. Describe soil boring logs
   c. Summarize soil laboratory test results
   d. Discuss what process or construction technique to use in solving soil problems
   e. If previously injected with lime or cement, discuss effectiveness of the process
   f. Complete soils engineers report on the drilled and tested project.

WISE INVESTMENT

In order to eliminate costly soil problems, it is always a wise investment to drill and test the soils so that you may have a fighting chance to solve your problem. This landslide (see slide) may have been alleviated or prevented if we had drilled, tested, and made recommendations prior to construction of the apartments. The specific reason for the slope failure at the aforementioned apartments was a perched water table which saturated the in-situ soils. The water table was eliminated with a simple French Drain which reduced the high water content in the soil embankment; thereby, increasing the soil shear strength and slope stability.

CONCLUSION

Just as it would have been a wise investment to drill the slope failure site prior to construction and subsequent failure, it would also be wise to drill and test the soils under railroad track soil problem areas prior to remedial improvements. It might be that these soils should be cement grouted instead of lime injected, or maybe a combination of fly ash and hydrated lime should be injected to stabilize the soils. Or possibly, some other completely simple construction technique could solve the problem.

From prior experience, we have seen jobs that were injected to seven (7) feet and the problem existed at eight (8) feet. Accordingly, drilling and testing would have prevented this waste of money and effort. For an example of how people can abuse a good process, a residence in Irving, Texas was lime injected without prior knowledge of the existing subsurface soil conditions. The house foundation underwent severe distress, resulting in damage to the superstructure. We were then requested to determine the causes for the distress. Subsequently, we drilled the house and from the results of our drilling operation and laboratory tests, we proved that lime injection should not have been used in these clays because of the absence of voids within the existing soils along with the lack of joints, seams, or fissures. Because lime injection was misused, the builder had to pay over $35,000.00 in remedial repairs. Thus, it may be seen how poor judgment and abuse of a good process, such as lime injection, can develop a bad name.

Therefore, it may be concluded that for your railroad track soil related problem areas, drilling and testing will enable the soils engineer to make recommendations relative to the successful use of lime injection and prevent the abuse of the process in the future.

Rone Engineering, Inc. is experienced in all phases of soil drilling and testing for lime injection and other engineering problem solving techniques. We are looking forward to solving your railroad track soils related problems.

Luncheon Speech

THE NEED FOR REGULATORY REFORM IN RAILROAD TRANSPORTATION

Grant M. Davis
Oren Harris Professor of Transportation
University of Arkansas

It is certainly my pleasure to visit with you here in Little Rock today and to participate in this important conference dealing with U. S. railroads. When I first mentioned to my wife that I would be participating in this program, she said that the railroad industry isn't the only area that needs some sort of roadbed stabilization. Our front street could certainly stand something other than a half an inch of concrete that the city of Fayetteville poured for us. Anyway, it is my pleasure to be here with you today to participate in this conference, and I would like to address myself to some of the pressing capital needs that certainly determine whether or not roadbed stabilization programs can be meaningful in addition to recommending some regulatory reforms that I believe could be made to assist the transportation industry in general as well as the railroad industry.

Each and every one of us in this room should be aware of pressing transportation problems that the railroad industry is facing. I personally am disturbed because many of the administration's proposals emanating from Washington suggest a move to partial nationalization of our domestic railroad, or a tendency towards more government interference in management decisions. Let us examine briefly some of the economic significant statistics affiliated with our domestic railroad industry. What most people are unaware of is that between 1973 and 1974 the Class I railroads increased their net investment in plant and equipment from $27 billion to almost $29 billion. Rolling stock was increased and overall capital expenditures were expanded 16.6% during that interval. Now I ask you, if the industry is in such serious financial stress, why do we find this increased financial investment by management? Let me propose that while the railroad industry is not a sick industry, only parts of it need help, but these segments are detrimental to the overall industry because of their geographical location.

Unfortunately, many of the proposals for reform and modification of our regulatory proceedings and bills dealing with transportation certainly are lacking in terms of substance. In fact, I am disturbed with President Ford's transportation messages and his misunderstanding of the way our transportation institutions function to serve the consuming public. Unfortunately, many of the changes that are being advocated in Washington could certainly be detrimental to the transportation industry. One bill that does seem to have some merit is the Railroad Revitalization Act of 1975. Unfortunately, many provisions of this proposed bill will not assist the industry, so perhaps, the term is misleading. However, the proposed legislation does provide for uniform cost accounting, financial assistance through loan guarantees, and the elimination of discriminatory state and local taxation of interstate carriers. Other than that, however, I find the bill to be lacking in substance.

To my way of thinking, there are six major areas of change that are needed to assist the industry in terms of meeting its capital requirements and being more consumer oriented. These include finance, the elimination of excess capacity, modification of the railway labor act, development of one uniform national transportation policy, the elimination of excessive wasteful and needless federal safety requirements, and lastly, certain regulatory modifications. Now let us briefly examine each of these areas.

With respect to finance, let us be realistic with each other--the railroad industry is a simple growth industry--not a dynamic growth industry. For many years now the rate of return on railroading has been inordinately low, and on many occasions, the industry has been assisted by government guarantee loan programs. The railroad industry is a vital and essential part of our national transportation system, and as such, resources must be devoted to this industry that will enable the movement of bulk commodities over great distances.

Perhaps, management should examine their capital structures with a view toward debt elimination and increasing the productivity of capital. The industry is characterized by a massive amount of fixed costs, and industries of this nature should pay as much attention to the productivity of their capital as they do to overall production trends.

Closely related to the problem of inadequate capital sources is the problem of excess capacity. There is no question in my mind but that excess capacity is probably the primary reason, together with certain other factors, for the financial problems of the railroads today. There is a need to identify the core rail routes in this country and to modify our abandonment procedures in such a manner that the excess capacity can rapidly be eliminated. Diligent care must be exercised, however, to make certain that the carriers do not negate their common carrier responsibilities.

Traditionally, firms with excess capacity in regulated industries can eliminate costly but duplicate assets by mergers and consolidations. In the rail industry, however, because of unique labor relation laws, any displaced union employee brought about by a merger must be given four years severence pay or placed in a comparable position for four years. We recently completed a study of productivity in certain selective mergers before and after the protection period expired. At our Center it came as no surprise to us to find that productivity almost doubled in certain select rail mergers once the labor protection provision period expired.

Also, closely related to excess capacity and inadequate capital sources is the antiquated Railway Labor Act, which to my own personal way of feeling is the worst single piece of legislation that has ever been forced on the transportation industry. It makes absolutely no sense, whatsoever. Why should a railroad operating under a monopoly grant and a union operating under a monopoly grant not be able to reconcile their differences in some way that does not require an ad hoc decision on part of Congress and a subsequent inflationary settlement. The Railway Labor Act, of course, recognizes numerous crafts because of its liberal recognition factor, and unions are organized along craft lines and are interested only in the long-range survival of the craft. Rail management, on the other hand, can not effectively develop any sort of meaningful labor relation program when it has to deal simultaneously with 42 different unions. This Act needs to be repealed and all transportation labor should come under Taft-Hartley, and the fragmented unions should be consolidated using democratic processes, of course, into one union. I would like to point out to each of you that the actual number of people who operate trains in this country has increased from .9 percent of the labor force in 1950 to approximately 31 percent today.

Another area of change that is sorely needed concerns the elimination of needless federal safety requirements. None of us in this room are opposed to safety--in fact, safety in all walks of life should be an important societal goal. But, let me point out one instance to you that shows where a zealous government bureaucrat can cost you and I, the consumer, a tremendous amount of money. Each of you will recall the propane explosion that occurred in Illinois several years ago. The explosion was a result of somebody switching a car through a switchyard at approximately 18 miles per hour when the system is designed not to exceed 10 miles per hour. However, as a result of this particular explosion, federal safety rules have been proposed and enacted stipulating that every car carrying certain chemicals be switched one at a time or in conjunction with a buffer car. Also, they have required extensive modification of tank cars that will cost users millions of dollars. All of this is going to cost the American consumer a tremendous amount of money. My question is: Why can't the Department of Transportation and the industry get together and resolve these differences without extensive rule making provisions that end up costing everyone money.

The last major change that I would advocate concerns regulation. Economic regulation is essential to the continuing development and sophistication of the domestic transportation network, but certain parts of it need to be modified. For example, the commodities clause is still on the books today, and this particular Act could possibly retard the development of an important energy source in this country, that is coal. I would be the first to agree that the commodities clause was probably well intended when it was first enacted. However, that was almost 70 years ago, and the rail industry no longer enjoys the monopoly position today that it once occupied. Another regulatory reform should be the deregulation of movement of agricultural products. The rail industry is placed at an extreme competitive disadvantage when competing with carriers exempt from regulation. To develop uniform regulation and balanced public policy, this particular traffic should be deregulated.

Another important regulatory change concerns due process. Let's face it, one man's red tape is another man's due process. However, we do know that many decisions take years and years and years from the state and federal commissions. Efforts should be made to increase the speed at which these decisions are rendered.

Another important modification of regulation concerns a plea on my part not to eliminate rate bureaus' right to protest independent action and carrier single-line immunity. To do so merely will increase the overall cost of transportation regulation, and I can cite you example after example after example to support my particular view.

The last major change that I would advocate in the area of regulation concerns increased use of economic analysis. I never cease to be amazed at the lack of economic analysis employed in our regulatory decision making. The Interstate Commerce Commission, of course, has a bureau of economics and a capable staff. However, many administrative law judges simply do not understand the basic functioning of economic analysis, and it is beyond me how they can render a decision in an important economic area without some basic analytical knowledge of how the system functions.

I certainly appreciate being invited to speak to you today, and I hate to sound so negative, so I would like to end this speech on a positive note. We have the most effective and efficient transportation system in the world. But, like anything else functioning within America's institutional environment, it can be made more efficient. And, I think we need to address ourselves to increasing this efficiency during this nation's bicentennial celebration.

Thank you very much.

FIELD EVALUATION OF PRESSURE INJECTION LIME TREATMENT FOR STRENGTHENING SUBGRADE SOILS

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

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INTRODUCTION

GENERAL

Many existing transportation support systems (airport runways and taxiways, highway pavements, railroad roadbeds) are not adequate for carrying present and/or anticipated traffic. In recent years, substantial increases in vehicle weights and traffic volumes have been experienced.

One approach for strengthening an existing facility is to increase the strength and/or stiffness of the in situ subgrade. Several procedures (electro-osmosis, electro-chemical, grouting, drill-hole lime, pressure-injection lime, "post-hole pile", thermal treatment) have been evaluated in recent University of Illinois studies (1, 2, 3). Those procedures which appeared most promising for strengthening in situ subgrades were drill-hole lime, pressure injection lime, and "post-hole pile".

Based on recommendations developed in the University of Illinois studies, the Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico, funded the design, construction, and monitoring of several field test sections at Altus Air Force Base, Oklahoma (4). The major objective of the field test section study was to establish whether or not the various techniques, including "pressure injection lime" (PIL), produce a beneficial strengthening effect on the structural response of the pavement system. In this paper, the efficacy of the PIL procedure is considered based on Altus Air Force Base test section data.

TEST SITE SELECTION

Based on the University of Illinois studies (1, 2), the beneficial effects of subgrade strengthening are probably most easily discerned in flexible pavements. It thus seemed desirable to consider flexible pavements in the field testing program.

Two of the promising deep layer stabilization procedures, drill-hole lime and pressure injection lime, have the greatest potential with fine-grained soils. The adverse effects of "Jumbo Jet" loadings will also be accentuated in pavement sections with fine-grained subgrades. The most encouraging field data from pressure injection lime treatment have been achieved in soil formations characterized by the presence of cracks, fissures, joints, etc., characteristic of residual soil profiles.

Based on the above facts, it seemed appropriate to locate the test sections in an area where fine-grained residual subgrade soils predominate.

In order to expedite the construction of the field project, minimize interference with base operations, provide easy access for trafficking and monitoring of instrumentation, the test section site ideally should not be in use.

Fortunately, a site which possessed all of the desirable features was available at Altus Air Force Base (AFB), Oklahoma. Altus AFB is the C-5 training base so it was possible that C-5 trafficking could be used if desired.

The selected test site was an abandoned section of taxiway (Taxiway 10) as shown in Figure 1.

ALTUS AFB TEST SITE

GENERAL

Taxiway 10 is 75 ft wide and has 50 ft of paved shoulder on each edge. As-built record drawings indicate that the present flexible pavement section is as shown in Figure 2. Soils data from Phase II of this project and plan and profile data from the construction plans for the adjacent north-south runway suggest that the subgrade is a CL soil with liquid limits ranging from about 25-39 percent and averaging approximately 30 percent. Bedrock, probably the Hennessey Shale, is encountered at depths of approximately 11 to 12 1/2 ft below the present pavement, based on Phase II field boring data. Late spring, 1970 observations indicated that the water table depth varied from about 5 to 9 ft below the pavement surface.

LABORATORY TESTING OF BULK SUBGRADE SAMPLE

Extensive laboratory testing of a subgrade soil sample obtained adjacent to the shoulder of the north-south runway (Location A in Figure 1) yielded the data presented in Table 1.

The resilient behavior of laboratory compacted samples of the subgrade soil is summarized in Figure 3. The samples were compacted at various water contents to a density equal to 93 percent of AASHO T-99 and tested at various stress levels. The repeated load duration was 0.060 seconds and the load was repeated 20 times per minute. The stress-dependent behavior of the subgrade soil is apparent. Similar response (stress-dependent resilient properties) has been noted for most fine-grained soils. It is obvious that the moisture content of the soil substantially influences the resilient behavior.

Additional subgrade soil data were obtained for thin-walled tube samples taken from the subgrades in the various sections. The data are presented in Reference 4.

LABORATORY TESTING OF PAVEMENT MATERIALS

Bulk samples of the base course, subbase No. 2, and subbase No. 1 were obtained from the pavement. Subbase No. 3 included "salvaged old pavement," was rather heterogeneous, contained very large fragments (as large as 2 inches) and was not sampled. Grain size analysis (ASTM D 422), Atterberg limits (ASTM D 423, ASTM D 424), moisture-density tests (ASTM D 1557), and CBR (ASTM D 1883) tests were conducted with the base course and subbase No. 2 materials. A grain size analysis and Atterberg limits test were conducted with subbase No. 1 which was a "select soil".

Laboratory test data for the bulk pavement material samples are summarized in Table 2. Grain size distribution curves are presented in Figure 4 for the base course and subbase No. 2. The select soil had 66 percent passing the No. 200 sieve and contained 19 percent clay (< 0.002 mm).

TEST SECTION DESIGN CONSIDERATIONS

TEST SECTION SIZE

The primary method of traffic loading used at Altus was the twin-tandem (TT) load cart used at WES in the Multiple Wheel Heavy Gear Load (MWHGL) test sections (5). Based on surface deflection profile data for the flexible pavement sections at WES (6) and allowing some distance for lateral distribution of traffic and longitudinal transitions, the test sections for the TT cart were established to be approximately 25 ft wide and 40 ft long.

TREATMENT DESIGN

At the present time, there are no "rational" procedures for designing PIL treatments for pavement or track systems. Experience generally is utilized to establish the treatment pattern. In most cases, pre-construction and post-construction data are insufficient to quantitatively determine the relative effects of such design parameters as spacing, treatment depth, etc.

The major factors of importance in PIL treatment design are spacing and depth of injection, the nature of the lime-water slurry, and injection pressure.

Injection Spacing

Typical spacings range from 3 ft to 5 ft on centers. If spacings less than 3 ft are used, the degree of pavement disturbance is excessive. The rate of lime migration is very slow and varies as the square root of time (1). To influence the soil mass to the greatest extent in a limited time period, spacing should be minimized.

An additional spacing consideration is that the pressure injected lime-water slurry will be forced along fracture zones, fissures, cracks, etc. that may exist in the soil mass, thus increasing the effective treatment zone. However, substantial slurry movement is not obtained in a "tight soil". Spacings of 3 to 5 ft on center are common in PIL treatment for building foundation work. In conferring with various PIL contractors in the Dallas-Ft. Worth, Texas area, it was found that the amount of lime-water slurry that could be injected per unit volume of soil was independent of injection probe spacing. Thus, probe spacing was not considered as a variable in the Altus test sections and only an intermediate injection spacing of 4 ft was utilized.

Injection Depth

Injection depths are variable, but may be as great as 10 to 12 ft. Injection depths for the Altus AFB test sections included a "deep injection" to a depth of 10 ft below the pavement surface in order to maximize the stabilization benefits and a "shallow injection" to a depth of 5 ft.

Lime Slurry

Contractors in the Dallas-Ft. Worth, Texas area routinely engaged in pressure injection work normally use a lime-water slurry composition of 2 1/2 to 3 lb of lime per gallon of water. A "wetting agent" is also mixed with the slurry in accordance with the manufacturer's recommendations. Apparently a slurry of the above composition is satisfactory, based on extensive field experience. The slurry utilized in the injection process at Altus was a 33 percent hydrated lime concentration (weight basis) and contained a "wetting agent".

Injection Pressure

Although injection pressures as high as several hundred psi can be developed with most lime slurry injection equipment, the majority of the work is injected in the pressure range of 50 to 200 psi. In this pressure range, it is normally possible to disperse the maximum amount of slurry into the soil. The Altus test sections were injected at a pressure of approximately 200 psi.

FINAL TEST SECTION LAYOUT

The final PIL test section design included two PIL treated sections and one control section. Table 3 summarizes the nature of the various test sections.

TEST SECTION INSTRUMENTATION AND DATA COLLECTION

GENERAL

Personnel from the University of New Mexico Civil Engineering Research Facility (CERF) installed the instrumentation and collected the data at the Altus Test Sections. A detailed description of test section instrumentation and data collection can be found in Reference 7.

The general sequence of instrumentation and data collection events that occurred at the Altus AFB test site was as follows:

1. Instrumentation Placement,
2. Load Cart Trafficking,
3. Initial Data Collection,
4. Construction,
5. Post-Construction Data Collection

TEST SECTION INSTRUMENTATION FOR STATIC LOADING

Two types of static pavement response, subgrade deformation and subgrade stress, were monitored at the Altus AFB test sections.

Subgrade Deformation Monitoring

In order to evaluate the "strengthening" potential of the various treatment procedures used in the test sections, it was necessary to monitor the load-induced deformation of the upper layer (about 100 in.) of the subgrade. Bison sensors were used to monitor subgrade deformation.

Subgrade Stress Monitoring

Compressive stress distributions in the test section subgrades were detected using WES type SE soil stress gages installed at strategic locations.

Instrumentation Installation and Locations

In general, both the Bison sensors and WES stress gages were installed in the subgrade of the various test sections by first boring a 12 in. diameter hole through the pavement and into the subgrade to the desired depth. The materials removed from each bore hole were carefully saved and later used to back-fill each hole.

Bison Sensors

In each test section, Bison sensors were installed in vertical "stacks" or arrays at two locations. A plan view of the relative location of each array of Bison sensors is shown in Figure 5. In location B1, Figure 5, the sensors were placed at approximately 7 in. vertical spacings, extending downward from the top of the subgrade for about 100 in. In location B2, the Bison sensors were placed at about 14 in. vertical spacings.

WES Stress Gages

WES stress gages were installed in test Sections 1 and 5. Figure 5 illustrates a plan view of the stress gage locations relative to the treatment points and Bison arrays.

TEST SECTION TRAFFICKING AND STATIC LOADING

Initial Trafficking of Test Sections

After the instrumentation was installed in the test sections, the pavement was subjected to a small amount of trafficking using a twin tandem load cart, Figure 6. Fairly severe pavement distortion was displayed by the pavement after a few coverages; consequently, trafficking was discountinued after 8 coverages.

Description of Load Cart

The load cart used was a twin-tandem assembly described in detail in Ref. 5. General load cart dimensions are given in Figure 6. For initial trafficking, the cart was loaded with lead surcharge so that 30,000 lb wheel loads were obtained and tire pressures were adjusted to 120 psi.

For subsequent "static" loading of the test sections, total weight of the load cart was increased using lead as surcharge until wheel loads of 43,000 lbs were reached. The tire pressure was adjusted to 155 psi.

Static Loading Positions

Static loading of each test section was imposed prior to section treatment and at various intervals following section treatment. For static loading, the load cart was pulled into position at two strategic locations relative to the Bison sensor arrays and WES stress gages and then the instrumentation was monitored. Figure 5 depicts the loading positions relative to the instrumentation and treatment locations.

NONDESTRUCTIVE TESTING

In addition to static loading of the test sections with the load cart, the AFWL nondestructive vibratory testing equipment (NDT) was used to evaluate pavement response. A more detailed description of the characteristics of this equipment can be found in References 7 and 9.

Basically, a 12 in. diameter rigid steel plate is placed on the pavement surface and a repeated load of known magnitude, frequency, and pulse duration is imposed. Maximum plate deflection is measured during loading. For the Altus AFB test sections, deflection under a 5500 lb dynamic load was the primary type of data taken. These data were later converted to dynamic stiffness modulus (DSM) values (DSM = dynamic load ÷ dynamic deflection, lbs/in.). Figure 7 shows the general locations where the NDT tests were conducted.

DATA COLLECTION

Six different periods of data collection occurred at the Altus AFB test site, one prior to the initial trafficking, one prior to treatment of the various sections, then four subsequent to treatment. The data collection periods were:

1. March, 1973
2. June, 1973
3. September, 1973
4. November, 1973
5. March, 1974
6. October, 1974

During these periods, the following types of data were collected:

1. Subgrade deformation monitored with the Bison sensors for static loading positions 1 and 2, Figure 5 (periods 1-5)

2. Subgrade stress monitored with the WES stress gages for static loading positions 1 and 2, Figure 5 (periods 1-5)

3. NDT dynamic deflection values for the locations depicted in Figure 7 (periods 1-6).

Note: Only NDT data were obtained during the October, 1974 data collection period.

TEST SECTION CONSTRUCTION

The construction of the PIL test section was jointly accomplished by the Air Force Weapons Laboratory, the Air Force Civil Engineering Center, University of New Mexico - CERF, and the Woodbine Corporation during the period July-August, 1973. The University of Illinois acted only in an advisory capacity during the construction phase.

PIL treatments were constructed in Sections 5 and 6. The only construction variable examined was depth of injection. The hole spacing remained constant at 4 ft.

The PIL construction sequence consisted of the following:

1. Six in. diameter holes were augered to a depth of approximately 15 in. on 4 ft center-to-center spacings.

2. A small earth dike was placed around each test section to impound the excess lime slurry emitted from the injection holes. This excess lime was later removed.

3. Woodbine Corporation, Ft. Worth, Texas pressure injected the lime-water slurry (33 percent hydrated lime concentration plus "wetting agent") into two holes at a time. A pressure of approximately 200 psi was used and the slurry was injected at approximately 12 in. intervals from the top of the subgrade downward to the desired depth.

4. Sections 5 and 6 were injected with 3815 and 3140 gallons of lime-water slurry, respectively. These quantities are equivalent to approximately 50 gallons per injection hole for Section 5 and 44 gallons per hole for Section 6.

5. After removal of the excess lime from the pavement surface, granular material was placed and compacted in the holes to within 6 in. of the surface.

6. The holes were then filled with a cold mix asphalt patching material and compacted flush with the existing pavement surface.

Section 1 did not receive any treatment and was designated as the control section.

DATA PRESENTATION

GENERAL

Test section response data (static deformations, subgrade stresses, and non-destructive vibratory testing information) were obtained by CERF during the measurement periods previously defined.

STATIC DEFORMATION DATA

In general, static Bison sensor deformation data were obtained for load cart Positions 1 and 2 and Bison sensor arrays 1 and 2 as previously described. A complete data listing has been included in CERF's instrumentation report (7).

The deformation data for the various test sections, testing dates, load positions, and Bison sensor arrays were plotted in the form of cumulative subgrade deformation versus depth below the pavement surface. Plots for Sections 1, 5, and 6 (load position 1, Bison array 1) are shown in Figures 8, 9, and 10.

STATIC SUBGRADE STRESS DATA

Subgrade stress gage data were collected as previously described. Preliminary analysis indicated that the data were often spurious, inconsistent, and apparently erroneous. The data are presented in Reference 7, but have not been further considered in this report. During the instrumentation planning phase of the study, the inadequacies and difficulties associated with monitoring subgrade stresses had been recognized.

NON-DESTRUCTIVE VIBRATORY TESTING DATA

A complete listing of the data is included in the CERF instrumentation report (7). Table 4 is a summary of the dynamic stiffness modulus data based on the average of four readings.

TEMPERATURE AND PRECIPITATION DATA

Temperature and precipitation data for the period January, 1973 through March, 1974 are presented in Figures 11 and 12, respectively. Dates on which pavement response data were collected are indicated in Figures 11 and 12. The data were taken from Altus AFB weather records.

DATA ANALYSIS

GENERAL

Two types of data, nondestructive vibratory testing and static loading deformation information, were available for analysis. The various analyses conducted are described in the following sections.

NON-DESTRUCTIVE VIBRATORY TESTING

Statistical analysis of the pre-treatment data for the Altus test sections indicated that there were no significant ($\alpha = 0.05$) dynamic stiffness differences in the sections. Thus the sections were similar prior to treatment.

Analysis of variance techniques were used to determine:

a) if there were significant time effects for a given section, and

b) if there was a significant difference between the structural response of Sections 5 and 6.

Analysis of variance results for time effects are shown in Table 4. The results indicate that there was no significant time effect for Section 1 (control section, no treatment), but significant time effects are indicated for both Section 5 and Section 6. Included within Section 5 and Section 6 "time effects" data is the influence of treatment. June, 1973 data are based on "pre-construction" information while September, 1973, November, 1973, March, 1974, and October, 1974 data represent "post-construction" conditions. If significant "time effect" differences are obtained, the PIL treatment applied has influenced the structural response of the treated section. Duncan's Multiple Range procedure was used to determine which time period data were different. Results are shown in Table 5.

Since a significant time effect (treatment effect) was noted, Table 4, response data for Sections 5 and 6 were compared to determine if depth of injection (Section 5 - 10 ft depth; Section 6 - 5 ft depth) was a significant parameter. A non significant F value ($\alpha = 0.05$) was calculated indicating that Sections 5 and 6 displayed similar behavior.

DEFORMATION DATA

The subgrade deformation data cannot be treated in a statistical sense. For a given measurement date, the test section pavement response was evaluated at only one Bison sensor column location. Multiple observations are thus not available and only direct quantitative comparisons can be made along the data obtained for the various measurement dates.

To facilitate the analysis of the subgrade deformation data, a response parameter defined as "the area under the depth-subgrade deformation plot for a specified depth range" was developed. This response parameter, called the "Deformation Stiffness Index" (DSI), is a subgrade system response factor as opposed to a "point" measurement. A schematic representation of DSI is shown in Figure 13. Large and small DSI values are indicative of "weak" and "stiff" subgrade support, respectively.

In this study, DSI values were determined based on data for twin-tandem load cart position 1 and Bison sensor array 1 deformations as shown in Figure 5. Areas under the depth-subgrade deformation curves were determined with a planimeter. Scale factors were applied to convert the area data to the appropriate measurement units (depth (in.) x deformation (in.) = in.^{2}). The areas were multiplied by a factor of 100 to obtain the DSI. DSI values for the various test sections and measurement dates are summarized in Table 6.

Analysis of the DSI data for all of the Altus test sections (4) indicated that approximately 68 percent of the DSI was accumulated in the upper 50 in. of the subgrade. Table 7 indicates the percentage of the DSI accumulated in the upper 50 in. for the control and PIL sections for various measurement dates.

Based on an extensive analysis of all of the Altus data (4) significant regression relations were established between dynamic stiffness modulus and DSI values. The relations are shown in Figures 14 and 15. It is important to note that the regression coefficients in Figures 14 and 15 are negative indicating that a decrease in DSI (stiffer subgrade) corresponds to an increase in dynamic stiffness modulus. Pavement behavior theory considerations would show the same trend.

GENERAL DISCUSSION

Analysis of the test data indicates that the PIL deep layer stabilization procedures significantly modified the structural response of the test pavements. The effectiveness of the PIL stabilization procedures is considered below.

CONTROL SECTION BEHAVIOR

The dynamic stiffness modulus data analysis, Table 4, did not indicate any statistically significant differences among the data for Section 1. Static deformation data, Figure 8, and the DSI data, Table 6, likewise did not suggest any substantial differences in the static behavior of the control section. Since there were no statistically significant differences among the various sections prior to construction and the untreated control section did not show significant structural response changes during the time period considered (June, 1973 to October, 1974), it is reasonable to assume that structural response changes in the test sections were due to treatment effects.

PRESSURE INJECTION LIME

Based on the dynamic stiffness modulus data, Table 4, both of the pressure injection lime treated sections (Sections 5 and 6) displayed significant changes with time. Table 5 indicates that there were no significant differences between the June, 1973, September, 1973, and October, 1974 responses. The November, 1973 and March, 1974 dynamic stiffness moduli were significantly greater than the other values. The statistical comparison of the dynamic stiffness modulus values for Sections 5 and 6 indicated that the depth variable (10 ft for Section 5, 5 ft for Section 6) was not a major factor for the Altus AFB test conditions.

Deformation stiffness index (DSI) data for Sections 5 and 6, Table 6 show an initial weakening of the sections following pressure injection lime treatment (probably due to an increase in moisture content in the granular layers and the subgrade), but the DSI values later decreased. The DSI decrease was most pronounced for the 10 ft injection treatment, Section 5.

It should be recognized that further changes in the behavior of the PIL sections may occur as time progresses. Soil-lime strengthening reactions and lime migration are time and temperature dependent. Increased time since construction completion and the advent of higher spring and summer temperatures may be beneficial.

It is important to note in this regard that the October, 1974 dynamic stiffness moduli values for the PIL sections did not show an increase, but were similar to the June, 1973 (untreated response data) and September, 1973 data. It is not known what factors contributed to the loss of dynamic modulus in Sections 5 and 6. No differences were noted among the response data for the control section (Section 1).

SIGNIFICANCE OF IMPROVEMENTS

Dynamic stiffness modulus and static load deformation data indicate that the PIL treatments produced significant changes in the structural responses of the Altus AFB pavements. However, in order to assess the potential applications of deep-layer stabilization procedures, it is necessary to evaluate the effect that the treatments might have on performance. It is therefore essential to examine the relations between structural response and performance.

Only limited information (8) is available for track systems relating structural response to track performance (serviceability of the track as a function of time). More definitive relations have been established for highway and airfield pavements.

Although surface deflections per se were not measured at Altus AFB, substantial reductions in subgrade deformation (compared to September, 1973 post-construction data) were achieved in the PIL sections. Since a substantial percentage of the surface deflection of a flexible pavement is accumulated in the subgrade, it is reasonable to conclude that a reduction in subgrade deformation would correspond to a decrease in surface deflection. Many studies of highway and airfield pavements have shown that decreased deflections are indicative of a longer pavement life. A recent WES study (9) for the Federal Aviation Administration summarized several of the past studies and the various pavement deflection - coverages to failure relations that have been developed. WES (9) has proposed the relation:

$$D = \frac{1.1315}{C^{0.233}} \qquad (1)$$

where:

D = elastic deflection, in.

C = coverages to failure

By rearranging the equation, it is apparent that a small decrease in deflection corresponds to a substantial increase in the number of coverages to failure.

$$C = \frac{1.699}{D^{4.292}} \qquad (2)$$

In the WES study (9), nondestructive evaluation curves were developed for various aircraft. A typical curve for a single-wheel gear is shown in Figure 16 (10). It is apparent that a substantial improvement in flexible pavement performance can be achieved if the DSM is increased. Correlation studies relating CERF dynamic stiffness modulus data to data obtained using the WES 16-kip vibrator (9) indicate that at DSM values less than 1000 kips/inch there is good agreement. Thus, the CERF data for the Altus AFB pavements can be used for entry into Figure 16.

It is apparent that the significance of PIL effected improvements in track system structural response can not be satisfactorily evaluated at this time. Highway and airfield pavement data indicate that only a small decrease in surface deflection or slight increase in dynamic stiffness modulus may be quite beneficial to performance. Perhaps small changes in track system response would improve track system performance in a similar manner.

Deflection and dynamic stiffness modulus effects on performance are "primary" factors. However, other more subtle secondary effects may be equally important in some situations.

In a recent paper (11) Thompson considered many factors that influence the properties and behavior of granular materials in flexible pavements. The concepts developed have equal applicability to ballast and sub-ballast layers in track systems. The importance of:

1. maintaining high levels of confinement in the granular layers,

2. minimizing the loss of interface shear strength at the granular layer-subgrade interface,

3. providing good subgrade support, and

4. reducing the effects of seasonal fluctuations in subgrade support (moisture change, freeze-thaw effects)

were considered in detail.

Thus, PIL treatment may beneficially influence the overall performance of ballast and subballast materials in ways that are not easily discernible from static load-deflection or dynamic stiffness modulus data. The rather subtle PIL treatment effects may be more strongly manifested in track system performance than in structural response parameters.

SUMMARY AND CONCLUSIONS

PIL treatment effected structural response changes (dynamic stiffness moduli) in the Altus test sections. Initial strengthening effects were detected, but the most recent data (October, 1974) indicate that the dynamic stiffness moduli for the PIL sections are not significantly different from the pre-treatment values. No significant structural response changes occurred in the untreated control section (Section 1).

It is not known what factor(s) effected the decreases in dynamic stiffness moduli noted in Sections 5 and 6. Based on soil-lime reaction concepts (12), additional strengthening should be achieved with increased curing time (time since PIL treatment).

The Altus AFB test section data considered in this paper indicate that permanent strengthening effects were not achieved in the PIL treated sections. It is not known what effect increased curing time (beyond October, 1974) has produced.

Since the Altus test sections were not subjected to trafficking following construction, it was not possible to determine the influence of PIL treatment on pavement performance. Perhaps, as discussed earlier in this paper, PIL treatment affects transportation support system performance without producing major discernible changes in structural response.

It is emphasized that the findings of this study are based on only the Altus AFB test sections. Indiscriminate and broad scale extrapolation of the findings to other soil types, geographical areas, etc. is not advised.

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12. Robnett, Q. L., and Thompson, M. R., "Alteration of Soil Properties as Effected by Various Lime Treatment Procedures," paper presented at Roadbed Stabilization Lime Injection Conference, University of Arkansas - Federal Railroad Administration, Little Rock, Arkansas, August, 1975.

TABLE 1

Properties of Altus Subgrade Soil
(Bulk Sample)

1. Unified Classification - CL
2. AASHO Classification - A-6(13)
3. Plasticity Properties
   a) LL - 39
   b) PI - 23
4. % Passing #200 - 79
5. % < 2μ clay - 41
6. AASHO T-99 Compaction
   a) Maximum dry density - 110.0 pcf
   b) Optimum moisture content - 16.4%
7. AASHO T-180 Compaction
   a) Maximum dry density - 122.3 pcf
   b) Optimum moisture content - 13.2%
8. CBR (AASHO T-99 Compaction)
   a) Unsoaked - 6.4
   b) Soaked - 3.0
9. Unconfined Compressive Strength (AASHO T-99 maximum density and optimum moisture content) - 54 psi
10. Modulus of Elasticity (test conditions same as 9) - 1900 psi

TABLE 2

Pavement Material Data

Material	Liquid Limit, %	Plasticity Index, %	CBR, % Unsoaked	CBR, % Soaked	$\gamma_D$max, pcf[1]	Optimum Moisture Content, %[1]
Base Course	22.8	6.2	73	69	137.7	6.6
Subbase No. 2	—	NP	40	26	120.4	8.4
Subbase No. 1	22.1	10.2	—	—	—	—

Notes:

1. AASHO T-180 Compaction

TABLE 3

Summary of Deep Layer Stabilization
Test Section Designs

Section	Section Width, ft	Section Length, ft	Treatment Details
1	25	40	Control - no treatment
		Pressure Injection Lime	
5	24	40	4 ft c-c spacing, 10 ft depth
6	24	40	4 ft c-c spacing, 5 ft depth

TABLE 4

Summary of Dynamic Stiffness Modulus Data

	Average Dynamic Stiffness Modulus, kips/inch[1]						
Section No.	June, 1973	Sept., 1973	Nov., 1973	March, 1974	Oct., 1974	Grand Mean[2]	F Value[3]
1	546	606	701	654	624	626	2.71
5	534	523	692	705	575	606	11.15*
6	539	514	672	699	544	594	35.24*

Notes:

1. Average of four determinations.

2. Average based on June, 1973; November, 1973; March, 1974, and October, 1974 data.

3. Analysis of variance results

* Significant @ $\alpha = 0.05$

TABLE 5

Dynamic Stiffness Modulus Analysis
Duncan's Multiple Range Test
(Time Effects)

Section No.	Ascending Order Listing of Means (Kips/Inch)				
5	523(2)	534(1)	575(5)	692(3)	705(4)
6	514(2)	539(1)	544(5)	672(3)	699(4)

Notes:

1. ( ) Indicates Measurement Period
   - (1) June, 1973
   - (2) September, 1973
   - (3) November, 1973
   - (4) March, 1974
   - (5) October, 1974

2. Any two means not underscored by the same line are significantly different ($\alpha = .05$).

   Any two means underscored by the same line are not significantly different ($\alpha = .05$).

TABLE 6

Summary of Deformation Stiffness Index Data

Deformation Stiffness Index (DSI)

Section No.	Total-Upper 100 Inches				Upper 50 Inches				Lower 50 Inches			
	J	S	N	M	J	S	N	M	J	S	N	M
1	768	794	748	865	148	265	284	368	50	69	69	45
5	303	671	484	381	84	213	155	123	24	63	44	28
6	394	652	626	535	129	226	213	161	130	81	38	44

Notes:

J - June, 1973; S - September, 1974; N - November, 1973; M - March, 1974.

Static data were not obtained in October, 1974.

TABLE 7

Deformation Stiffness Index Accumulation Data

% DSI Accumulated in 0-50 Inch Depth[1]

Section No.	June, 1973	Sept., 1973	Nov., 1973	March, 1974
1 (control)	74.7	79.3	80.5	89.1
Pressure Injection Lime				
5	77.8	77.1	77.9	81.5
6	49.8	73.6	84.9	78.5

Notes:

1. $$\frac{(0\text{-}50 \text{ Inch DSI})}{(0\text{-}50 \text{ Inch DSI}) + (50\text{-}100 \text{ Inch DSI})} \times 100 = \text{\% DSI Accumulated in 0-50 Inch Depth}$$

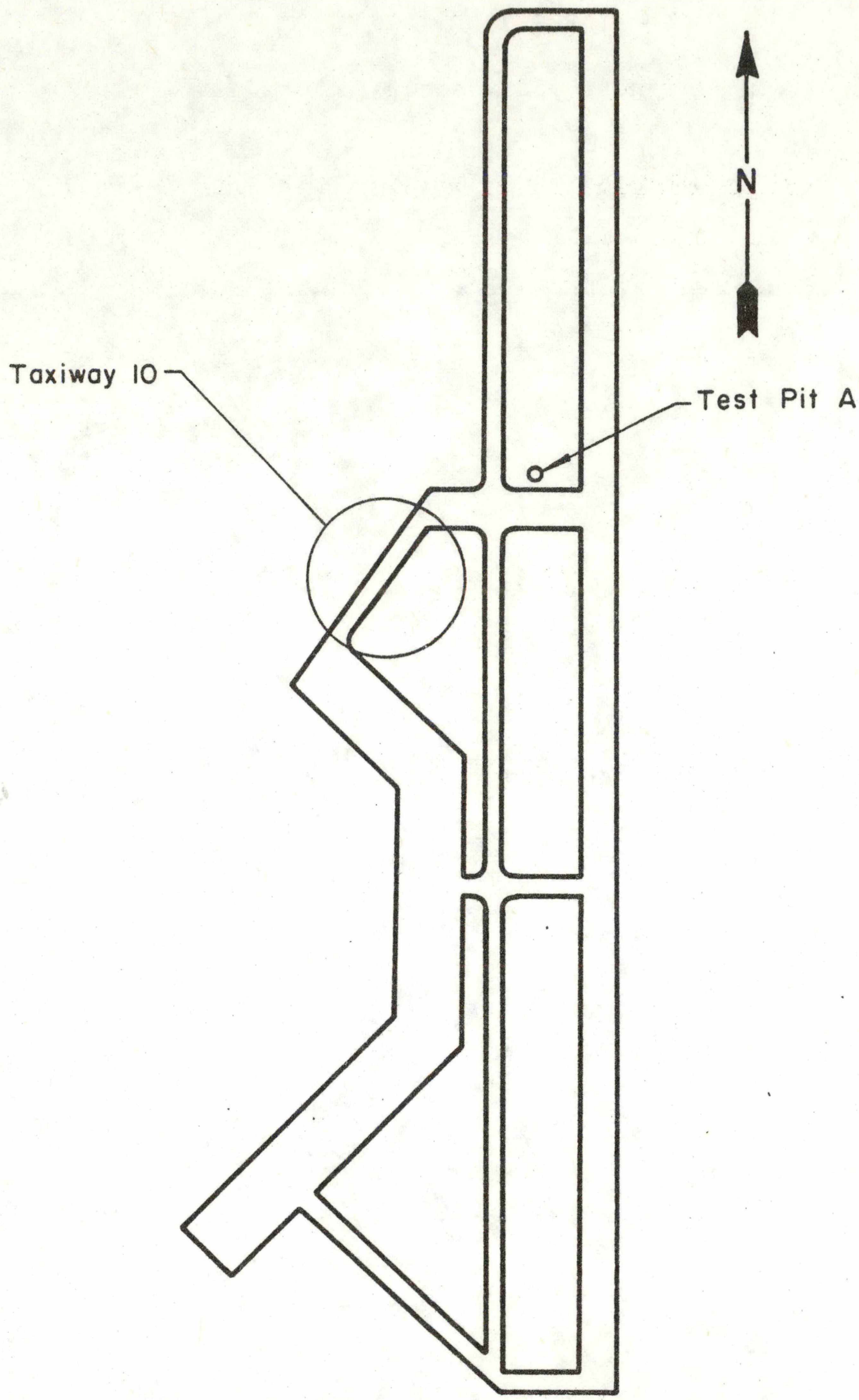


Figure 1. Test Section Site Location - Altus Air Force Base, Oklahoma

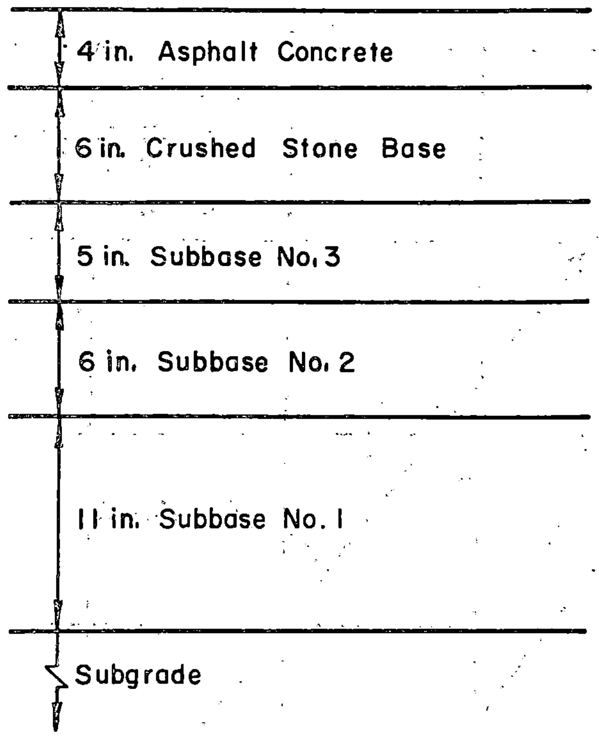


Figure 2. Typical Flexible Pavement Section - Taxiway 10 - Altus AFB, Oklahoma

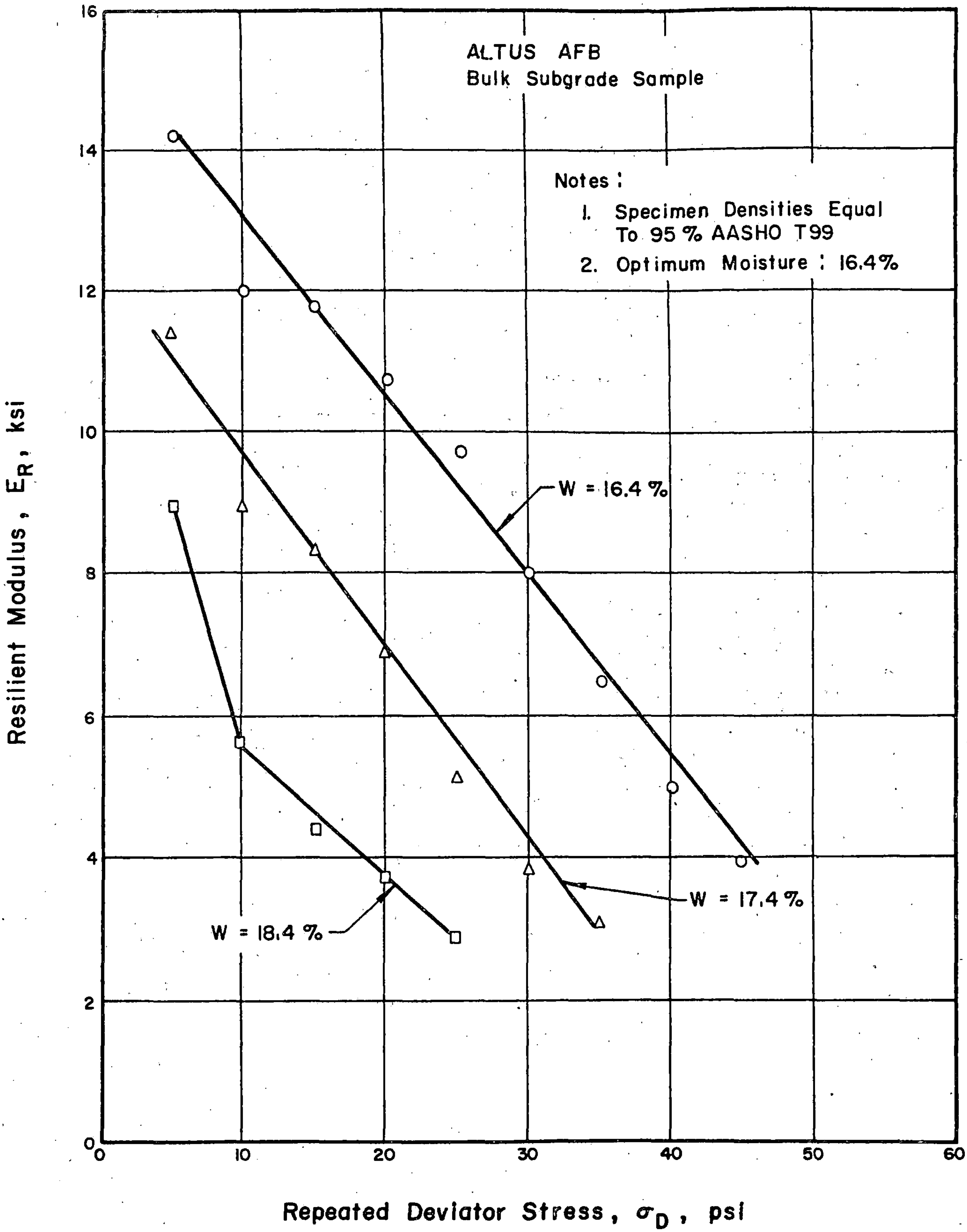


Figure 3. Resilient Behavior of Altus AFB Bulk Subgrade Sample

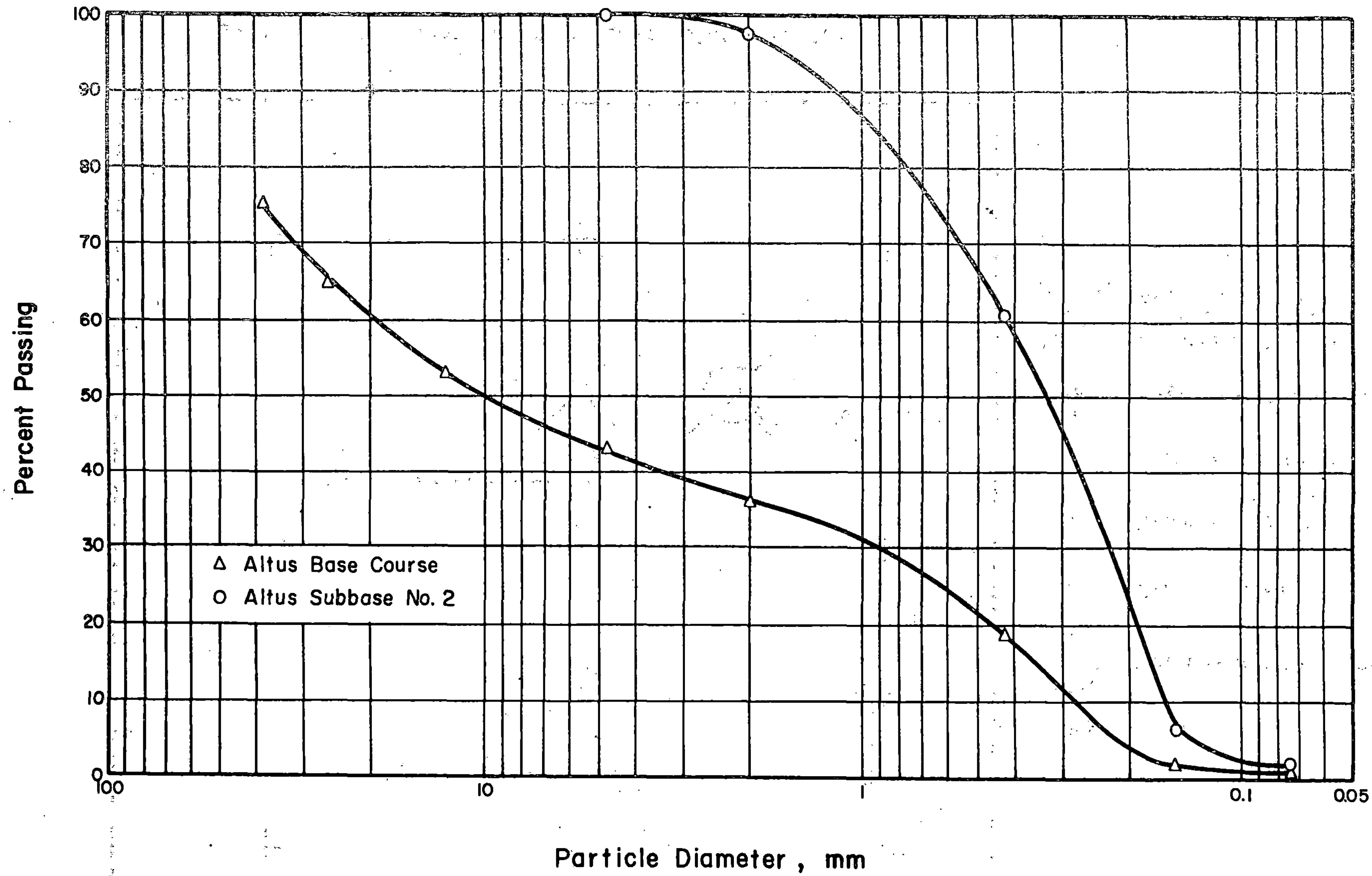


Figure 4. Grain Size Distribution Curves for Altus AFB Base Course and Subbase No. 2 Materials

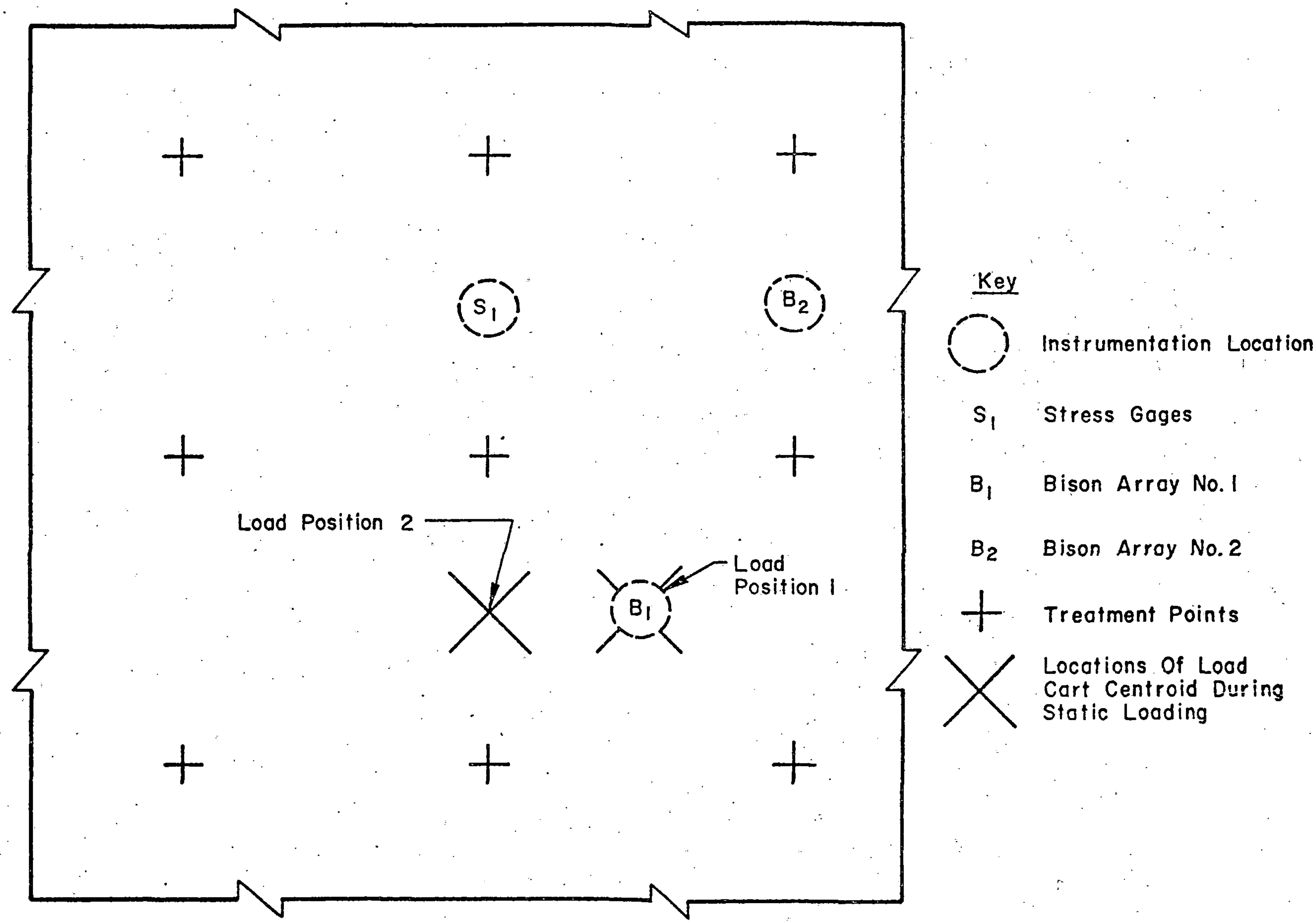


Figure 5. Instrumentation and Load Point Locations

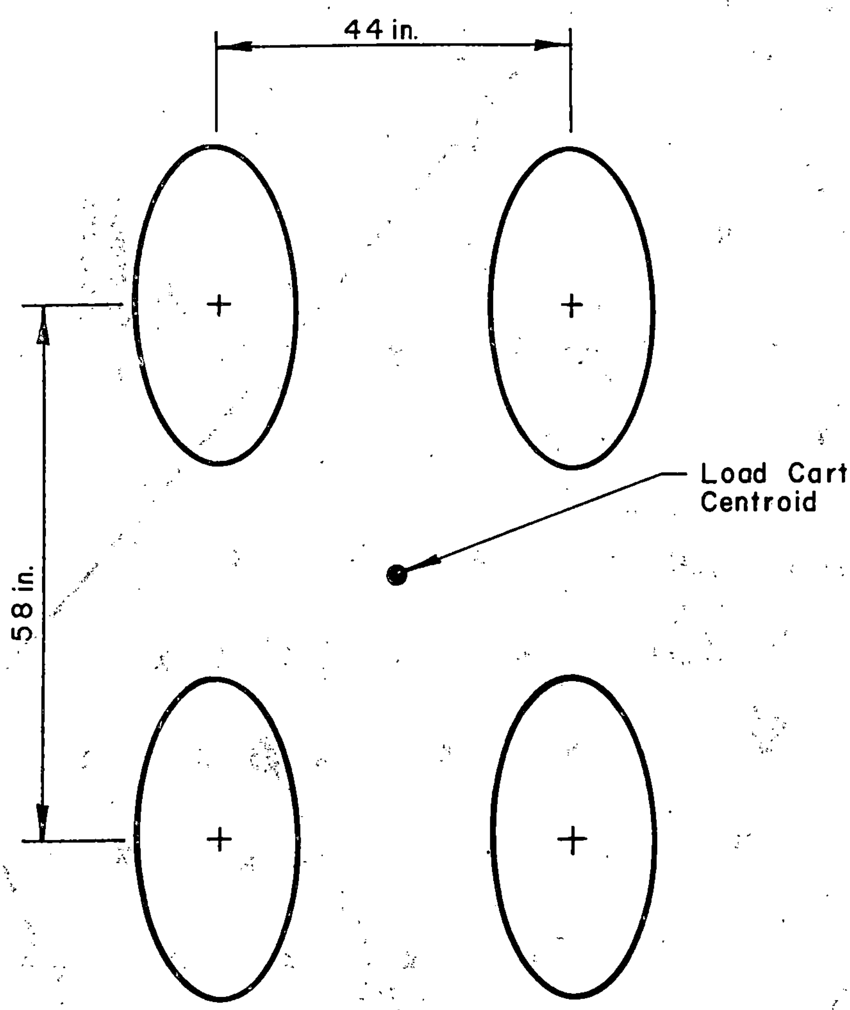


Figure 6. Details of the Twin-Tandem Load Cart

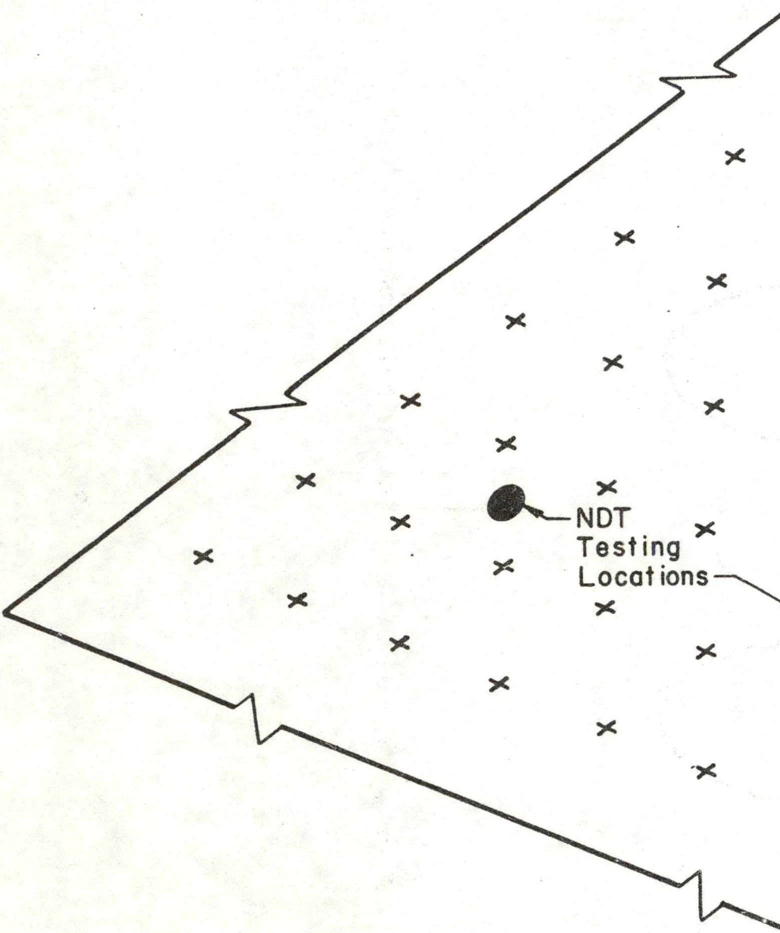
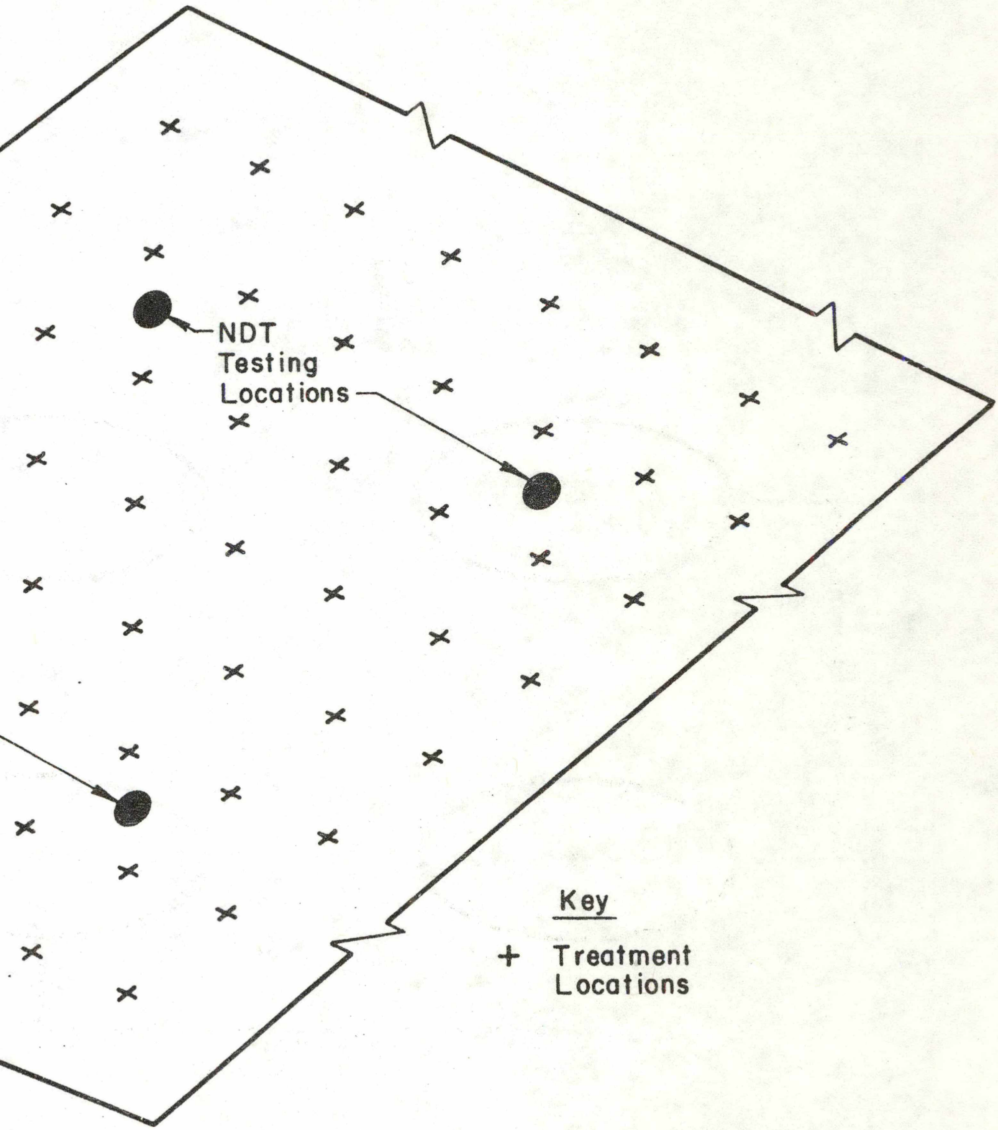


Figure 7.



NDT Testing Locations

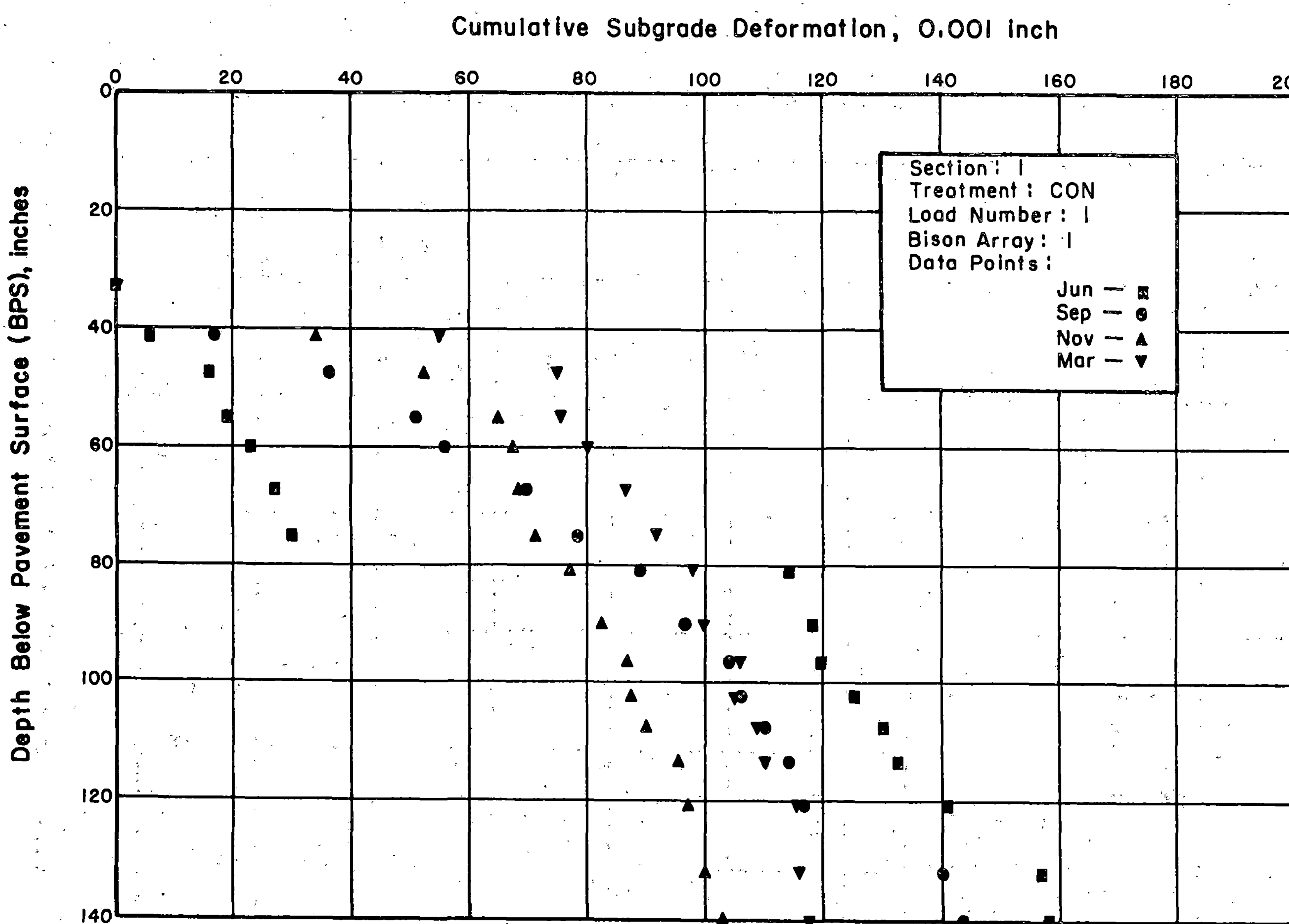


Figure 8. Cumulative Subgrade Deformation - Section 1, Load No. 1, Bison Array 1

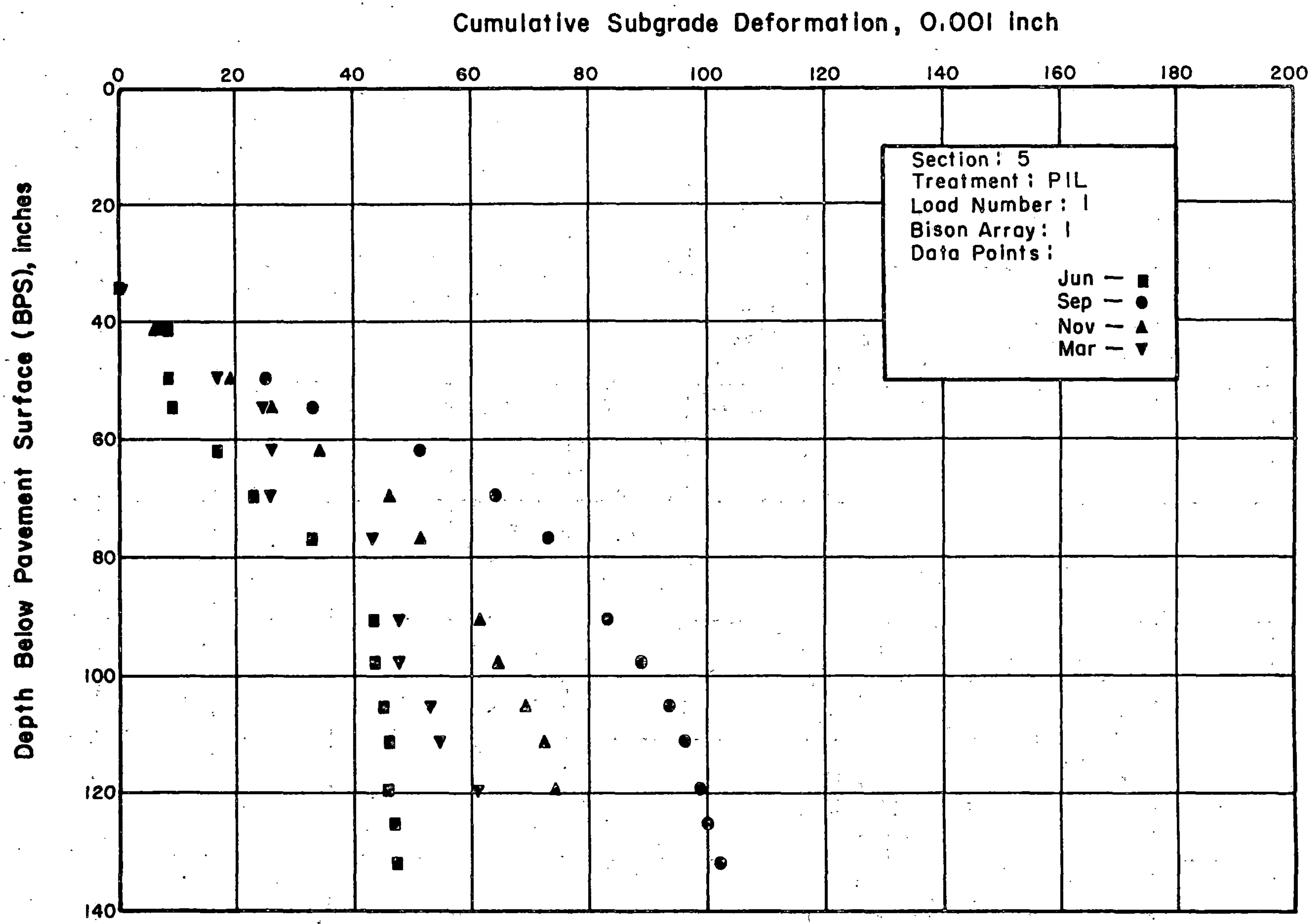


Figure 9. Cumulative Subgrade Deformation - Section 5, Load No. 1, Bison Array 1

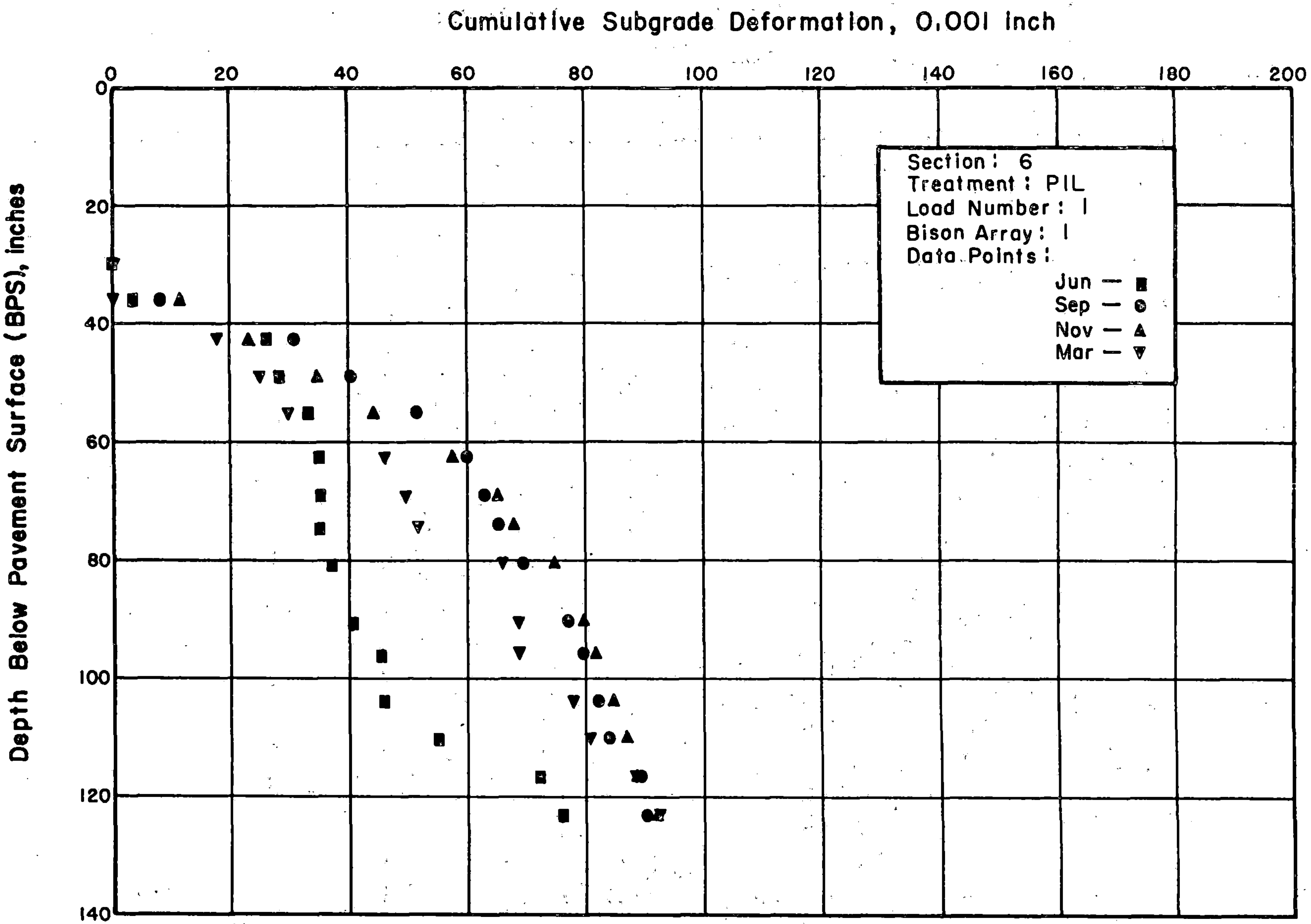


Figure 10. Cumulative Subgrade Deformation - Section 6, Load No. 1, Bison Array 1

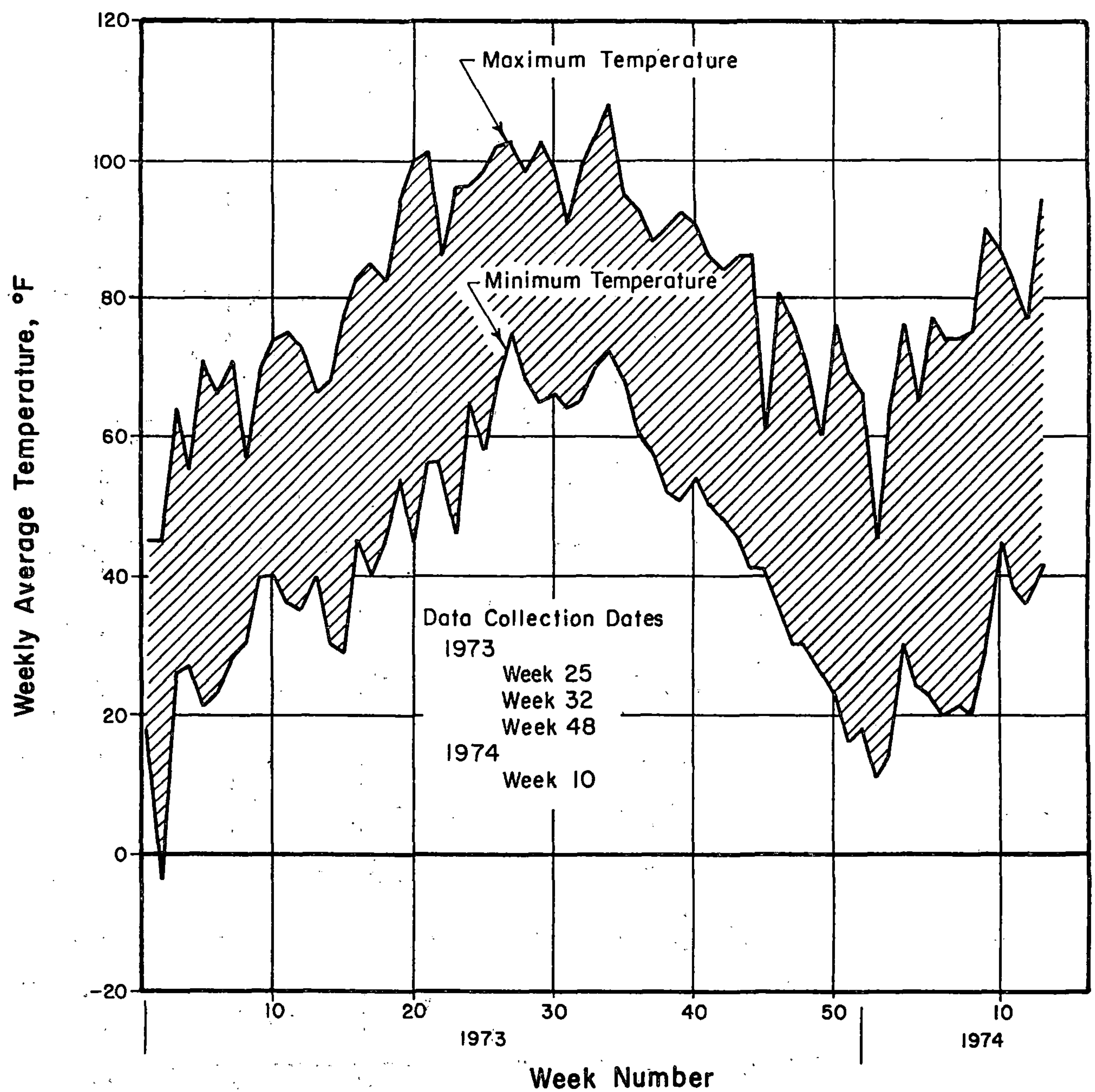


Figure 11. Weekly Average Temperature Data for Altus AFB

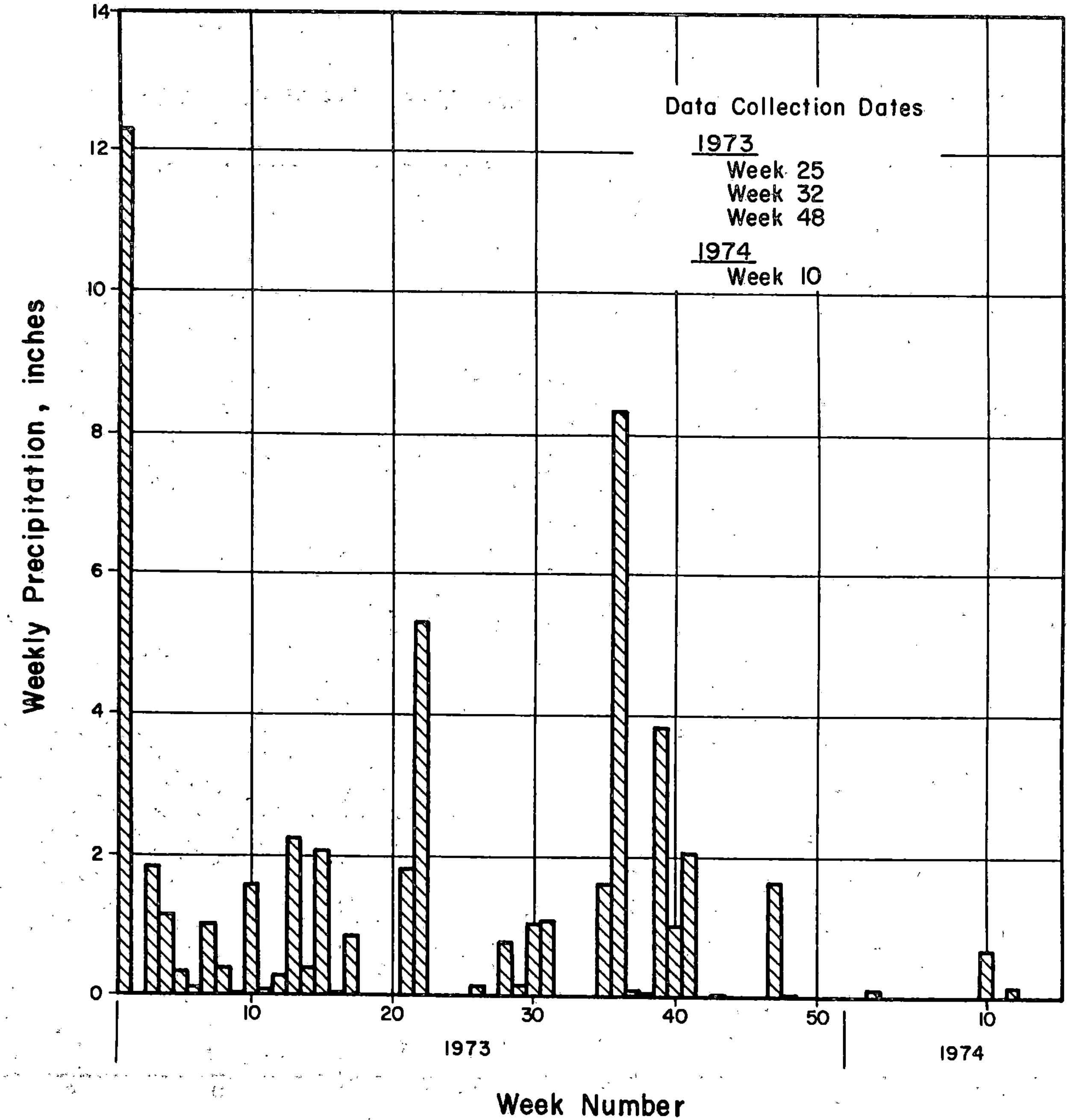


Figure 12. Weekly Precipitation Data for Altus AFB

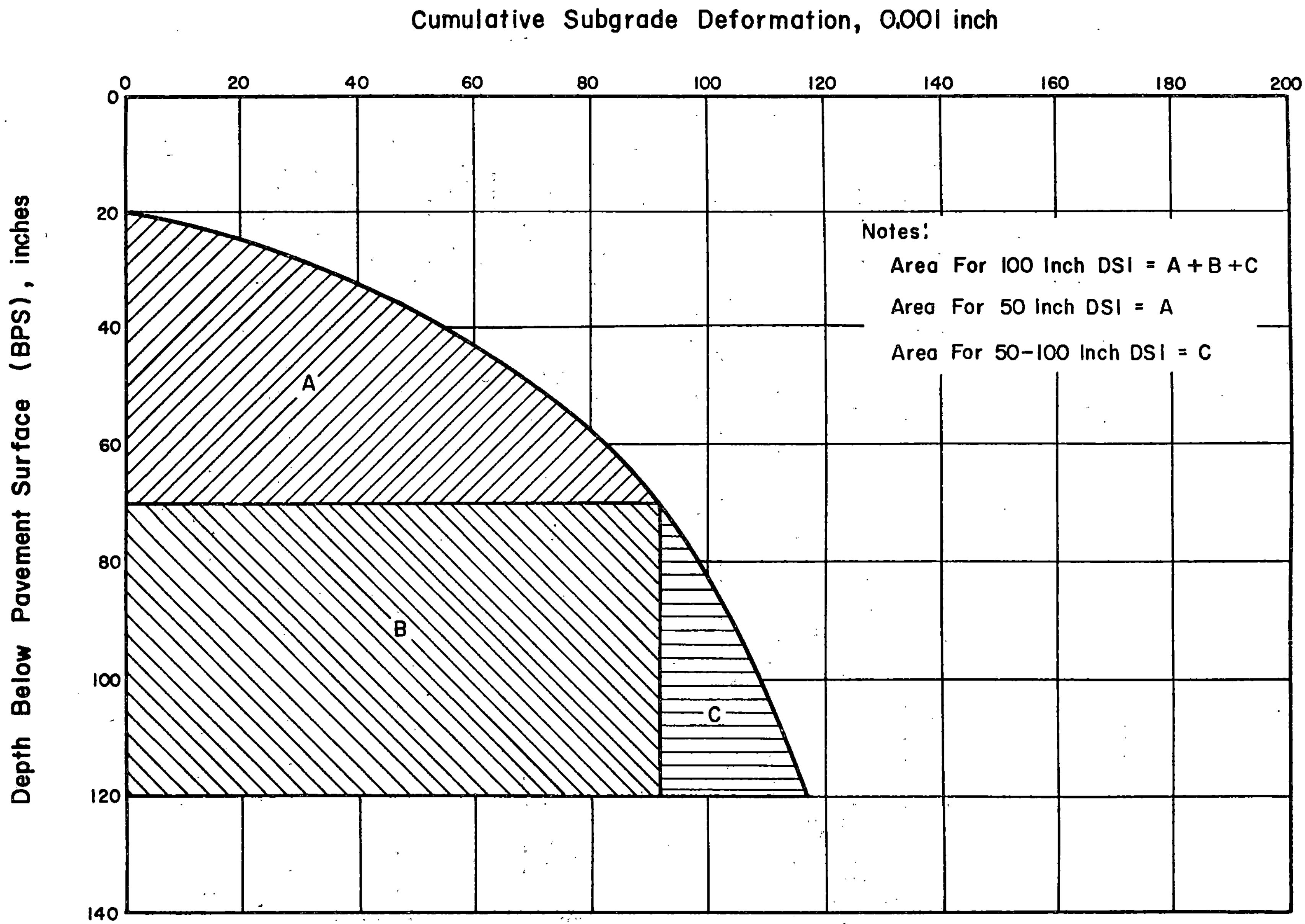


Figure 13. Schematic Representation of the "Deformation Stiffness Index" (DSI)

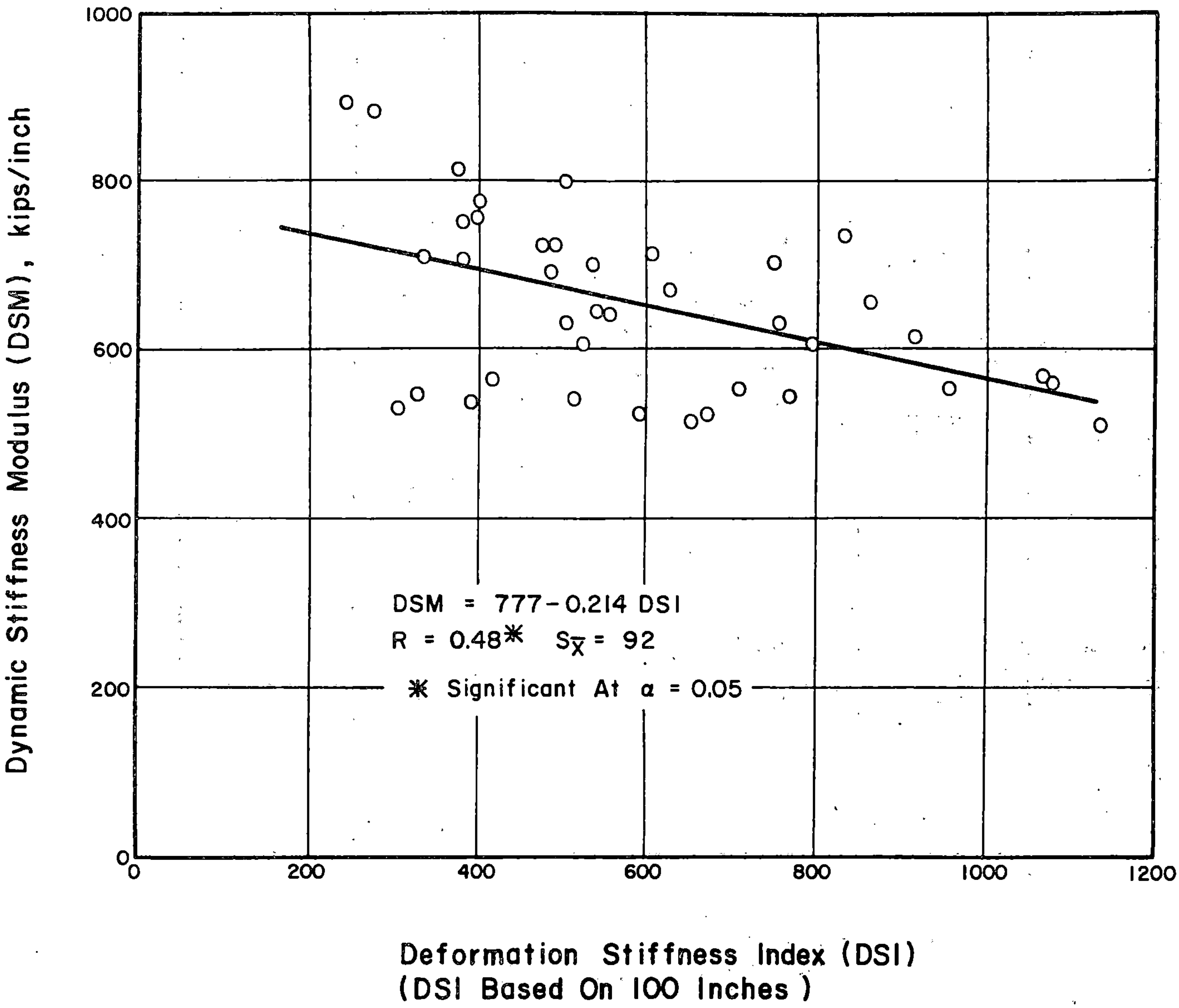


Figure 14. Dynamic Stiffness Modulus - Deformation Stiffness Index Relation

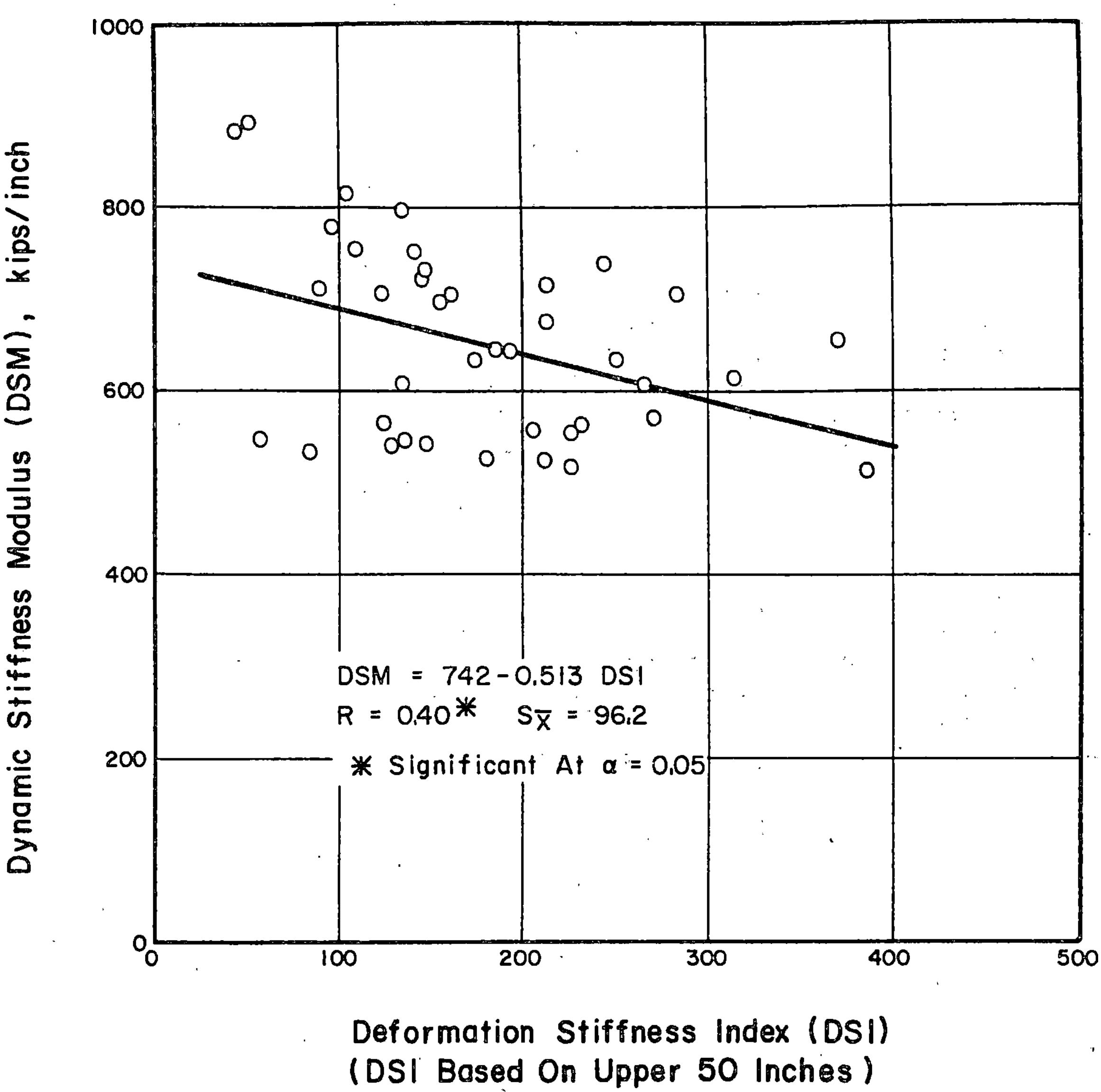


Figure 15. Dynamic Stiffness Modulus - Deformation Stiffness Index Relation

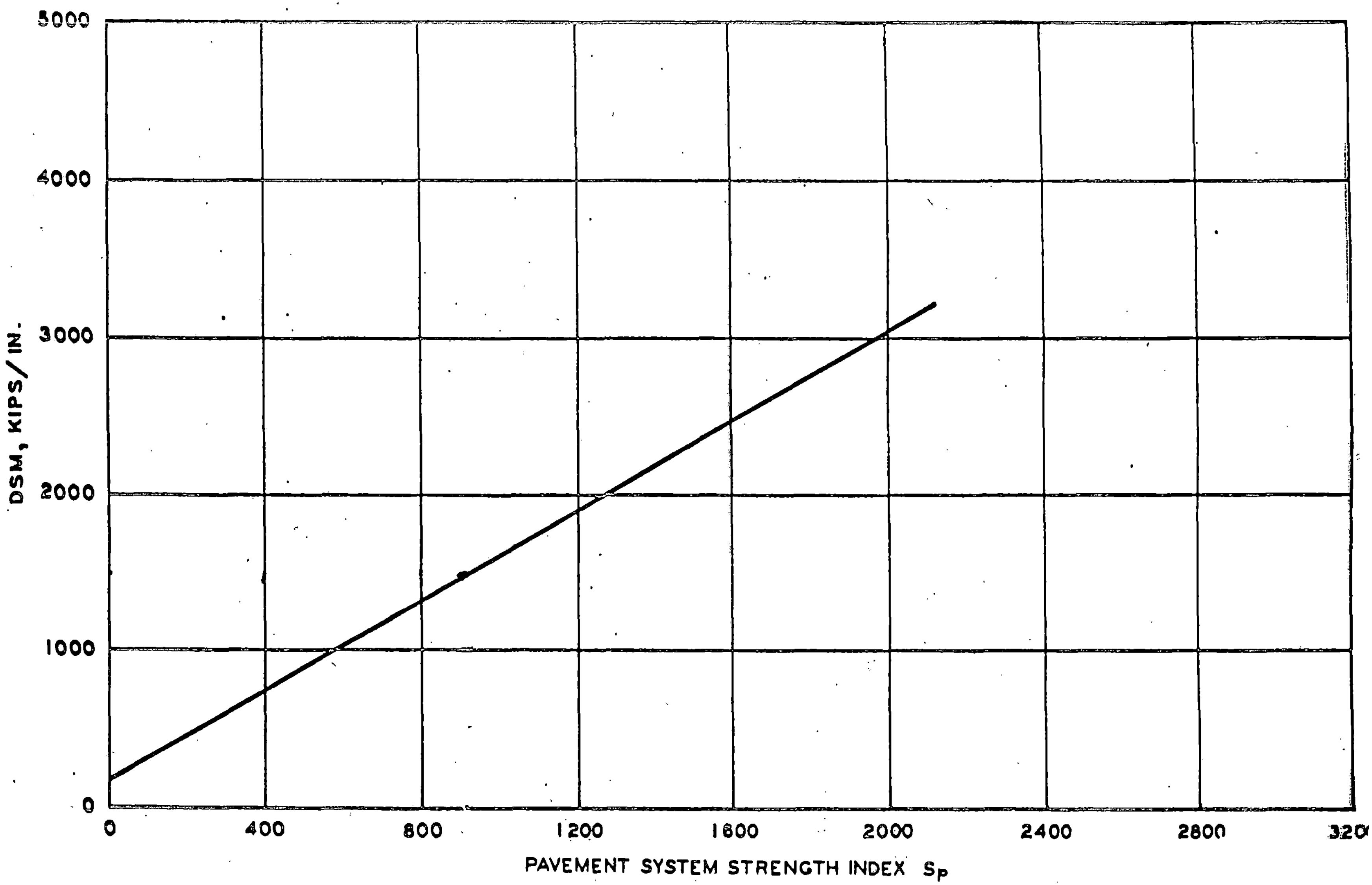


Figure 16. FAA Evaluation Curve for Flexible Pavement (Ref. 10).

ALTERATION OF SOIL PROPERTIES AS EFFECTED BY VARIOUS LIME TREATMENT PROCEDURES

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

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August 21 and 22, 1975

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INTRODUCTION

In recent years, many track structures have been deteriorating at an accelerated rate. Major contributing factors include a) increased weight and volume of traffic, b) inadequate ballast thickness, c) soft, pumping subgrades, and/or d) inadequate level of maintenance and upkeep. As the situation exists, many track structures are in need of major maintenance and/or rehabilitation in order to return them to a reasonable service condition.

Improvement of the structural behavior characteristics (rail deflection, rail moment, track modulus, subgrade strain, etc.) of the track system is badly needed in many cases. Numerous techniques for strengthening track structures have been tried with varying degrees of success. One method which is gaining popularity is subgrade stabilization. For new construction or for major reconstruction conventional stabilization with lime appears practical. However, for many in-service track structures, it may be impractical to take the track structure out of service long enough and/or to provide the funding to accomplish subgrade stabilization with conventional stabilization procedures which require exposure of the subgrade.

For a number of years, drill-hole lime (DHL) and pressure injection lime (PIL) stabilization techniques have been used in highway and airfield pavements applications for treating inaccessible in-situ soil masses. Lime or lime-water slurry is placed in shafts or injected into the soil mass. An intimate dispersal or distribution of the lime into the soil mass is not normally accomplished. However, migration/diffusion processes cause further lime distribution. Recently, pressure injection lime has been tried with railroad track structures.

The purpose of this paper is to evaluate the extent of soil property alteration effected by various lime treatment procedures.

LIME STABILIZATION PROCEDURES

Commonly, there are three techniques used to effect lime stabilization; these are:

1. Conventional mixed-in-place procedure,
2. Drill-hole lime(DHL), and
3. Pressure injection lime (PIL).

The DHL and PIL techniques are normally used when it is impractical to expose the subgrade for conventional mixed-in-place lime stabilization.

Conventional Procedure

By far, the majority of lime stabilization work is accomplished through a fairly standard procedure in which a) the soil is pulverized, b) lime is distributed and intimately blended with the soil (at desired moisture level), c) lime-soil mixture is compacted, and d) if the soil is reactive, the mixture may be cured to allow for additional strength gains. Details of this conventional procedure are covered in Reference 1.

Drill-Hole Lime (DHL)

The DHL technique basically consists of introducing quick or hydrated lime or a lime-water slurry into holes drilled in the soil mass. In highway and airfield pavement applications, small diameter holes (6-12 inches in diameter) are advanced through the pavement into the subgrade soil. Typically, the depth of drill holes range from 30-50 in. and hole spacings are about 4 to 5 ft. After the hole is made and the lime placed within, the hole is backfilled with soil or aggregate.

Once placed in the holes, the lime (usually hydrated lime in a slurry form) apparently migrates or diffuses into the soil system. On a volume basis, it is obvious that the lime is very poorly distributed in the soil mass.

Pressure Injection Lime

In an attempt to achieve better lime distribution in the soil mass, the pressure injection lime procedure was developed. In the PIL procedure, a lime-water slurry is pumped under pressure through hollow injection rods into the soil. Generally, injection rods spaced at 3 to 5 ft are pushed into the soil and slurry is injected at about 12 in. intervals.

The normal lime-water slurry composition is 2 1/3 to 3 lb. of lime per gallon of water with a "wetting agent" added in accordance with the manufacturer's recommendation.

Although injection pressures as high as several hundred psi can be developed with most lime slurry injection equipment, the majority of the work is injected in the pressure range of 50 to 200 psi. Injection depths are variable, but current equipment is capable of injecting at least 10 ft.

The degree of lime distribution accomplished in various soil masses may be quite variable, depending on the nature of the soil deposit. Soils with cracks, fissures, varves, etc. allow for better slurry distribution than "tight" soils.

Following initial deposition of the lime, migration/diffusion occurs increasing the size of the soil mass affected by lime.

LIME STABILIZATION MECHANISMS

Based on the previous discussion, it is obvious that the extent of lime distribution effected by these three treatment procedures is different. However, irrespective of the amount of distribution, when lime comes in contact with a fine-grained soil, certain reactions may be initiated.

Cation exchange and agglomeration/flocculation reactions take place rapidly and produce immediate changes in soil plasticity, workability, swell and immediate (uncured) strength and deformation properties. Dependent on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may commence. The pozzolanic reaction results in the formation of various types of hydrated calcium silicate and calcium aluminate cementing agents, or both. The cementing agents increase mixture strength and durability. Pozzolanic reactions are time dependent and strength development is gradual but continuous for a long period of time (several years in some instances). Temperature also affects the pozzolanic reaction. Temperatures less than 55-60F retard the pozzolanic reaction and increased temperatures will accelerate the reaction.

It is emphasized that these reactions occur at positions where lime is in contact with the fine-grained soil, i.e., around the periphery of a drill-hole or along the seams created during pressure injection of lime. As will be pointed out subsequently, migration/diffusion of the lime occurs, thereby increasing the volume of soil mass affected.

PROPERTIES OF LIME-TREATED SOILS

When lime is blended with a fine-grained soil, a number of reactions commence. As a result of these reactions, significant changes in a number of pertinent soil properties occur.

Plasticity

Substantial reduction in plasticity (reduced plasticity index (PI), increased shrinkage limit) is effected when lime is blended with a soil and in some cases the soil may become nonplastic, Figure 1. Generally, high initial PI and clay content soils require greater quantities of lime for achieving the nonplastic condition, if it can be achieved at all.

Volume Change

Swell potential and swelling pressures are normally significantly reduced in soil-lime mixtures. The reduced swell characteristics are generally attributed to decreased water affinity of the calcium saturated clay and the formation of a cementitious matrix which can resist volumetric expansion. CBR swell values of lime treated soils vary, but it is not uncommon to decrease swell to less than 0.1 percent (2). Thompson and Robnett have examined the use of PIL for control of swelling soils (3).

Strength and Deformation Properties

The strength and deformation properties of lime-treated soils are dependent on many variables. Soil type, lime type, lime percentage, and curing conditions (time-temperature) are the most important. The properties of a lime-treated soil are therefore not "static values" but will vary in response to changes in the variables listed above.

In the following discussion, a distinction is made between "uncured soil-lime mixtures" and "cured" soil-lime mixtures. The uncured mixtures experience the immediate effects of the rapid cation exchange and flocculation/agglomeration reactions but the time and temperature dependent soil-lime pozzolanic reaction does not occur without the benefit of curing. In contrast, the cured mixtures not only experience the cation exchange and flocculation/agglomeration reactions but also, if the soil is reactive, the soil-lime pozzolanic reaction will proceed with curing.

a. Unconfined Compression and Flexural Strength

Unconfined compressive strength of typical fine-grained soils ranges from about 25 to 100 psi. Depending on the reactivity of the soil, cured strength (28 days at 73F) may vary substantially (4, 5); strength increases for Illinois soils have ranged up to 265 psi with many soils displaying increases greater than 100 psi. In a recent study (6) a number of soils from the southeastern part of the United States displayed compressive strength increases in excess of 200 psi when treated with lime. The flexural strength of lime soil mixtures is about 25 percent of the unconfined compressive strength (2).

b. Shear Strength

The major effect of lime on the shear strength of reactive fine-grained soils is to produce a substantial increase in cohesion with some minor increase in ϕ. Typical reactive soils display ϕ values ranging from 25^{o}-35^{o} (7). Cohesion can be predicted based on unconfined compression results (7).

$$\text{cohesion (psi)} = 9 + 0.29 \text{ unconfined strength (psi)}$$

c. Modulus of Elasticity

The marked effect of lime on the compressive stress-strain properties of fine-grained soils is shown in Figure 2. The failure stress is increased and the ultimate strain is decreased for soil-lime mixtures relative to the natural soil. For uncured mixtures, data presented by Neubauer and Thompson (8) and shown in Figure 3 typify the effects of lime on strength characteristics. The compressive modulus of elasticity of a cured soil-lime mixture can be approximated by the following relation (7):

$$E \text{ (ksi)} = 9.98 + 0.124 \text{ unconfined compressive strength (psi)}$$

Thompson (2) states that in general, the flexural modulus of elasticity is substantially greater than the compressive modulus.

Suddath and Thompson (9) found that the resilient moduli of a cured lime soil mixture (Goose Lake Clay and 4 percent lime) ranged from 75 to 125 ksi. Static modulus of deformation values for the same mixtures ranged from 16 ksi to 45 ksi (9).

Robnett and Thompson (10) found that resilient moduli for uncured lime soil mixtures generally range from 12-20 ksi while for untreated soils the resilient moduli normally range from 3-12 ksi, Figure 4. Resilient moduli values for cured soil lime mixtures were found to be even higher (10).

d. California Bearing Ratio (CBR)

CBR and swell values for untreated and uncured and cured lime treated soils are typified by the data summarized in Table 1. These data indicate that the uncured CBR value of lime treated soils (reactive and non reactive soils) is generally substantially greater than the untreated soil and commonly ranges from 10-25. Note also that upon curing, the CBR values further increase and in some cases display CBR values in excess of 100.

e. Durability

Durability is the ability to retain structural integrity and is an important property of stabilized materials. Moisture increases and frost action can cause substantial strength reduction in nondurable materials.

Quality lime soil mixtures display soaked compressive strength on the order of 70-85 percent of the unsoaked values (11). Cyclic freeze-that action may decrease the strength of lime stabilized soil mixtures. Damage is normally characterized by volume increase and strength reduction. Figure 5 depicts typical unconfined compressive strength loss of lime soil mixtures as a function of freeze-thaw cycles and initial (0 cycle) strength.

Robnett and Thompson (10) demonstrated that the immediate (uncured) effects of lime treatment successfully counteract closed system freeze-thaw induced reduction in resilient moduli values, Figure 6.

DHL AND PIL TREATMENT MECHANISMS

From the previously presented information, it is apparent that two major lime distribution mechanisms are of concern relative to DHL and PIL stabilization procedures.

Lime Injection

Lime distribution by the PIL technique obviously is related to the capacity to "permeate" the soil mass with the lime slurry. The quantity of slurry that can be forced into a soil mass during a given interval of time can be approximated by an equation of the following form:

$$Q = \frac{\rho g}{\eta} \left(\frac{p}{\ell}\right) K \quad A \quad t \tag{1}$$

where: A = cross sectional area over which pressure acts

η = viscosity of fluid

g = acceleration due to gravity

p = pressure head

K = intrinsic permeability of medium; $K = Cd^2$
C is a shape factor
d is average pore size of medium

ℓ = length over which pressure head acts

Q = quantity of fluid flow

ρ = density of fluid

t = time of pressure application

An examination of Equation 1 indicates that the following factors exert major control over the quantity of "fluid" injected:

1. "fluid" viscosity,
2. injection pressure and time, and
3. intrinsic permeability of the soil medium

Since lime slurry is not an "ideal" fluid but rather a particulate suspension, the pore size distribution of the soil mass is an important consideration in the permeation process. Successful injection of the lime slurry into the soil mass would require that channels larger than the lime particles be present. The inherent pore size of most fine-grained soils is quite small relative to the lime particle size. Thus, appreciable lime movement through these pores is questionable although it is possible that a substantial quantity of water and dissolved lime may move into the pores.

Even though the permeability of the soil resulting from the inherent soil pore structure may be quite low, the "mass permeability" or conductivity of the soil may be substantially higher as a result of the presence of seams, fissures, cracks, varves, etc. and jetting or tearing of the soil effected by the pressure injection process. When high mass permeability exists, the potential for successful lime slurry injection of a "soil mass" is greatly enhanced.

Lime Diffusion/Migration

Once a lime source is placed in a soil mass it appears that migration/diffusion processes effect further distribution of the lime. Space limitations do not allow presentation and discussion of diffusion/migration theory. In general, however, factors such as differences in clay content, clay minerals, density, absorbed cations, and temperature have been found to affect the rate of diffusion (12, 13, 14, 15).

In an early study (16) Davidson, Demirel, and Handy suggested that the diffusion of calcium cations in a soil-lime water system is an example of the diffusion phenomena. They stated that the processes accompanying lime diffusion may include: a) transfer of lime into the soil, b) a chemical reaction between the lime and the soil, c) formation of nuclei and growth of the reaction produce, and d) further diffusion of the lime into the soil from the reaction product layer(16).

Limited laboratory and field studies have been conducted to evaluate the rate and extent of "lime migration." In a controlled laboratory study conducted by Fohs and Kinter (18), about 0.8 percent lime was found to migrate approximately 1 1/2 to 2 in. after 180 days. They concluded that the migration process for effecting translocation of lime in a soil system is very slow and that only very small amounts of lime can be translocated thereby rendering this process impractical for effecting substantial "soil mass" strength increases (18). Robnett, et al. (30) conducted a limited laboratory lime migration study. Typical results are shown in Figure 7.

In a field study, Lundy and Greenfield (19) found that after 1 year, approximately 3/4 to 1 1/2 in. of lime migration had occurred away from the lime seams. A Louisiana Department of Highways Study (20) found that about 1/2 to 1 1/2 in. of lime migration occurred after 4 years.

The following expression has been suggested for determining the rate of growth of a produce layer from the lime source (16, 17).

$$d = k_d \sqrt{t} \tag{2}$$

where:

d = distance of lime migration for a time t, inches

k_d = diffusion constant, in./day$^{1/2}$ (reported values range from 0.081 to 0.63 in./day $^{1/2}$ (16, 17).

t = elapsed time of diffusion, days.

If Equation 2 is used to estimate the required time for various distances of migration, the following is found (if k_d is assumed to be 0.10 inch/day$^{1/2}$):

d, inches	time, t
1	100 days
6	3600 days
12	40 years

Based on the previous information, it appears that lime translocation by diffusion/migration does occur but the process is very slow.

Diffuse Cementation

Recent comprehensive studies by Stocker in Australia (21) have led to the development of an integrated theory of soil-lime stabilization reactions termed "diffusion and diffuse cementation." "Diffuse cementation" theory as proposed by Stocker describes a process in which lime will diffuse into a natural soil lump. Based on his studies, Stocker stated, "The diffused lime is shown to react with all the clay present, including that within unpulverized lumps, leading first to volume-stabilization (against wetting and drying) and increase in soaked strength, and later to remarkable increases in even unsoaked strength (for relatively high stabilizer contents). This cementation is diffuse." Stocker also concluded, "It (diffuse cementation) is the dominant mechanism whereby included lumps of unpulverized soil are made impotent with respect to differential volume change and finally by which the lumps are increased in mechanical strength."

Stocker's "diffuse cementation theory" (21) suggests that in lime reactive soils, soil-lime pozzolanic reaction products may form in regions of low calcium concentration remote from the lime source. The applicability of the "diffuse cementation theory" is dependent on the soil being lime reactive (soil will react with lime to form calcium silicate and calcium aluminate hydrates).

PROPERTY CHANGES RESULTING FROM DHL AND PIL

Based on previous discussion, it is apparent that substantial property changes can be obtained with conventional lime stabilization of fine-grained soils. However, in DHL and PIL lime treatment applications, substantial distribution or mixing of the lime in the soil mass may not be obtained. Rather, seams, shafts, and other erratically distributed and concentrated lime sources exist. It is highly probable that the soil immediately surrounding these concentrated sources will display many of the lime-soil reactions previously outlined. But the extent of soil immediately affected is quite small. As time progresses, migration/diffusion processes (and diffuse cementation) apparently increase the volume of soil affected by the lime.

In trench studies of a soil, one year after being lime injected, Lundy and Greenfield (19) observed that "A putty knife could not penetrate these treated areas above and below the (lime) seams." Strength tests were conducted on tube samples taken (about 1 year after injection) from the two lime injected soil deposits. Figures 8, 9, 10, and 11 summarize the test results. For the lime treated glacial deposit, substantial increases in both shear strength, Figure 8, and stress-strain behavior, Figure 9 were found. The lime-treated post glacial soil deposit displayed increases in shear strength, Figure 10, but no marked change in stress-strain behavior, Figure 11. Lundy and Greenfield (19) noted that the lime appeared to be much better distributed in the post-glacial deposit which contained varves than in the glacial deposit. They concluded "lime diffusion can occur in sufficient amounts to cause both flocculation and pozzolanic reactions in the soil-water system" and "a definite reaction and strength increase has occurred in these areas where lime has migrated" (19).

Lundy and Greenfield (19) also noted a slight reduction in plasticity index in the lime-treated soils, Figure 12. Davidson, et al., (16) found that the plastic limit was changed in the soil surrounding a concentrated lime source, Figure 13.

DISCUSSION OF SIGNIFICANCE OF SOIL PROPERTY CHANGES EFFECTED BY DHL AND PIL LIME TREATMENT

Assuming that substantial soil strength and other property changes occur in the vicinity of the lime source, a logical question that arises is, how much soil has to be strengthened before a substantial increase in mass strength is effected? Some feel that DHL and PIL procedures can not be depended upon for substantial strengthening of a soil mass. For example, Fohs and Kinter (18) concluded that the migration process for effecting translocation of lime in a soil system is very slow and only small amounts of lime can be translocated thereby "rendering this process impractical for effecting substantial soil mass strength increases."

The purpose of the following discussion is to examine various aspects of mass strengthening as related to DHL and PIL treatment.

Laboratory Study

In a recent laboratory study (28), the concept of mass strengthening by means of ZONAL TREATMENT was examined. Three types of fairly simple laboratory tests were conducted.

a. Layering

The effect upon specimen strength and stiffness resulting from a layer of lime treated soil sandwiched in a weaker soil material was examined. Figures 14 and 15 depict results. It is noted that a stiff layer does increase strength.

b. Columnar Inclusions

One inch diameter by 2.6 in. long stiff columnar inclusions were placed in a compacted soil specimen. As noted in Figure 16, only a slight strengthening was effected by these inclusions. A slight increase was also noted when the inclusion stiffness increased from E = 30 ksi to E = 400 ksi.

c. Subgrade Scale Model

A subgrade scale model with stiff columnar inclusions was prepared. A plate bearing test was conducted to determine the effect of inclusions on the modulus of deformation as calculated from the following Boussinesq deflection equation:

$$E = \frac{1.18pa}{\Delta} \qquad (3)$$

where: E = modulus of deformation, psi

p = plate pressure, psi

a = plate radius, inches

Δ = plate deflection, inches

Figure 17 shows typical results. It can be noted that a marked increase in E is obtained with the inclusions present.

Composite System

A brief examination of composite system theory indicates that a substantial volume of stiff inclusions must be present if the stiffness of the composite mass is to be significantly affected. In Figure 18, it is noted that about 50 percent inclusions volume concentration is required if a doubling of the modulus of elasticity of the mass is desired (E of inclusions = 100 E matrix).

Analytical Behavior Studies

The effect of thickness and number of incremental stabilized zones (stabilized layer E = 50 ksi) on the calculated (elastic layer theory) surface deflection of a flexible pavement is depicted in Figure 19. It is shown that as the number and thickness of the stabilized layers increase, the calculated surface deflection is markedly reduced.

Another analytical study (26) considered the effect of stiff columnar "post hole piles" placed in the subgrade. A finite element procedure was used to evaluate the response of the pavement system shown Figure 20. The primary variable examined was pile stiffness (E varied from 50 ksi to 500 ksi). Other important dimensions were: dia. = 12 in., length = 7 ft., and spacing = 36 in. The results from the study of the flexible pavement pile system indicate the following:

1. For a pile stiffness of 50,000 psi, calculated surface deflection was not significantly reduced.

2. For a pile stiffness of 500,000 psi, calculated surface deflection was reduced about 15 percent.

Other Studies

A number of other studies have considered the strengthening potential of PIL and DHL procedures. Robnett (22) discussed the general concept of strenghhening airfield runways by means of strengthening the in-place subgrade. The Alabama Highway Department (23) reported reduced Benkelman Beam deflection values for a pavement one year after treatment with DHL.

Thompson and Robnett (24) have reported on a field evaluation of various techniques (including PIL and DHL) for strengthening an existing airfield pavement. Dynamic surface deflection and static subgrade deflection data were primarily used for structural response evaluation. For both the DHL and PIL sections, over the period of evaluation (June, 1973 to October, 1974) a significant change in structural response was noted, but no sustained strengthening effect was detected. Complete details of the PIL portion of the study are reported in another paper being presented at this conference (25).

SUMMARY AND CONCLUSIONS

The alteration of soil properties as effected by various lime treatment procedures has been reviewed. Of primary interest to the problem of roadbed stabilization is that of strength improvement.

When lime is intimately mixed with a fine-grained soil, substantial and beneficial changes in soil properties occur. Flocculation/agglomeration reactions effect strength improvements with all fine-grained soils. With some soils (termed lime reactive) pozzolanic reactions produce additional strength increases.

With DHL and PIL lime treatment procedures intimate blending of soil and lime is not achieved. However, as curing takes place, numerous studies have documented the fact that property changes are effected in the soil surrouding the concentrated lime source; migration/diffusion processes are apparently responsible for lime movement. The rate of diffusion/migration is very slow.

Various analytical, laboratory and field studies indicate that localized or zonal strength improvements such as might be effected by DHL and PIL may not cause substantial changes in "mass" strength.

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Table 1. CBR Values for Selected Soils and Lime-Soil Mixtures (From Ref. 31).

Soil	Natural Soil		Lime-Soil Mixtures				
				No Curing[a]		48 Hrs. Curing @ 120F	
	CBR,%	Swell,%	% Lime	CBR,%	Swell,%	CBR,%	Swell,%
Good Reacting Soils							
Accretion Gley 2	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	3.1	1.4	5	88.1	0.0	370.0	0.1
Bryce B	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. Till	6.8	0.2	3	10.4	0.5	85.0	0.1
Cisne B	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	7.2	1.4	3	--	---	98.5	0.0
Cowden B	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis Clay	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C	1.3	0.0	5	32.4	0.0	295.0	0.1
Illinoian B	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian Till	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian Till	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	1.8	4.2	3	15.9	0.2	127.0	0.0
Non Reactive Soils							
Fayette B	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	2.9	0.8	3	12.7	0.0	14.5	0.0
Tama B	2.6	2.0	3	4.5	0.2	9.9	0.1

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

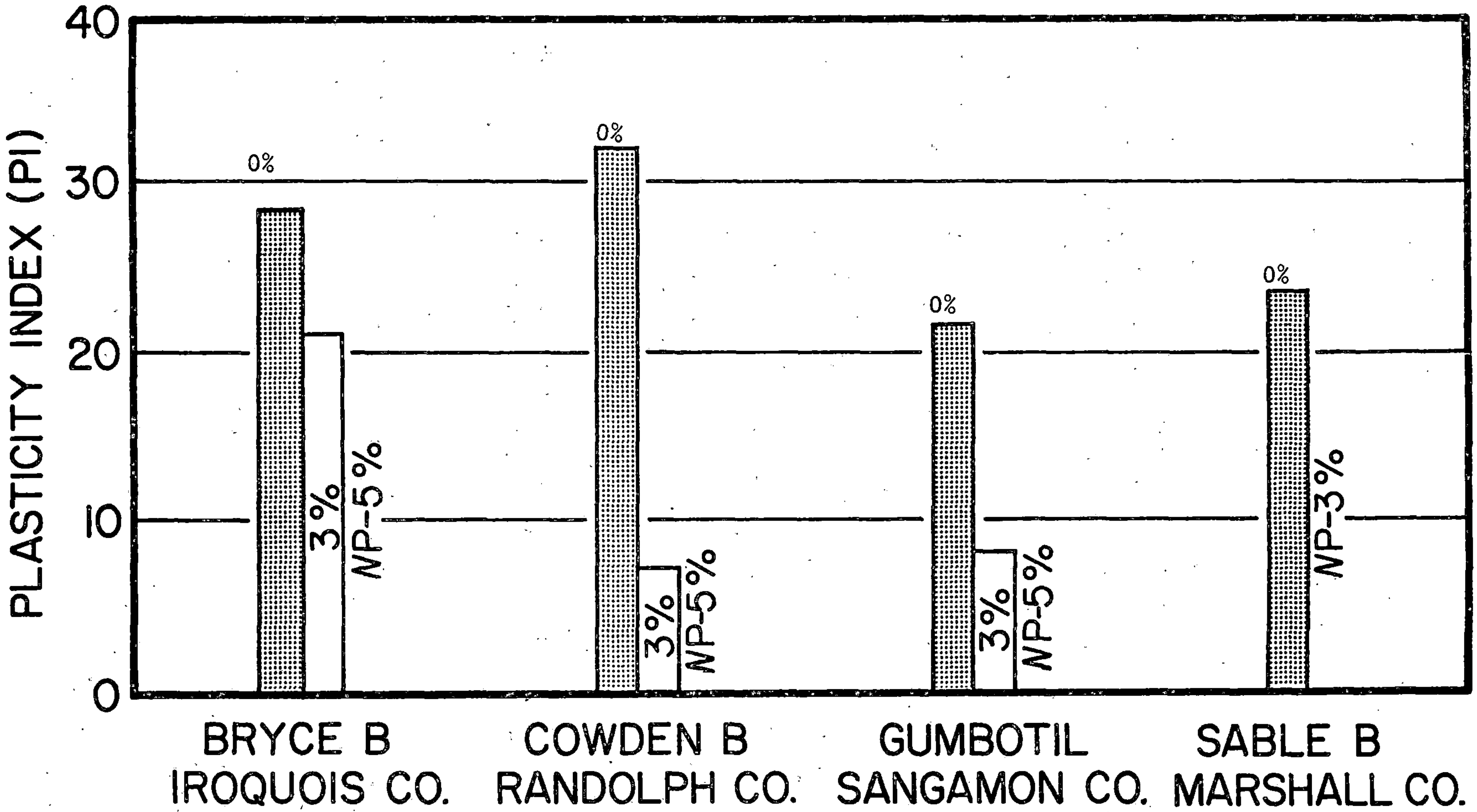


Figure 1. Typical Effect of Various Percentages of Lime on Plasticity Index

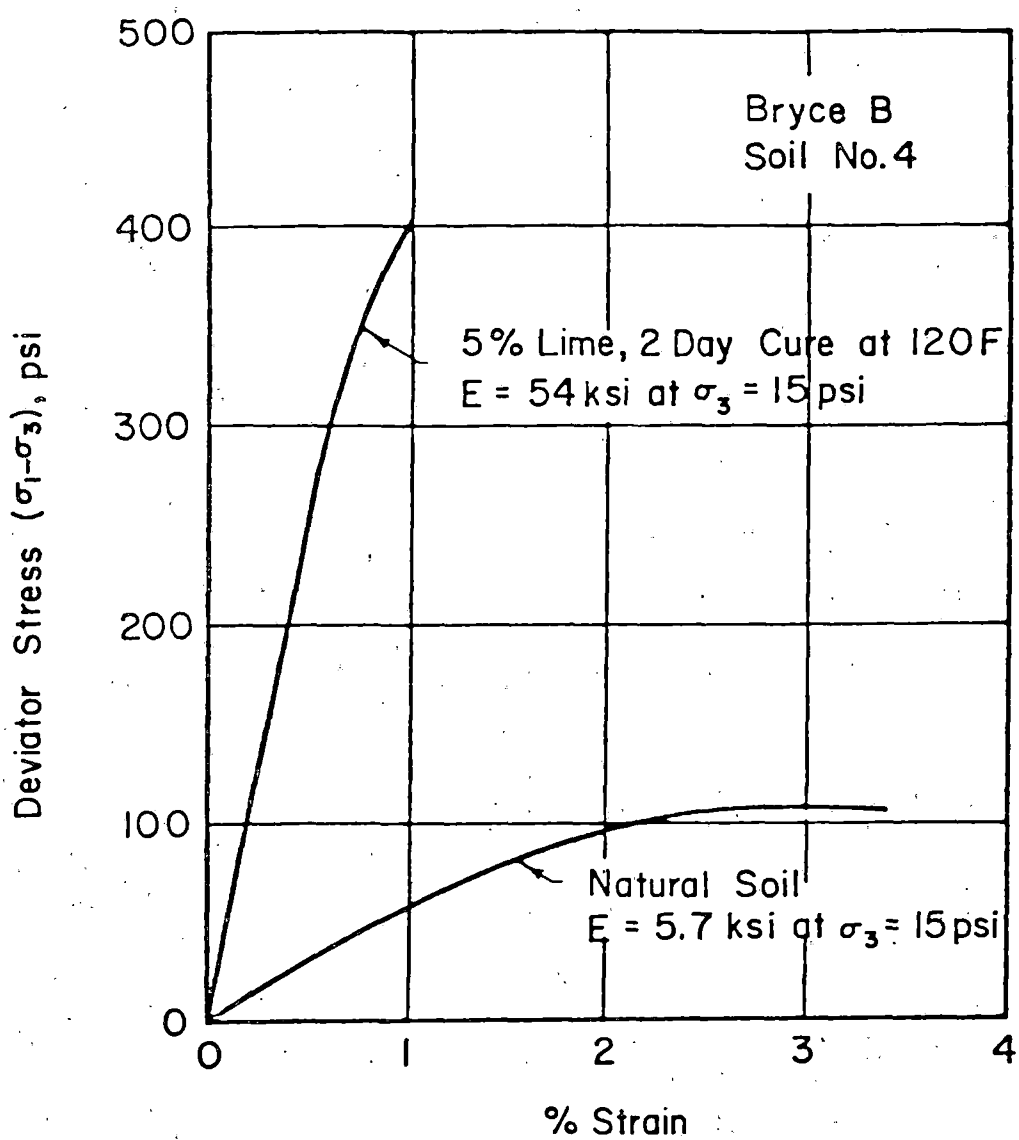


Figure 2. Typical Stress-Strain Curves for Natural and Lime-Treated Soil

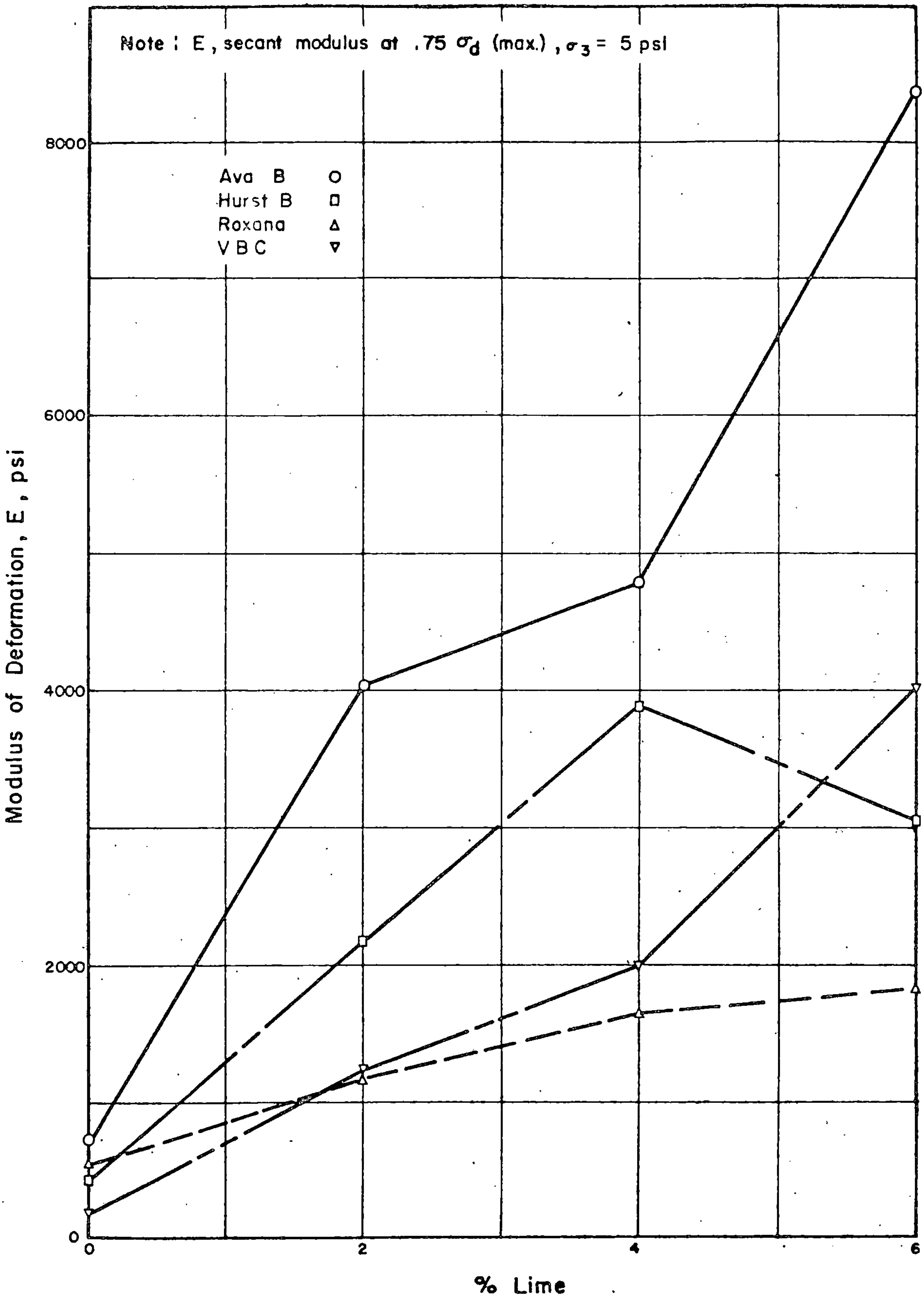


Figure 3. "Immediate Effects" of Lime Treatment on Static Modulus of Deformation (From Ref. 8).

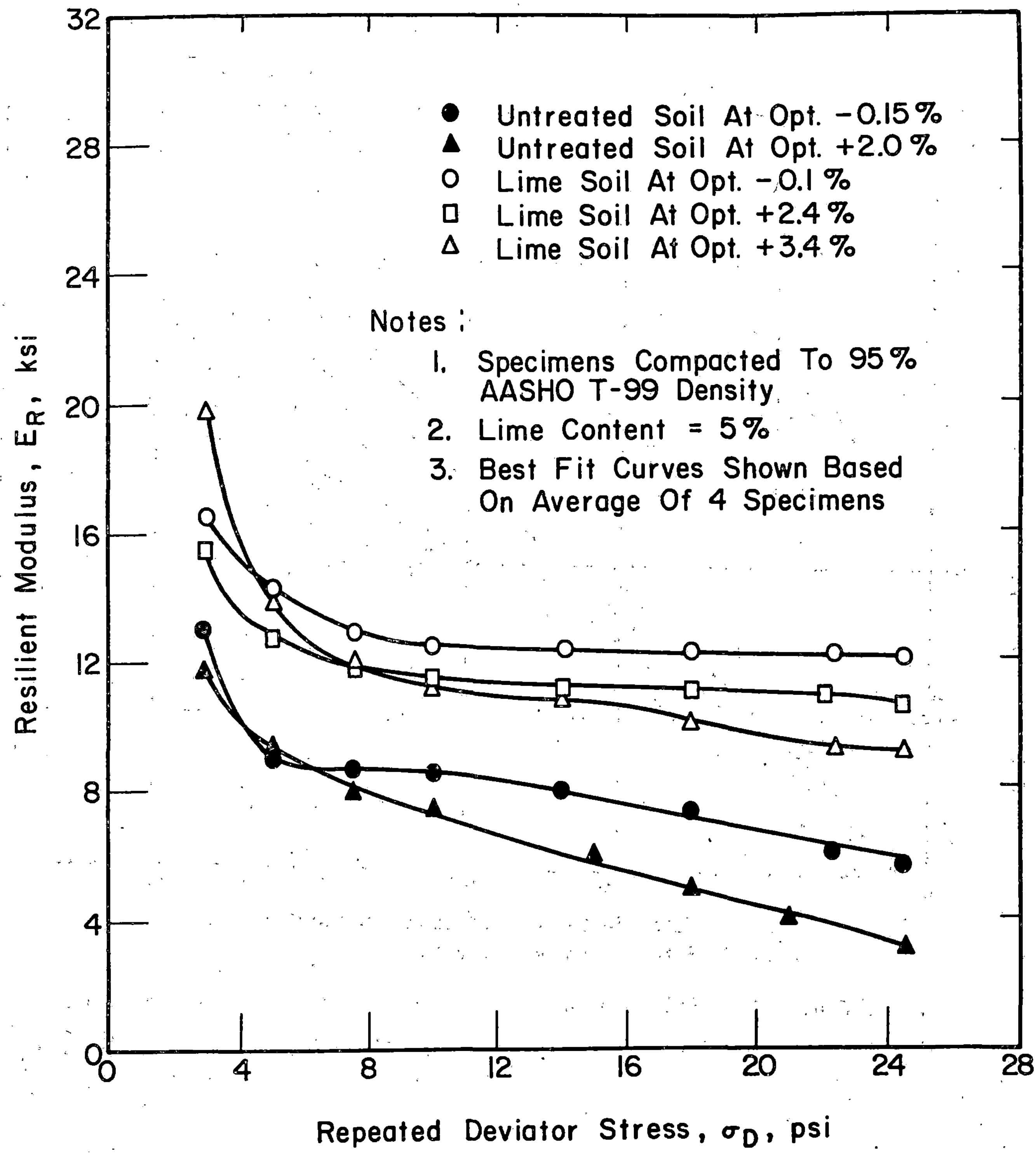


Figure 4. Typical Effect of Lime Treatment and Variable Compaction Moisture on Resilient Response of a Fine-Grained Soil (From Ref. 10).

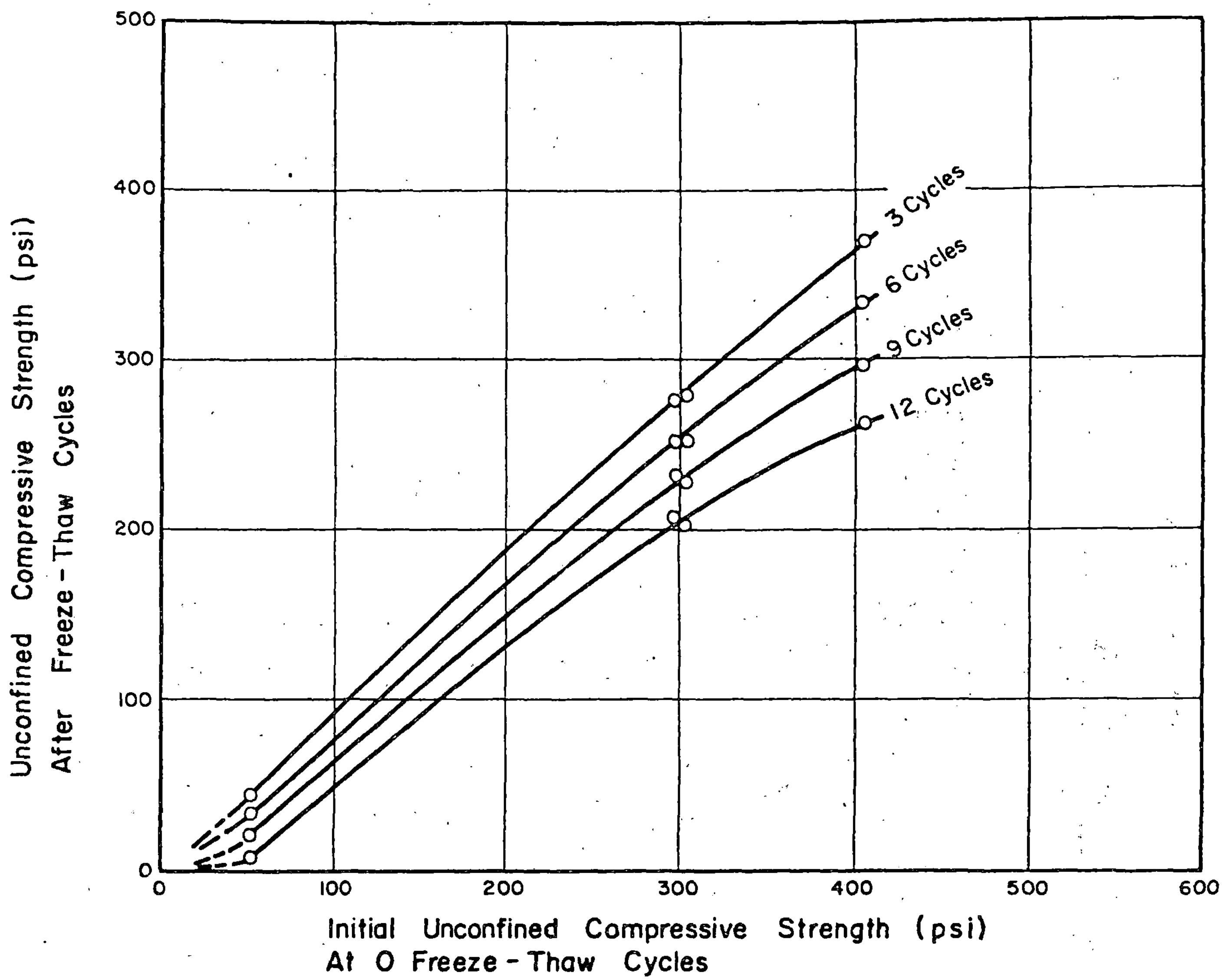


Figure 5. Influence of Initial Unconfined Compressive Strength on the Residual Strength after Freeze-Thaw Cycles (48 Hour Curing) (From Ref. 27).

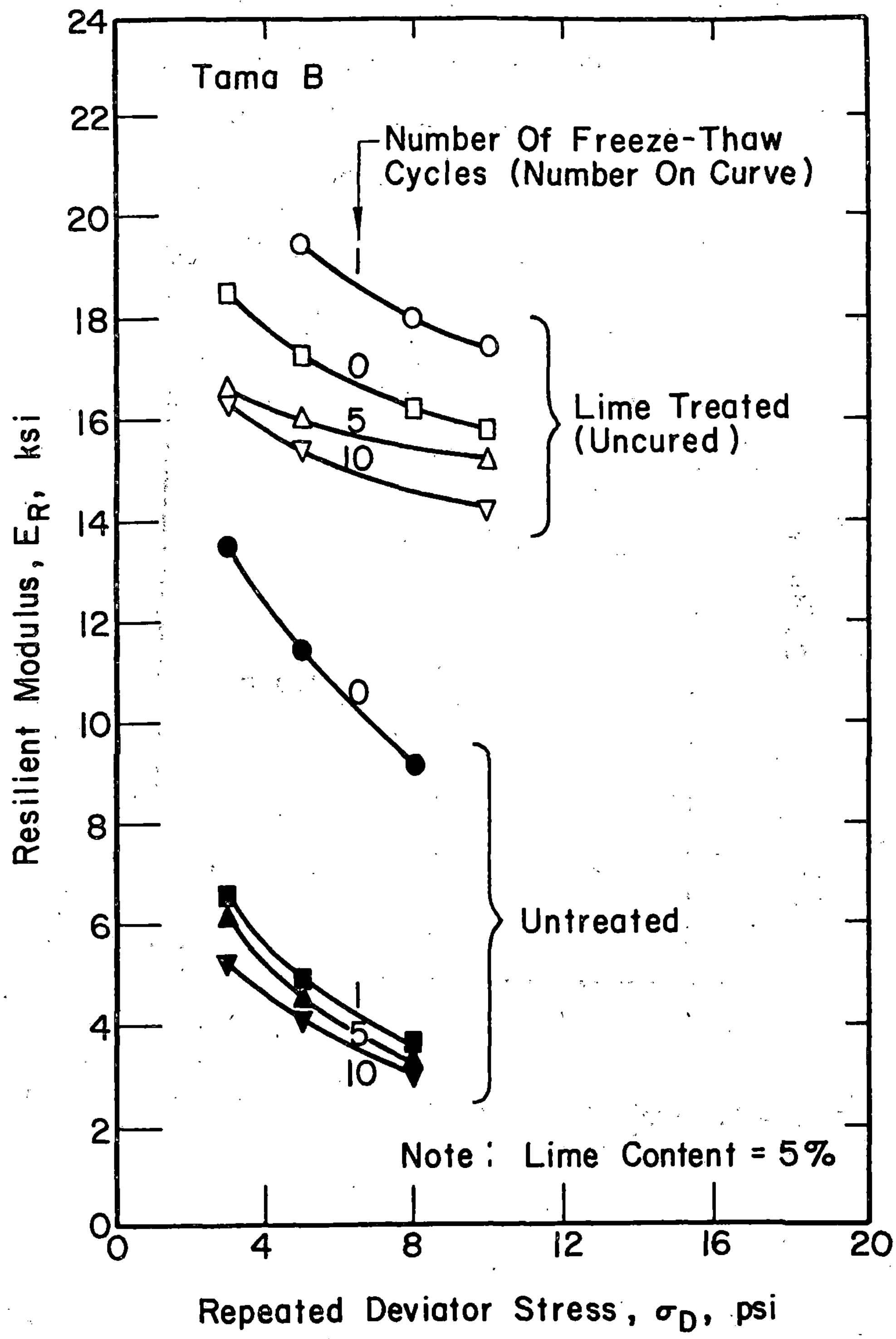


Figure 6. Effect of Lime Treatment and Freeze-Thaw on Resilient Response of Tama B Soil (From Ref. 10).

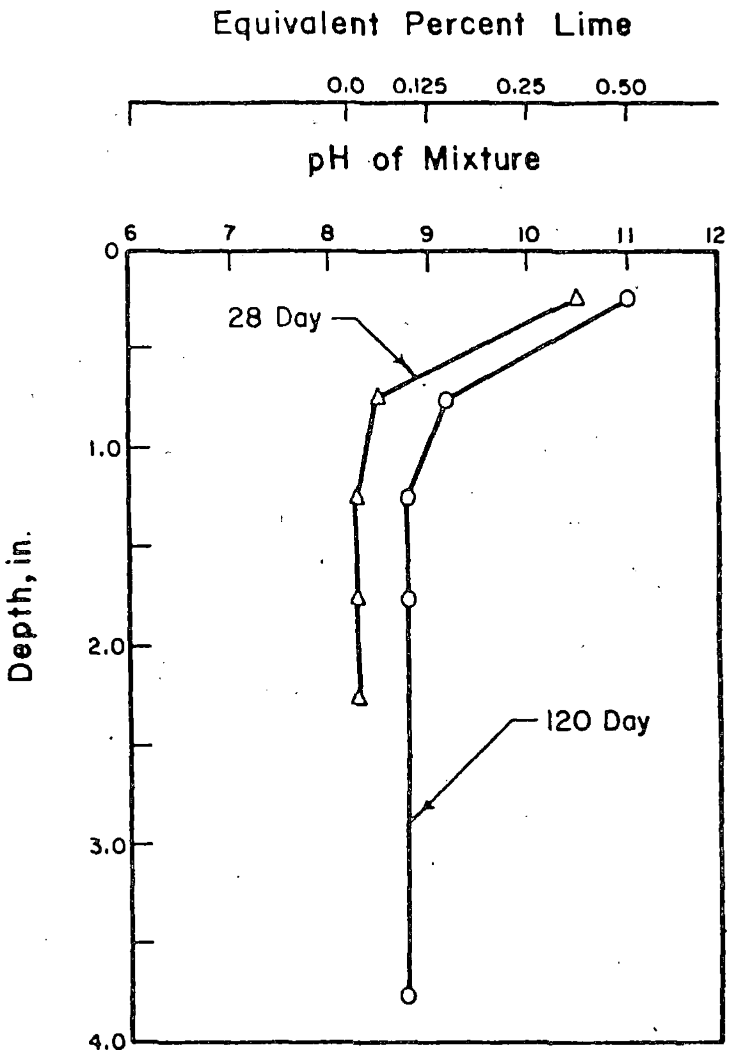


Figure 7. Typical Lime Migration Data for Altus Subgrade Soil (A-6(13)).

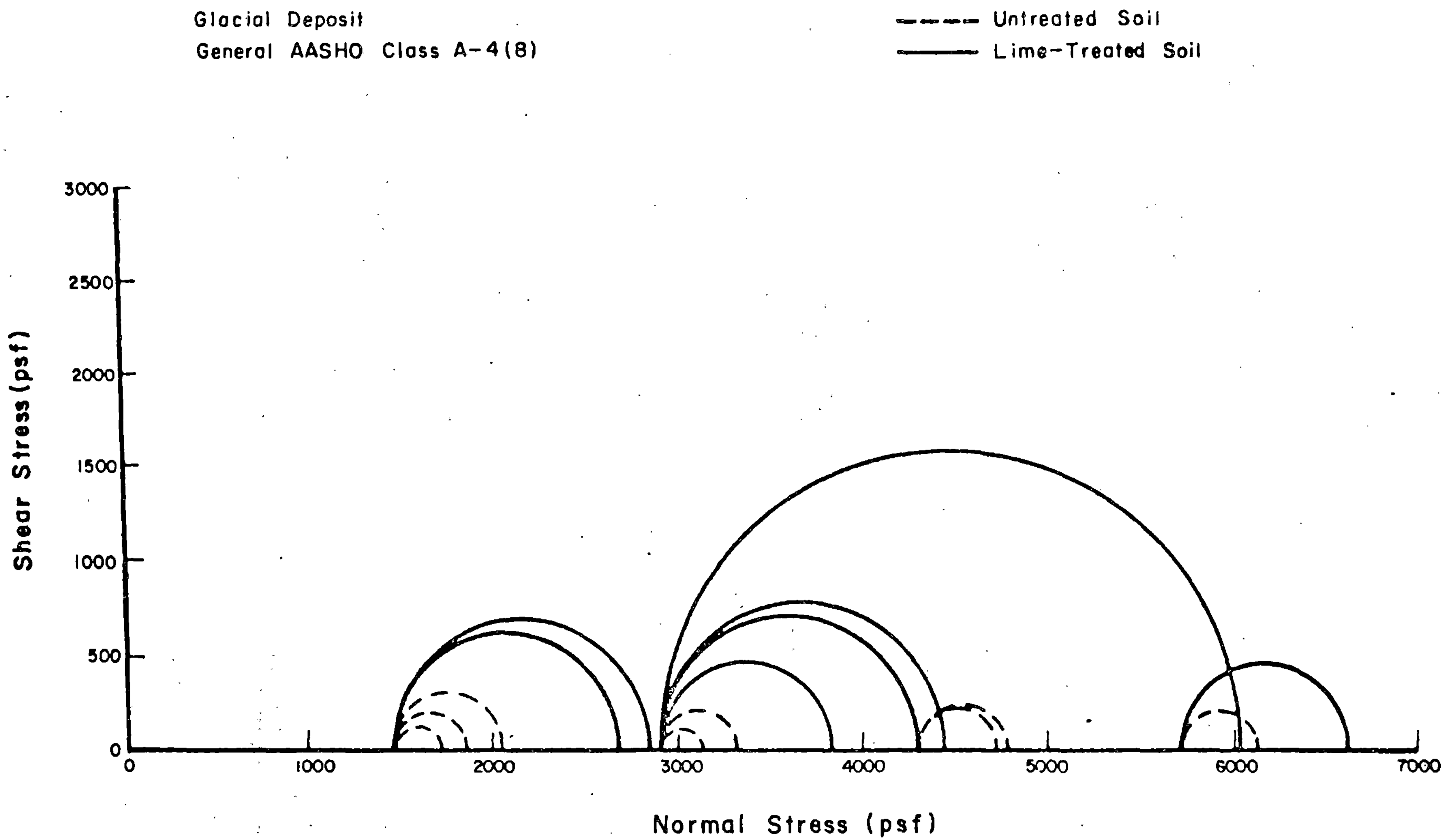


Figure 8. Plot of Shear Stress vs Normal Stress for Tube Samples of Untreated and Lime-Treated Soil (From Ref. 19).

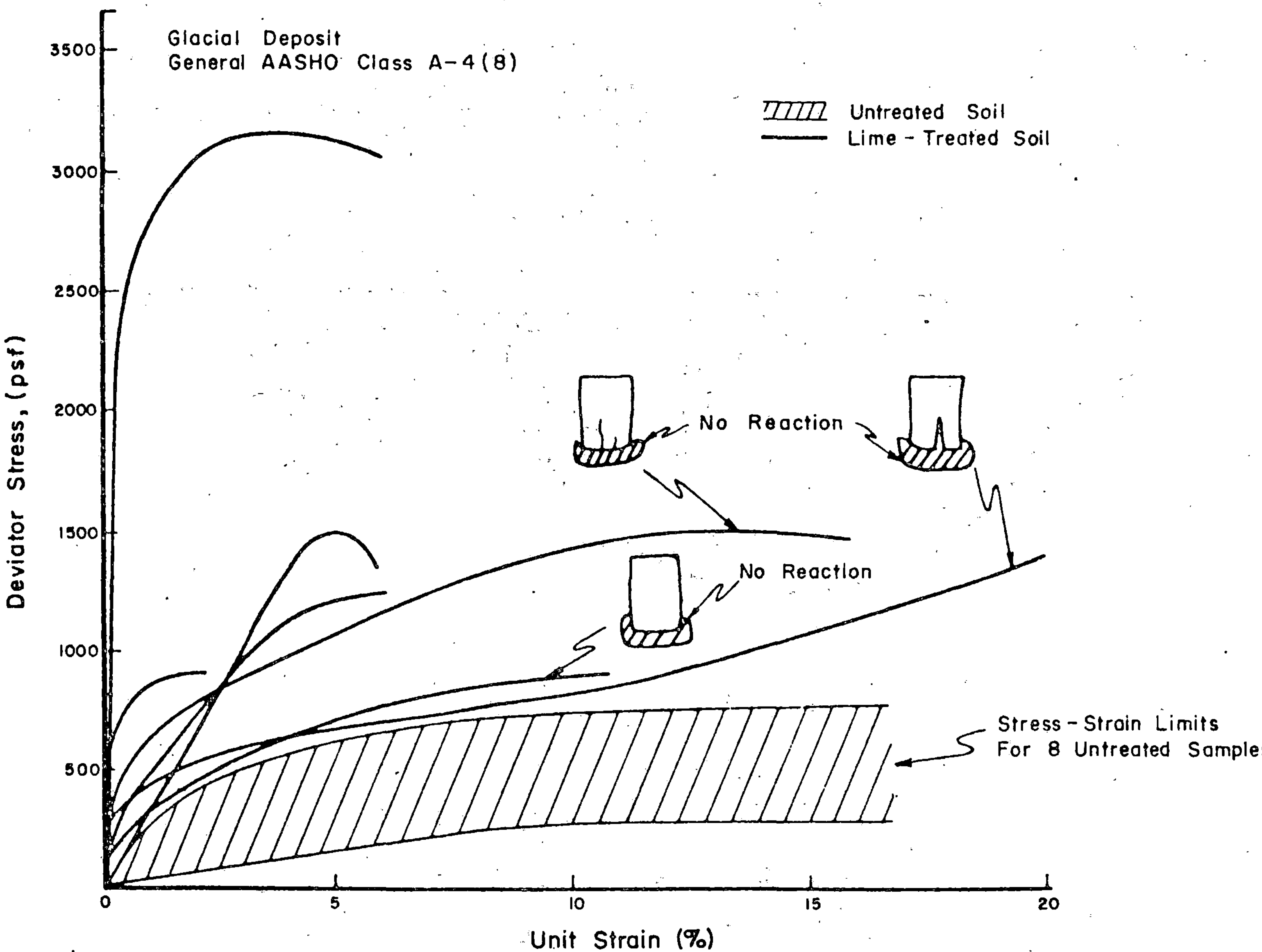


Figure 9. Stress Strain Plot for Tube Samples of Untreated and Treated Glacial Soil (From Ref. 19).

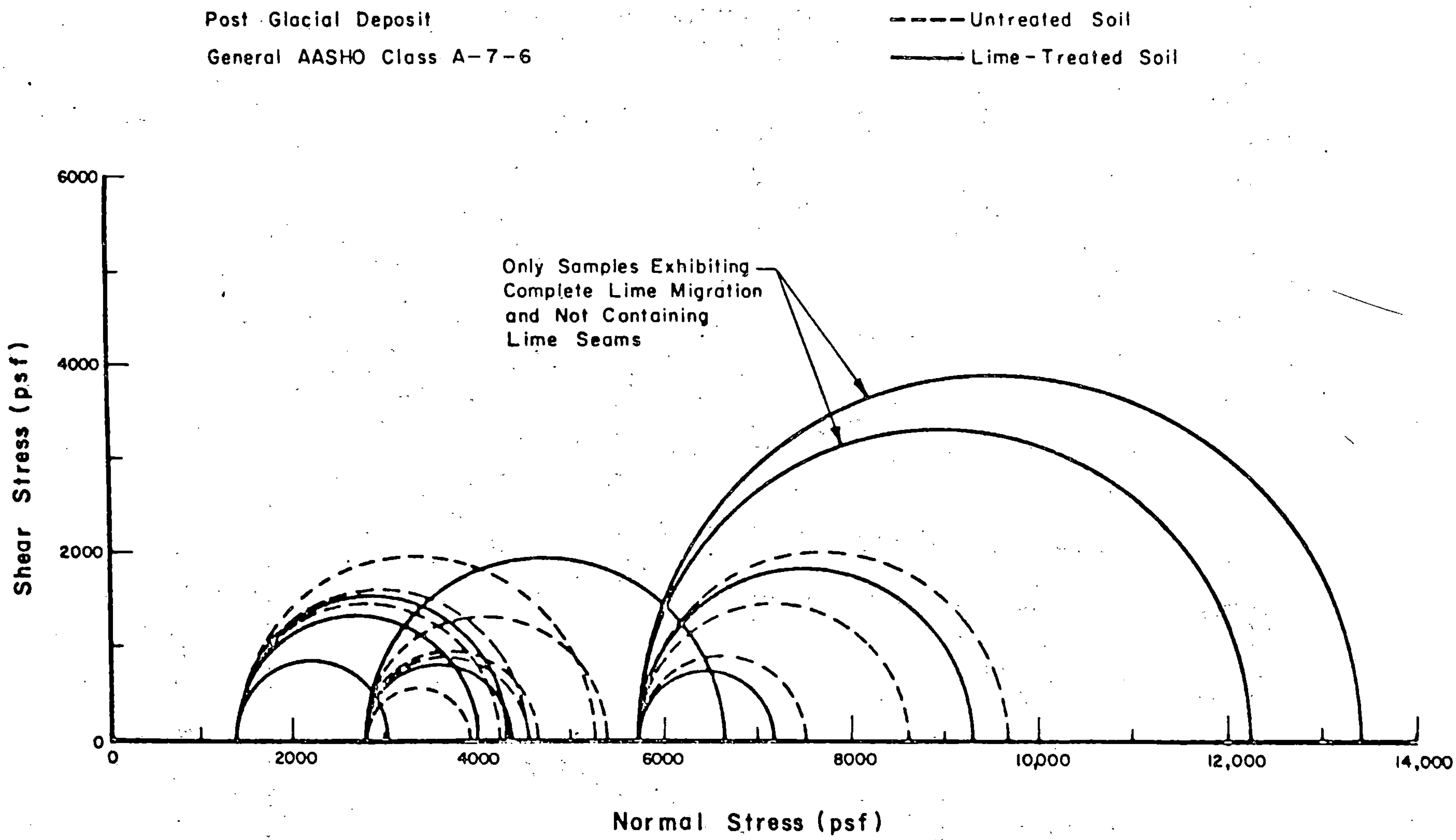


Figure 10. Plot of Shear Stress vs Normal Stress for Tube Samples of Post Glacial Soil Exhibiting Complete Lime Migration (From Ref. 19).

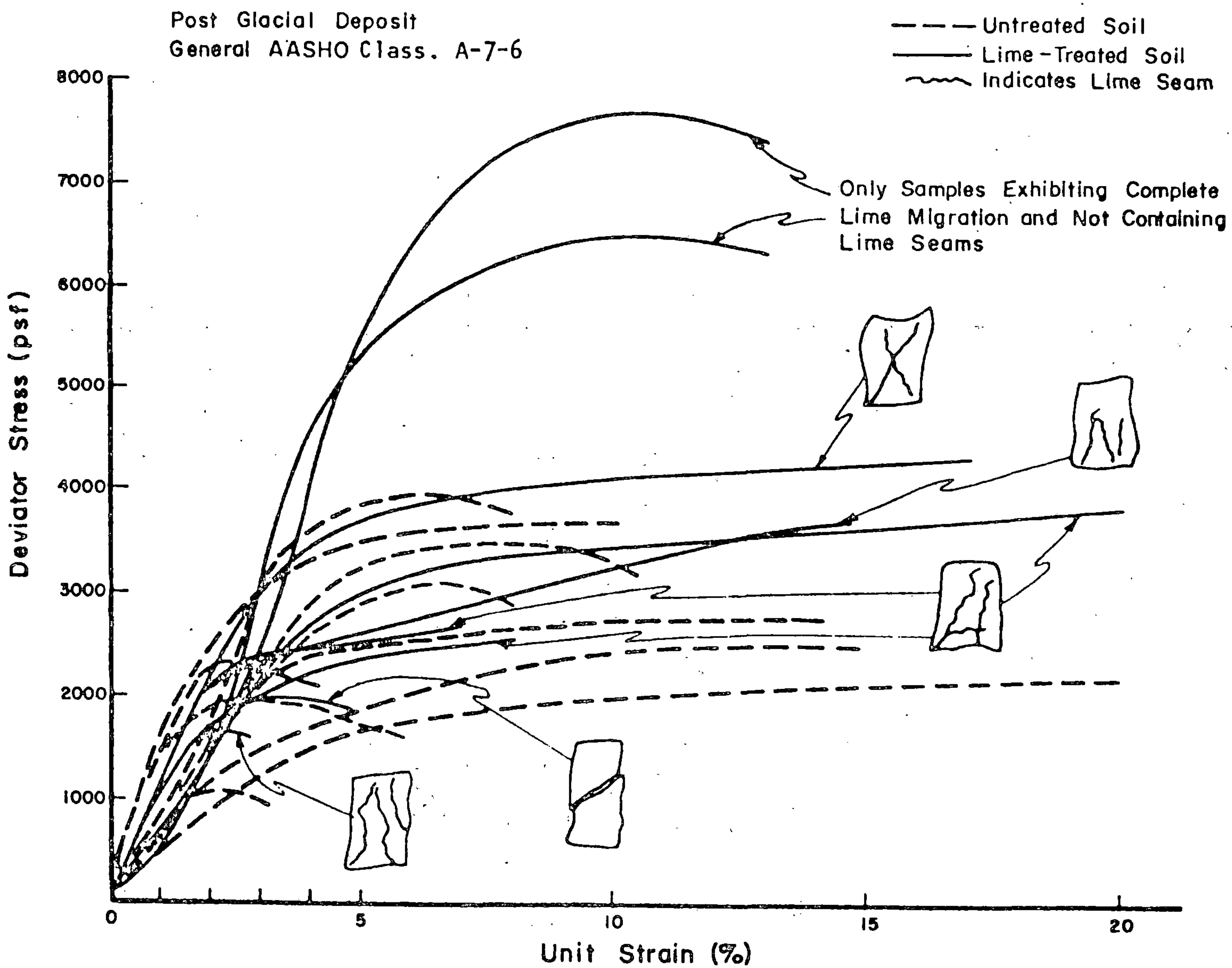


Figure 11. Stress-Strain Plots for Tube Samples of Untreated and Treated Post Glacial Soil (From Ref. 19).

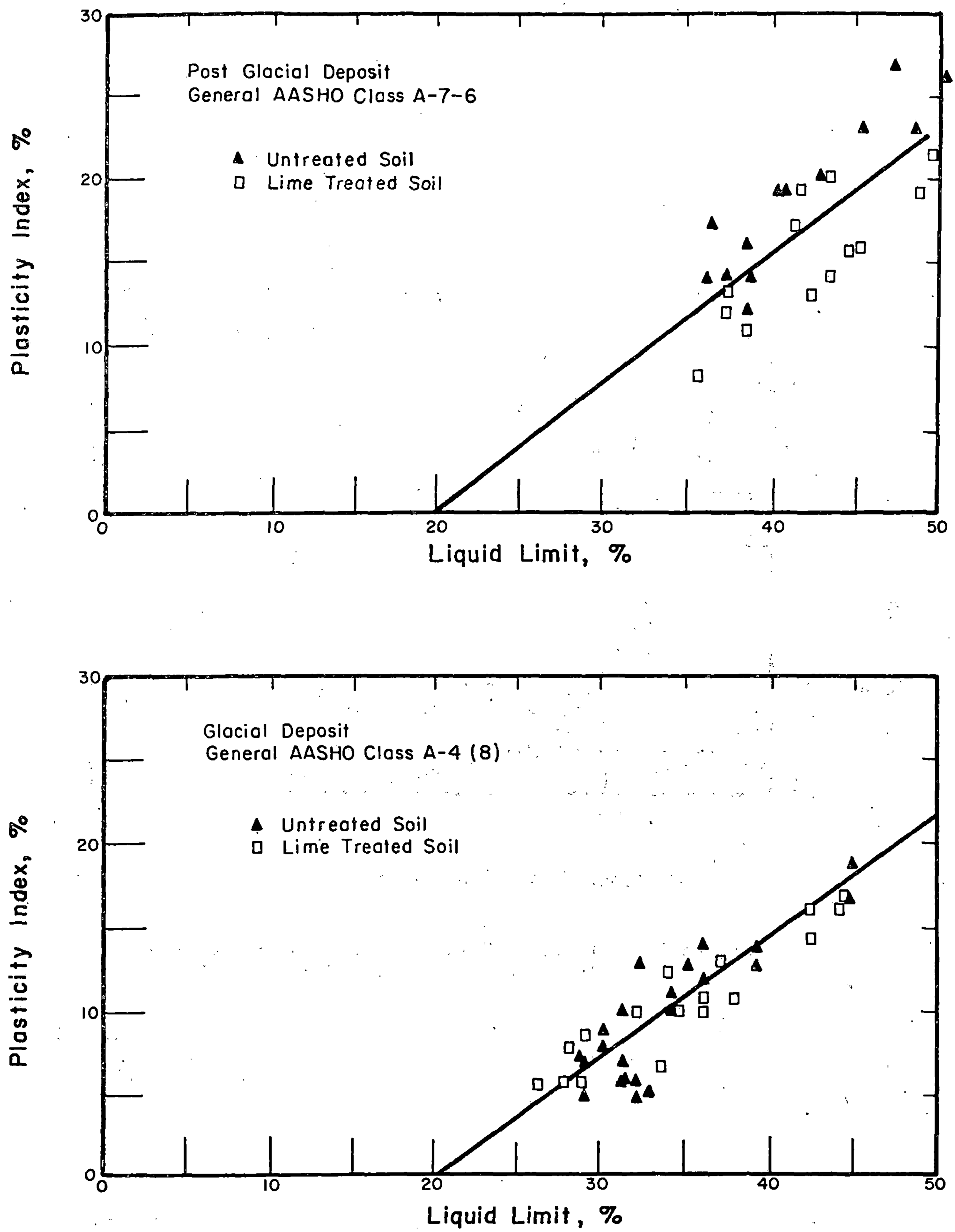


Figure 12. Effect of PIL Treatment on Plasticity Index (From Ref. 19).

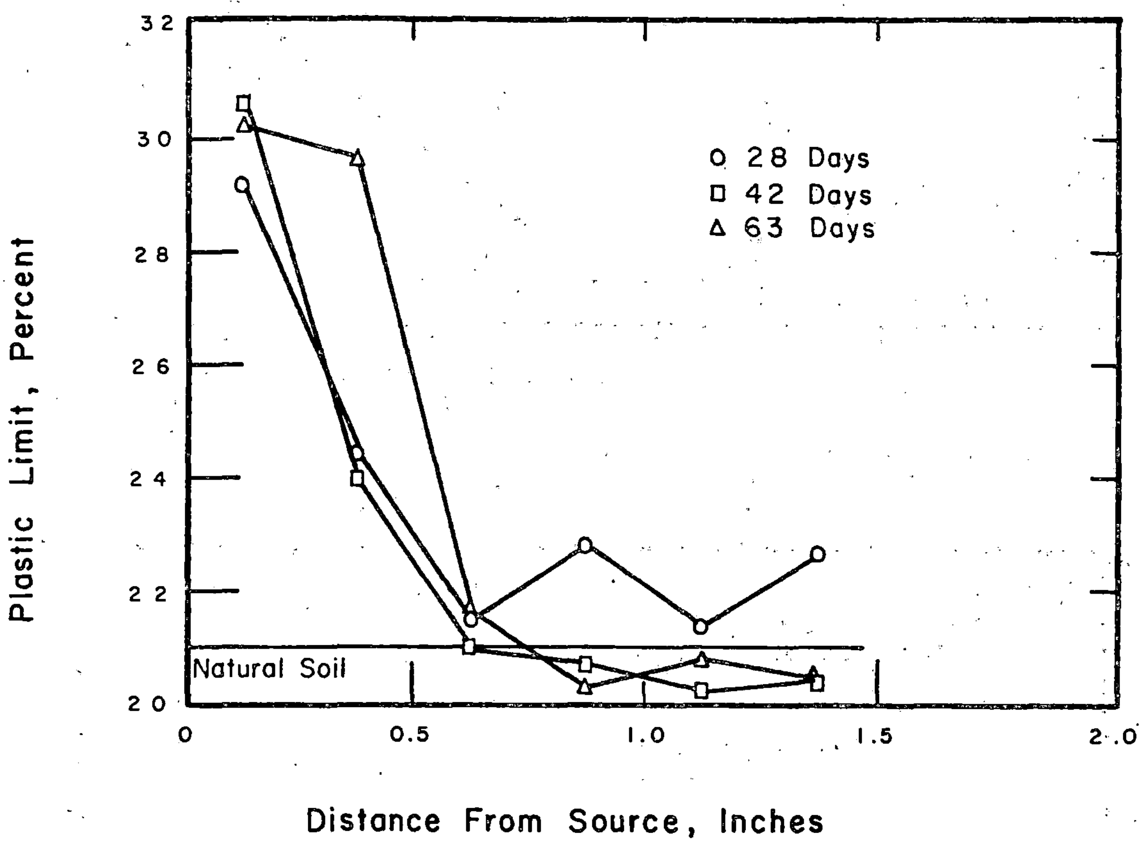
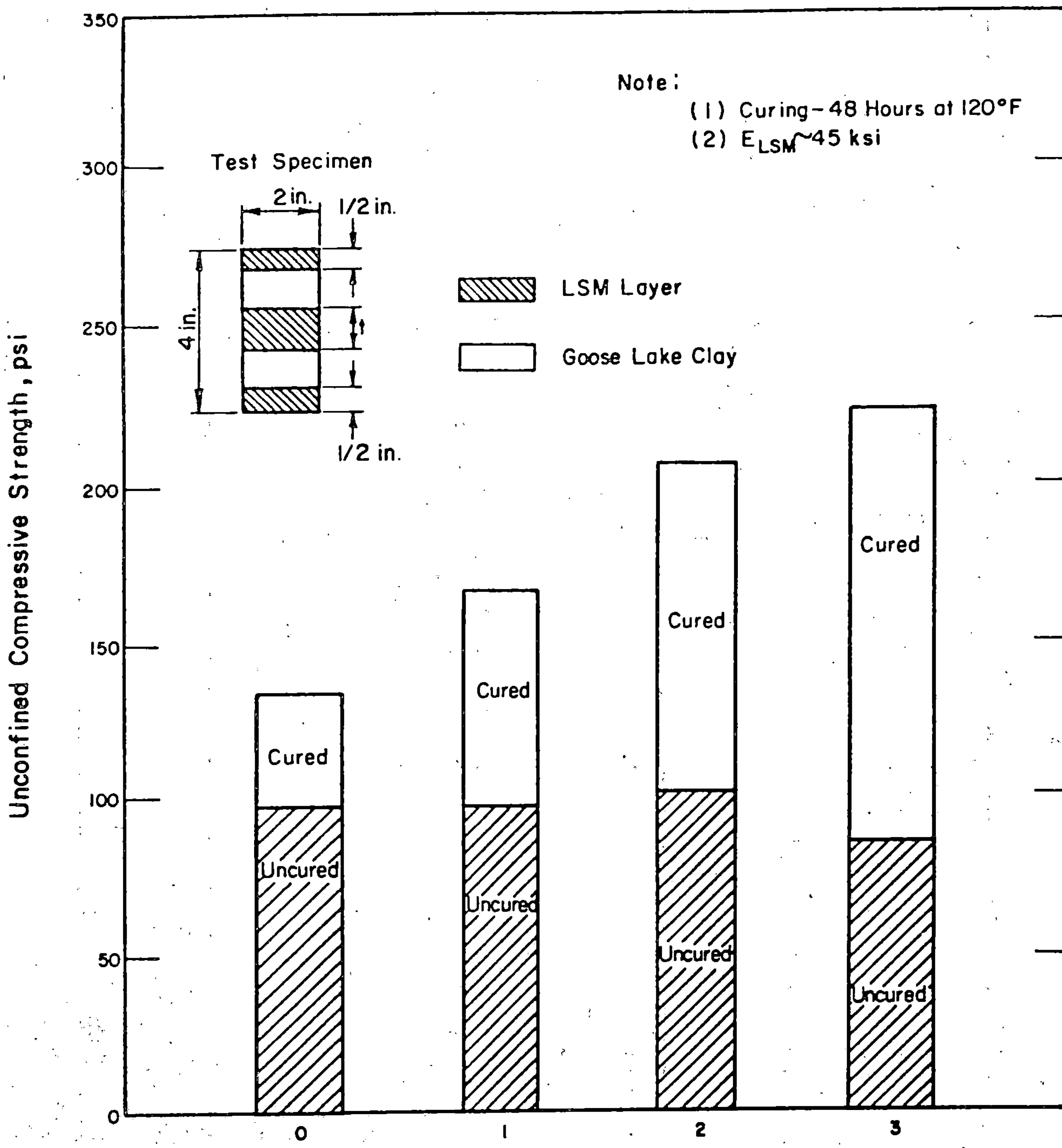
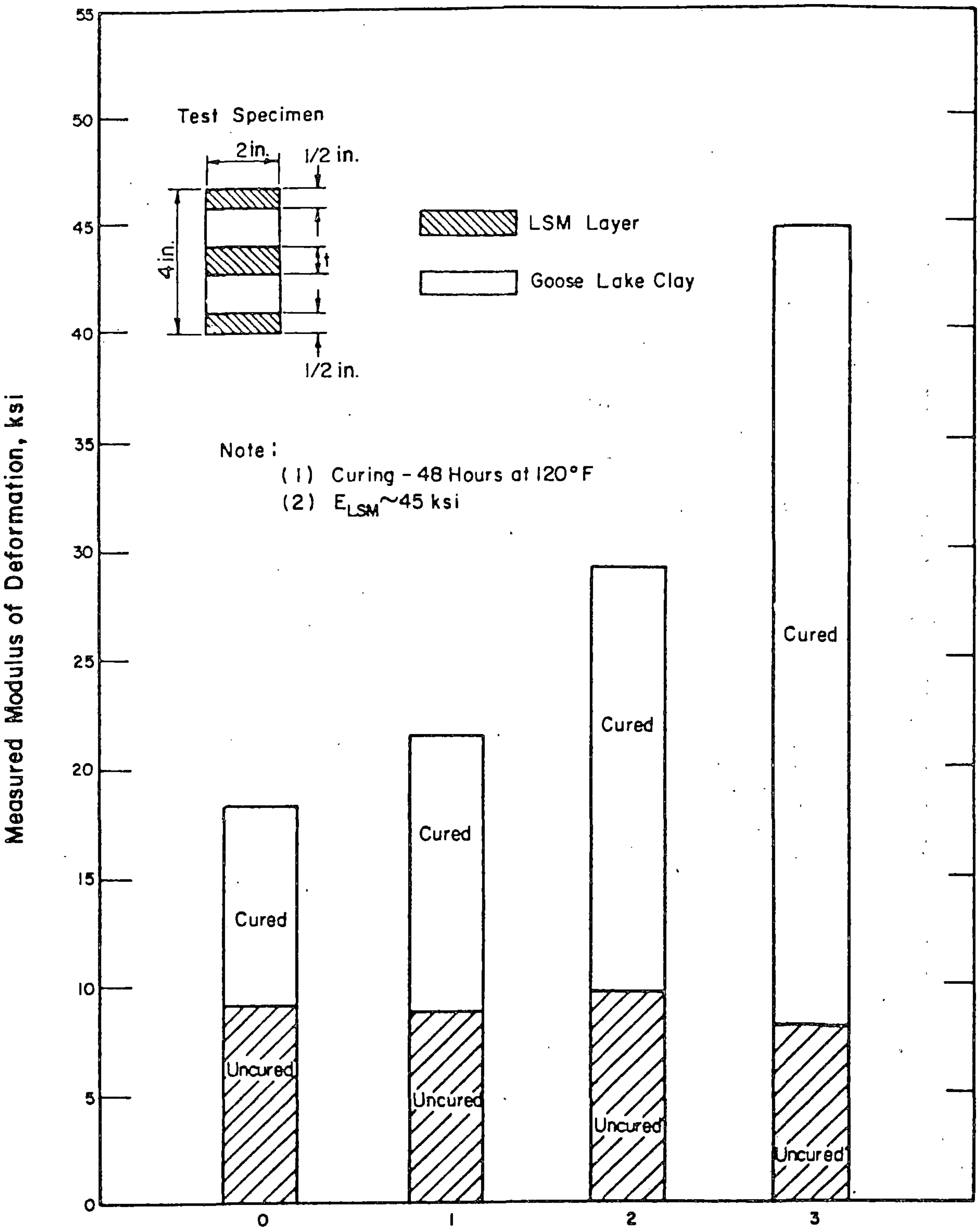


Figure 13. Effect of Distance from Lime Source and Time on Plastic Limit (From Ref. 17)..



Thickness of Lime-Soil Layer, t, inches

Figure 14. Effect of a Lime Soil Layer on Strength of Layered Composite Specimens (From Ref. 28).



Thickness of Lime-Soil Layer, t, inches

Figure 15. Effect of Lime-Soil Layer on Static Modulus of Deformation for Layered Composite Specimens (From Ref. 28).

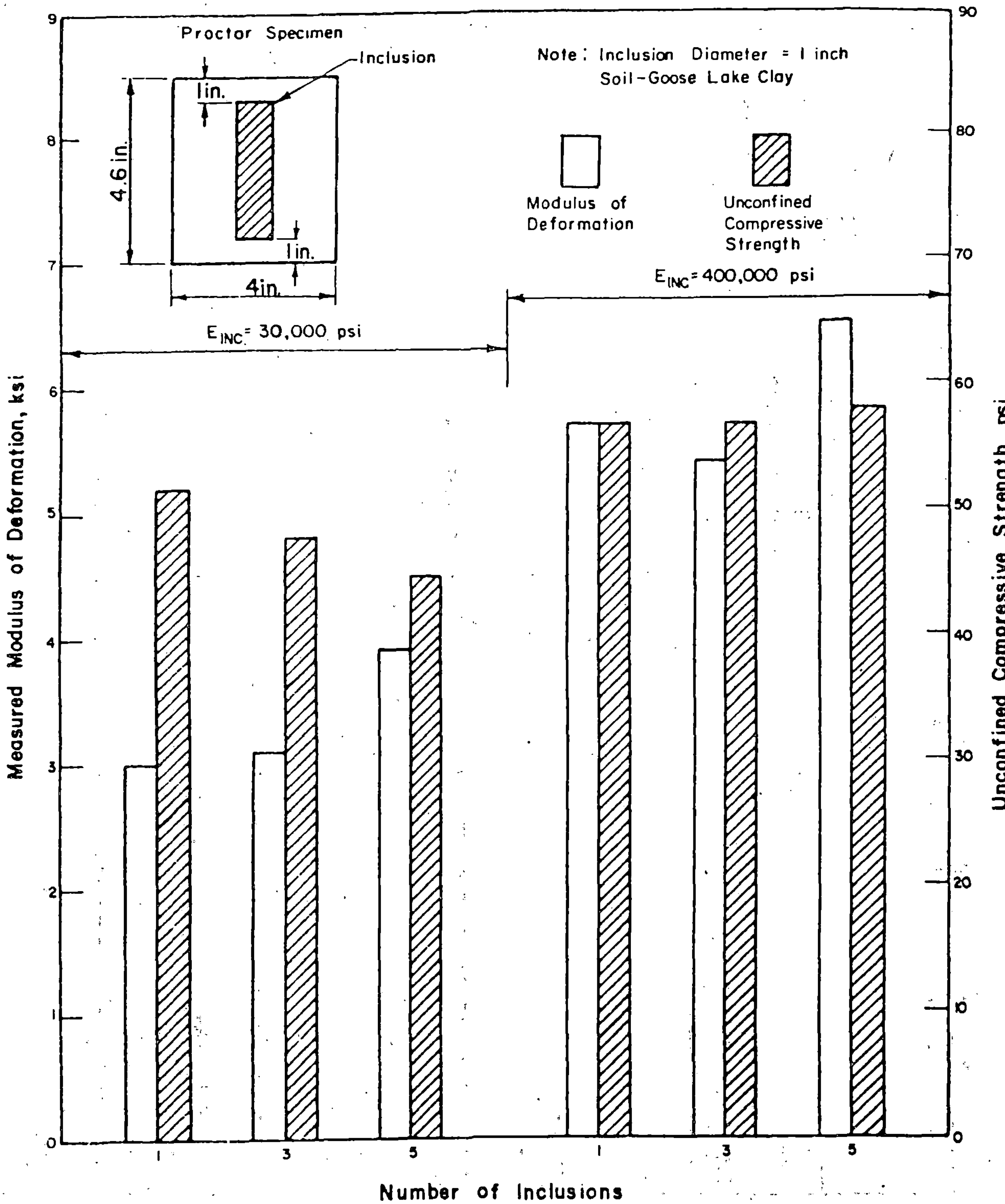


Figure 16. Effect of Columnar Inclusions on Strength of Cylindrical Composite Specimens (From Ref. 28).

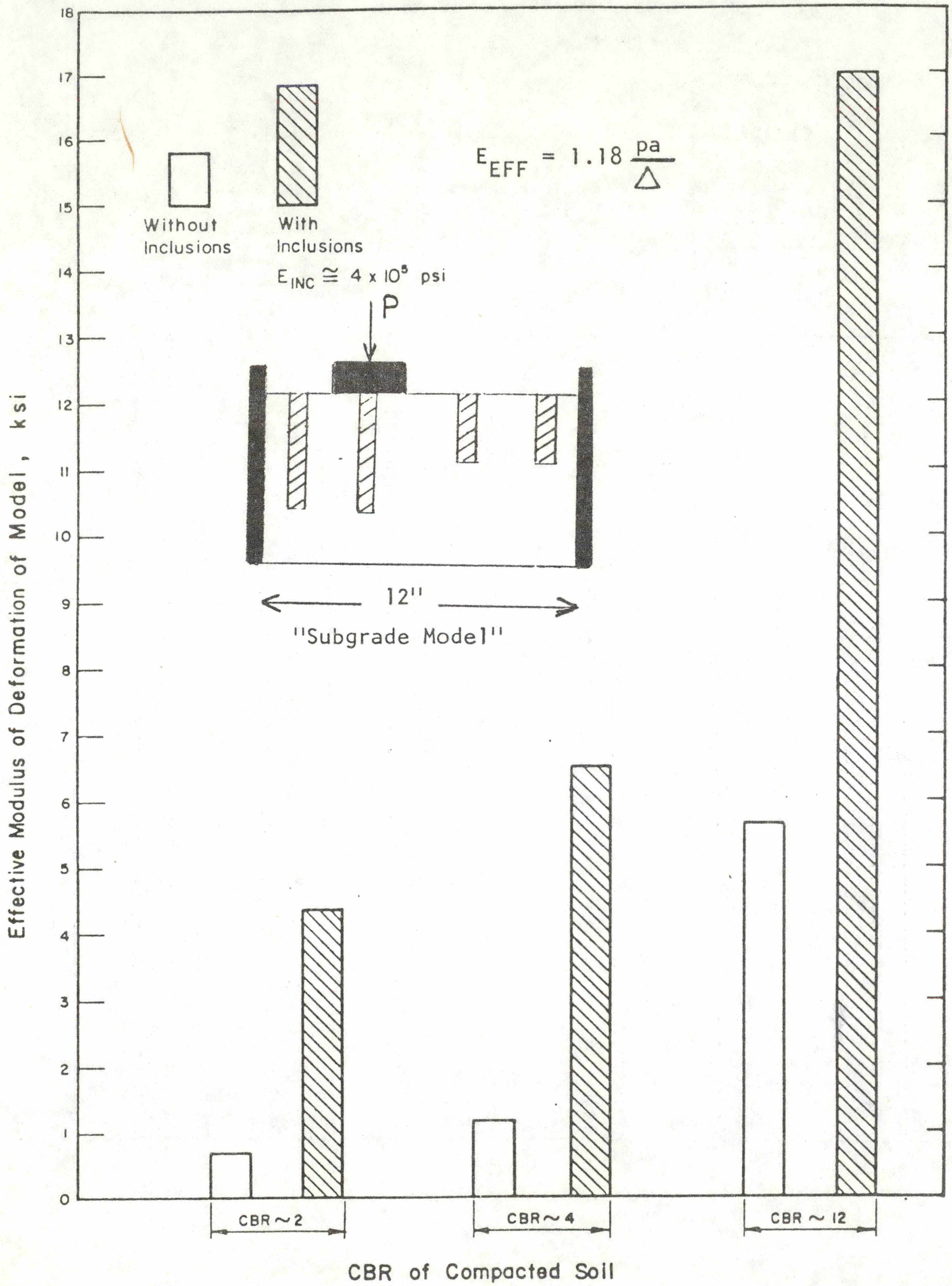


Figure 17. Effect of Subgrade Inclusions on Modulus of Deformation (From Ref. 28).

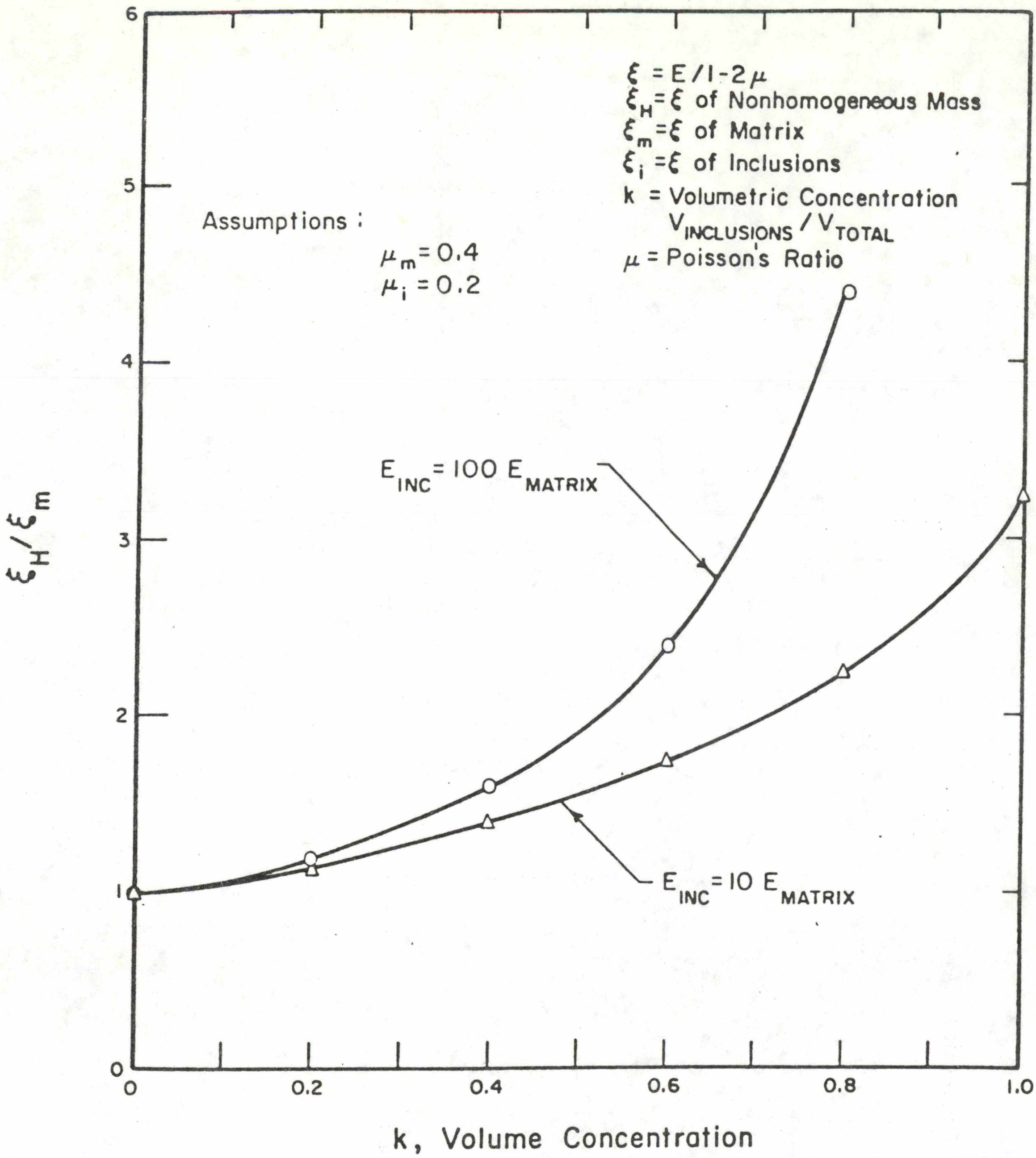


Figure 18. Influence of Volume Concentration and Modulus of Elasticity of Inclusion Behavior of a Heterogeneous Mass (From Ref. 29).

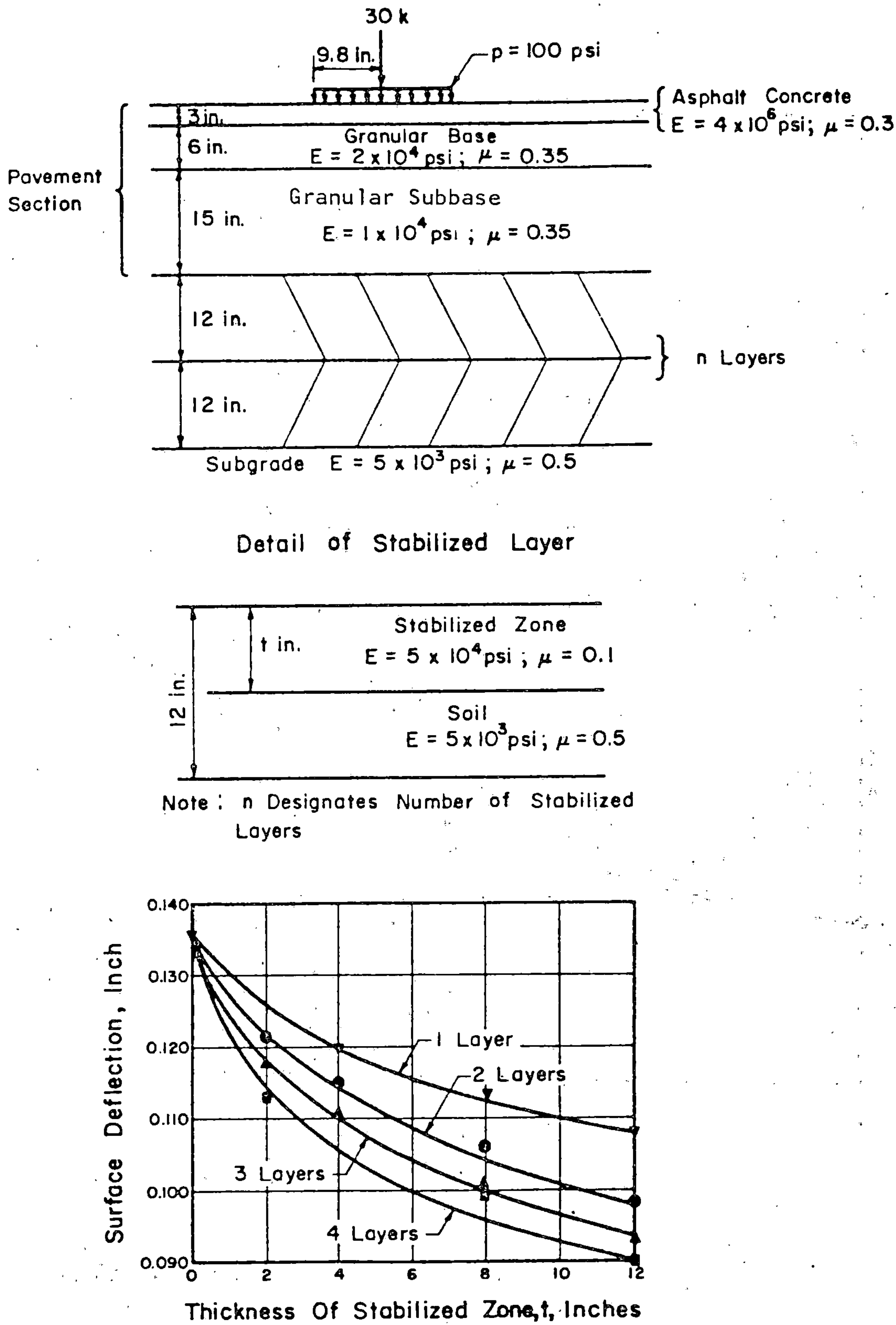
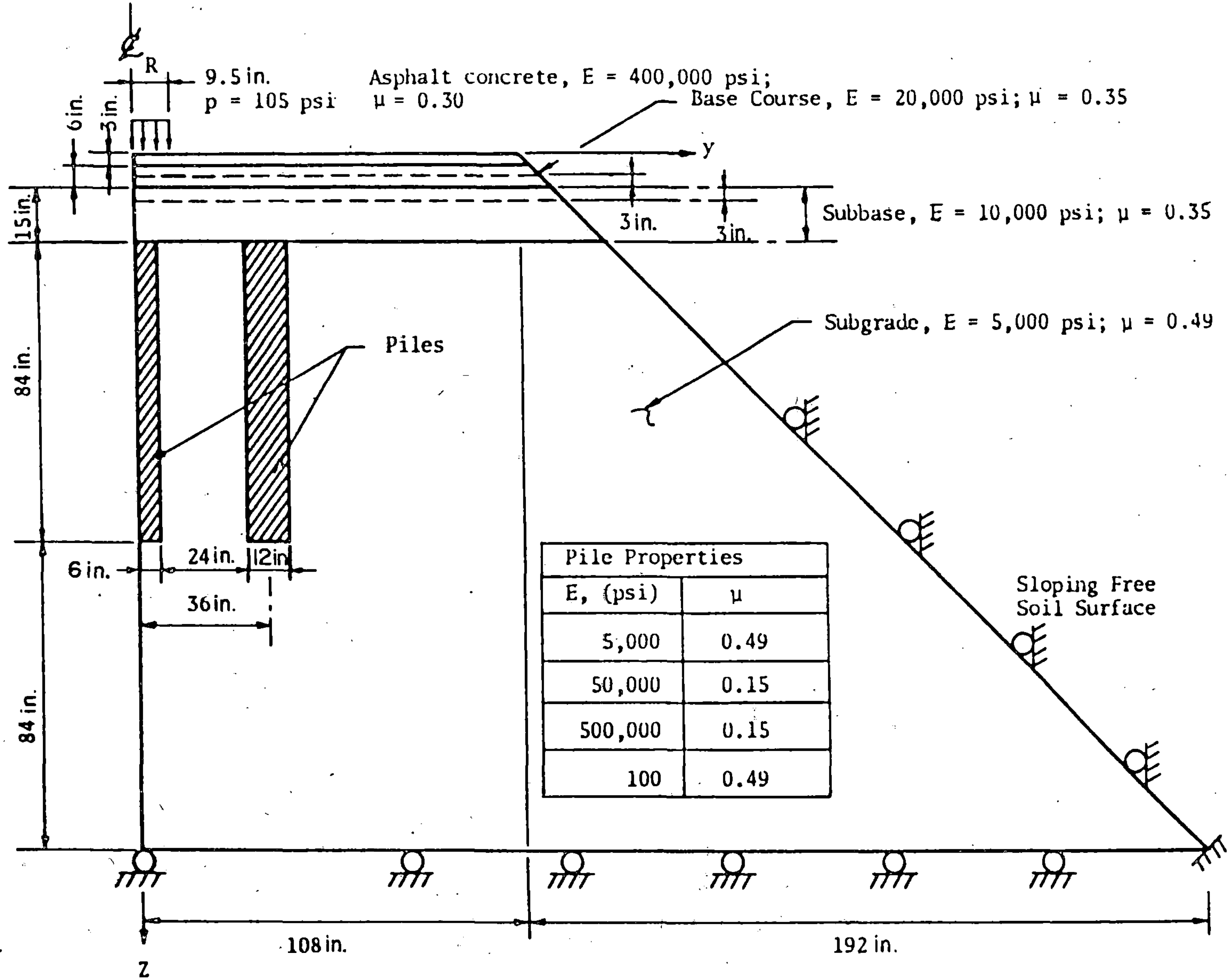


Figure 19. Effect of Thickness and Number of Incremental Stabilized Zones on Calculated Surface Deflection of a Flexible Pavement (Elastic Layer Theory) (From Ref. 30).



Pile Properties	
E, (psi)	μ
5,000	0.49
50,000	0.15
500,000	0.15
100	0.49

Figure 20. Force and Displacement Boundary Conditions for Flexible Pavement System Reinforced with Piles (From Ref. 26).

ELECTRO-CHEMICAL

HARDENING OF EXPANSIVE CLAYS

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS

AND

THE FEDERAL RAILROAD ADMINISTRATION

Little Rock, Arkansas

August 21 and 22, 1975

INTRODUCTION

Sponsorship

This report is based on work performed by the Arizona Department of Transportation, Research Section, with principal investigator Dr. Frank P. Mancini and Chief Consultant Dr. Charles O'Bannon to investigate "Electro-Chemical Hardening of Expansive Clays." Funds for the project were provided by the Arizona Department of Transportation in cooperation with the Federal Highway Administration.

Object of Research

The object of this research was to utilize existing 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.

LITERATURE REVIEW

Since this work with electro-osmosis is primarily concerned with stabilization of Chinle Clay formations it would do well to quickly review certain facts and models of clay minerology and electro-osmosis.

It is well known that as the soil particle size decreases the appearance and behavior of soils alter. These changes in soil behavior are due to the increasing effect of the forces between 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. Thus, 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 an 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 in the study of physical chemistry.

Formerly, it was believed that clay minerals were amorphous in character, but investigations carried out with improved equipment and techniques since the 1920's have shown them to be predominately crystalline, moreover, it is found that clay minerals are hydrated aluminum silicates in a crystalline form. Although the molecular structures are complicated, it has been shown that clay minerals are constructed essentially from two basic building blocks: The silica tetrahedron (SiO_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.

Base Exchange

The absorbed ions on a clay surface are present in a diffuse double layer. Each of the ions required to neutralize the change 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, speaking in terms of a sodium ion and a potassium ion, the potassium ion should be held nearer the surface with a correspondingly greater force than a sodium ion.

It must be borne in mind that an ion-exchange soil treatment can alter the physicochemical properties of the swelling clay, and it is one of the most effective ways to combat the swelling clay problem over a relatively small localized region on which an expensive inplace structure 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 mineral surface of the clay and the size of the clay particle. 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 substitutions within the lattice of the mineral and also to the number of broken bonds around the edges of particles and, hence, to the size of the clay platelets. The additional absorbed ion may be sodium or Ca^{++}, K^{+}, Li^{+}, Fe^{++} or other cations. The presence of the other ions does not alter the basic clay chemical formula, but they play a significant role in altering the engineering behavior of the clay mineral, since they affect the manner in which the various basic structural sheets are connected to one another and occupy spaces on the surface of the clay particle, which are thereby made unavailable to other cations or may interfere with the development of water layers at the surface.

After analyzing this base-exchange phenomena, the author employed electro-chemical stabilization to the problem of swelling clays. From this study it was determined 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.

LABORATORY WORK

General

The purpose of the laboratory work 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 323.8± on the westbound lane of Interstate 40, approximately 40 miles east of Holbrook, Arizona. For a plan view of the site see figure 1. Since this project was initiated as an implementation study in engineering, decision as to pre-test sampling technique was made at the onset of the project.

This engineering decision is based on the following general considerations: most problem areas of swelling clays traversed by a roadway are easily identifiable visually. After an area has been identified 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 Electro-osmatic work and apply the same techniques properly modified to enable a rapid evaluation of the swelling problems of a given area.

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 2. The samples were transported to the Arizona Highway Department Materials Division where they were prepared and used in laboratory studies.

Sample Preparation

The samples were prepared in the following manner, see figure 3. 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.

This method enabled sufficient soil samples to be obtained from which a proper engineering estimate of the swelling conditions in this 500' section could be made.

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 20% was 2 micron size. 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 1 for these data.

EXPANSIVE PRESSURE AND PERCENT SWELL TESTS

The expansive pressure of the untreated soil was determined 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.

DETERMINATION OF OPTIMUM gm KCl/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 electro-chemical effects induced by the electrical 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 4 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 electrochemical process, was adjudged to be sufficient for our purposes.

FIELD WORK

General

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

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

(2) To sample the electrochemically treated section.

(3) Evaluate the ability of existing maintenance resources to carry out this work with a minimum of specialized personnel in attendance.

Field Test

Design - The highway field installation was designed to reproduce the simple electrode configurations used in previous studies. To this end a design was chosen that incorporated the desirable characteristics of previous work with the realities of conducting the field work on a much larger scale and with relatively inexperienced personnel.

The electrical design was such that 3 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 5. It should be noted that the center section has, as anodes, vertical #8-1" rebar about 5' long. This is in contrast to the other two sections which have horizontal anodes made up of 20' sections 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' deep trench.

The cathode was formed of 20' 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 designed patterns exist 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.

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. 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', say 4', would result in an "overkill."

In order to properly suffuse the soil pores with the KCl solution, it was decided to drill 6" diameter auger holes on 8' centers, approximately 5.5' deep throughout the test section. The positioning of the auger holes was based on previous studies. There was a total of 285 of these auger holes positioned throughout the site.

To prevent caving of the blowsand subbace material each auger hole was sleeved with a 6" O.D. steel pipe, 27" long, topped with a 10" diameter, 1" thick steel plate. The plate had a 1 1/2" hole in the center to permit introduction of the KCl solution into the auger hole. The 10" diameter steel plate was obviously necessary to give a stable platform for the steel sleeves and to provide a sufficiently rigid surface for vehicular traffic.

The 10" diameter, 1" thick steel plate welded to the top of each steel sleeve certainly would give a rough surface to the roadway throughout the length of the 500' test section after the sleeves were in place. Obviously 285 protruding 10" diameter steel plates presented a potential traffic problem and made it necessary to countersink each auger hole with a concentric circular 10" diameter, 1" deep depression. The countersinking operation was accomplished easily by welding to the top of the auger drill stem one of the 10" steel plates.

After this was accomplished, 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.

Each of the 285 auger holes was sleeved down to 27" 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" 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" below the estimated base course material.

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.

The manner of mixing and placement of solution was accomplished by District IV personnel using a modified 2500 gallon capacity "goose neck", water truck. The water and KCl was mixed so that a 0.4 N solution was obtained.

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 liter of water.

Given the outside ambient water temperatures in the Holbrook area at that time of year (May through August about 23°C), this was the maximum amount of KCl soluble in water at that temperature. To attempt 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. In this manner the auger holes were filled twice a day for a period of approximately one month, in which time about 35,632 gallons of 0.4 N solution was introduced into the site region.

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 0.1 volt/cm. On the average about 133 amps flowed through each of the three sections during the field test. After approximately 30 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 6 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.

RESULTS AND ANALYSIS OF FIELD TEST

Results of Field Test

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

The information obtained from (1) would give data as to the effects of the electrochemical treatment on the engineering properties of the soil mass. Data obtained from (2) would yield information as to the effects on the lattice structure of the clayey soil. Part (3) compliments part (2) 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.

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 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 Braggs' 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 heat treatment of a dry oriented mount is useful to identify various clay minerals from the characteristics spacings which result from the different water layer thickness 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 relative amounts of the various clay and non-clay minerals are identified by the headings major, minor, or trace. 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 predominantly of smectite (montmorillonite).

A rough idea of the relative amounts of the various minerals is obtained from a comparison of the relative intensities of the characteristic peaks. Usually the non-clay mineral fraction is more abundant in accordance with the majority of the 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 than non-clay minerals due to their large specific surface area.

Conclusions from X-Ray Diffraction Data

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 Montomorillonides 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 typical X-ray diffraction pattern for the treated material is shown in figure 7.

Table 3 gives, in tabular form, the results of the X-ray 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 limitations of the project. However, some definite inferences can be drawn from this data.

It is apparent that the electrochemical treatment of Chinle clay does indeed modify the clay lattice, in particular the Montmorillonoids present in the clay are definitely altered in a fundamental fashion.

Transmission Electron Microscope Analysis

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

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

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

The acetone extracted something from the samples which showed up on the mocrographs 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 were not as well dispersed as was the post-test sample.

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 thin layers tended to remain build up into thicker crystals.

The diffraction patterns for the post-test materials, figure 8 and 9, shows diffraction spots on the circles where the pre-test diffraction pattern shows that some of the diffraction spots are displaced from the circles, figure 10 and 11. When diffraction spots are displaced from the diffraction rings, this indicates that the crystal layers 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.

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 suspension. 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 12.

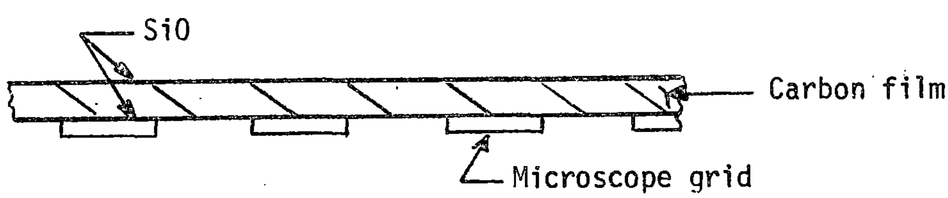


Figure 12

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 of 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 treated samples contain the postassium ions which neutralize the excess charges on the clay layers and reduce the attractive forces between them so that they can be dispersed more readily.

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 13.

The Old E.O. Site Crystals appeared to look like the post-treated samples as shown in figure 14.

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 structure 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 in 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 15 and 16, it is shown how the layers of the treated material appear much more transparent to the electrons than does the untreated material, indicating respectively, very thin layers as opposed to very thick layers.

Expansive Pressure and % Swell of Past 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.

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 realiability 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 figure 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 electro-chemical treatment.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This study focused primarily on 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 operation is 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. supplimented by minor purchases of specialized hardware and software.

It is the opinion of the research team that this process offers a viable method which will reduce or eliminate excessive swelling pressures if they are encountered in an area over which an existing structure rests.

Conclusion

(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) It is the opinion of the Research Team that the technique is economically feasible when compared to the continual maintenance and overlays required on roadways over this material.

(3) From the X-ray diffraction and transmission electron microscope data it is evident that the electrochemical treatment with KCl solution is very effective in permanently altering the mineralogical characteristics of Montmorillonites and thereby reducing the expansive pressure of the soil. The observed collapse of the crystalline lattice supports this conclusion.

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

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

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

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

(8) 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 noticeable heating of the soil which is an indication of high energy loss and waste associated with this loss.

Evidence from field tests indicate that the voltage gradient within the soil mass is not constant but changes as treatment progresses. The average gradient behind the leading edge of the ion-front steadily decreases whereas that between the leading edge and the cathode steadily increases.

Recommendations

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 on in-situ material. This will range from 3' to 5'. A typical design figure would be 4'.

(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 wt. of soil.

(4) Prepare site for electrochemical treatment using 6" diameter auger holes sunk approximately 6' below surface on 8' centers. Sleeve auger holes with 6" O.D. steel pipe with a 10" diameter plate welded to the top of the pipe. The 10" diameter plate will have a 1 1/2" 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".

(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.

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 $\#/ft^3$	% - Water	Expansive Pressure $\#/ft^2$	% - 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	

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	Aristobalite
#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	

TABLE 4

Expansive Pressure & % Swell Tests

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

TABLE 4 Cont.

Expansive Pressure & % Swell Tests

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

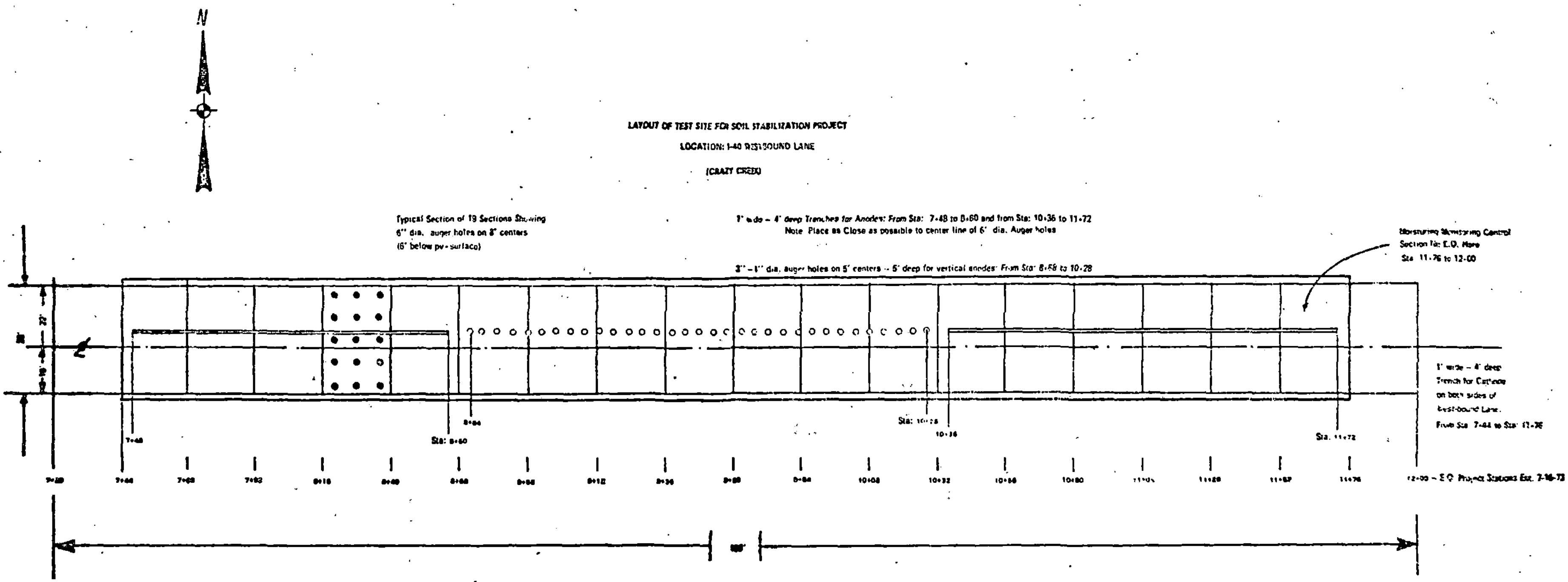
TABLE 5

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

Hole #	Depth	Sample #	Initial Moisture Content	Initial Dry Density	% Swell	Initial Moisture Content	Initial Dry Density	Expansive Pressure (PSF)
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



N
LAYOUT OF TEST SITE FOR SOIL STABILIZATION PROJECT
(CRAZY CREEK)
Typical Section of 19 Sections Showing
6'' dia. auger holes on 8' centers
(6' below pv-surface)
Note Place as Close as possible to center line of 6' dia. Auger holes
Sta: 8+60
Sta. 11+72
Section
Sta 11+76 to 12-00
Trench for Cathode

Figure 1

Note Stations 0 + 00 – 13 + 400 were set down July, 1973 by Dist. IV for Project HPR-1-11 (145)

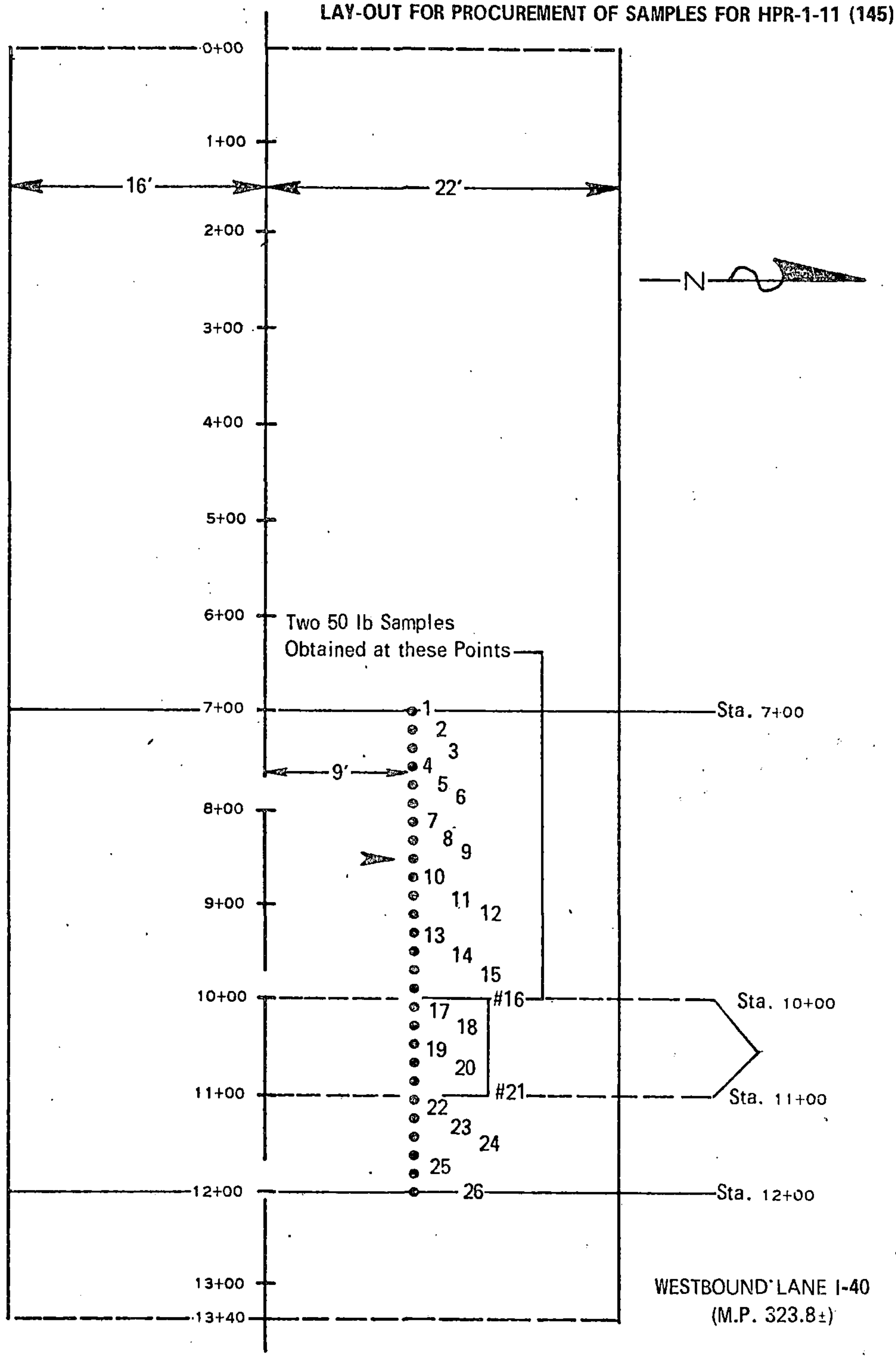


Figure 2

FLOW CHART FOR EXPANSIVE PRESSURE

(% SWELL FOR UNTREATED SAMPLES)

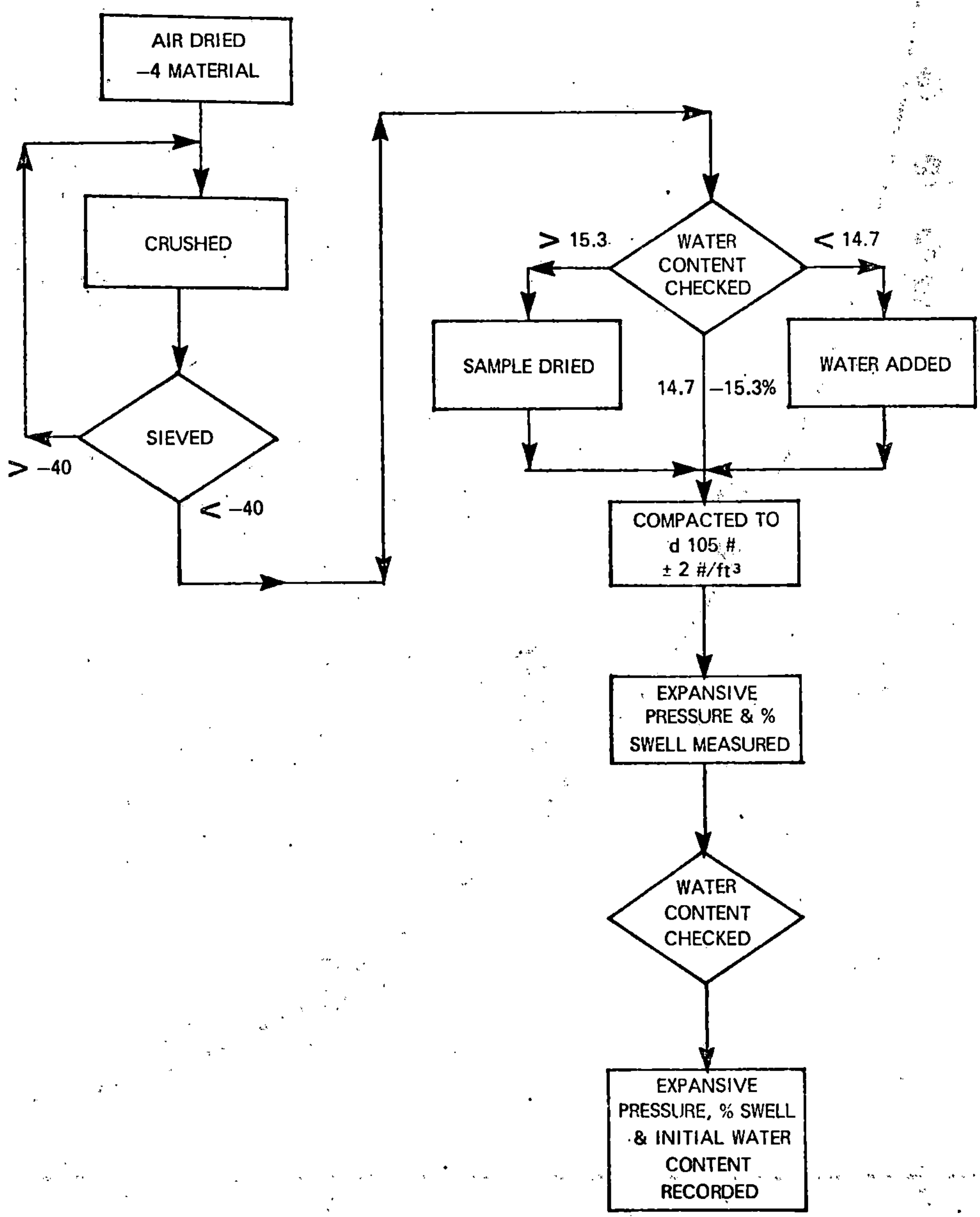


Figure 3

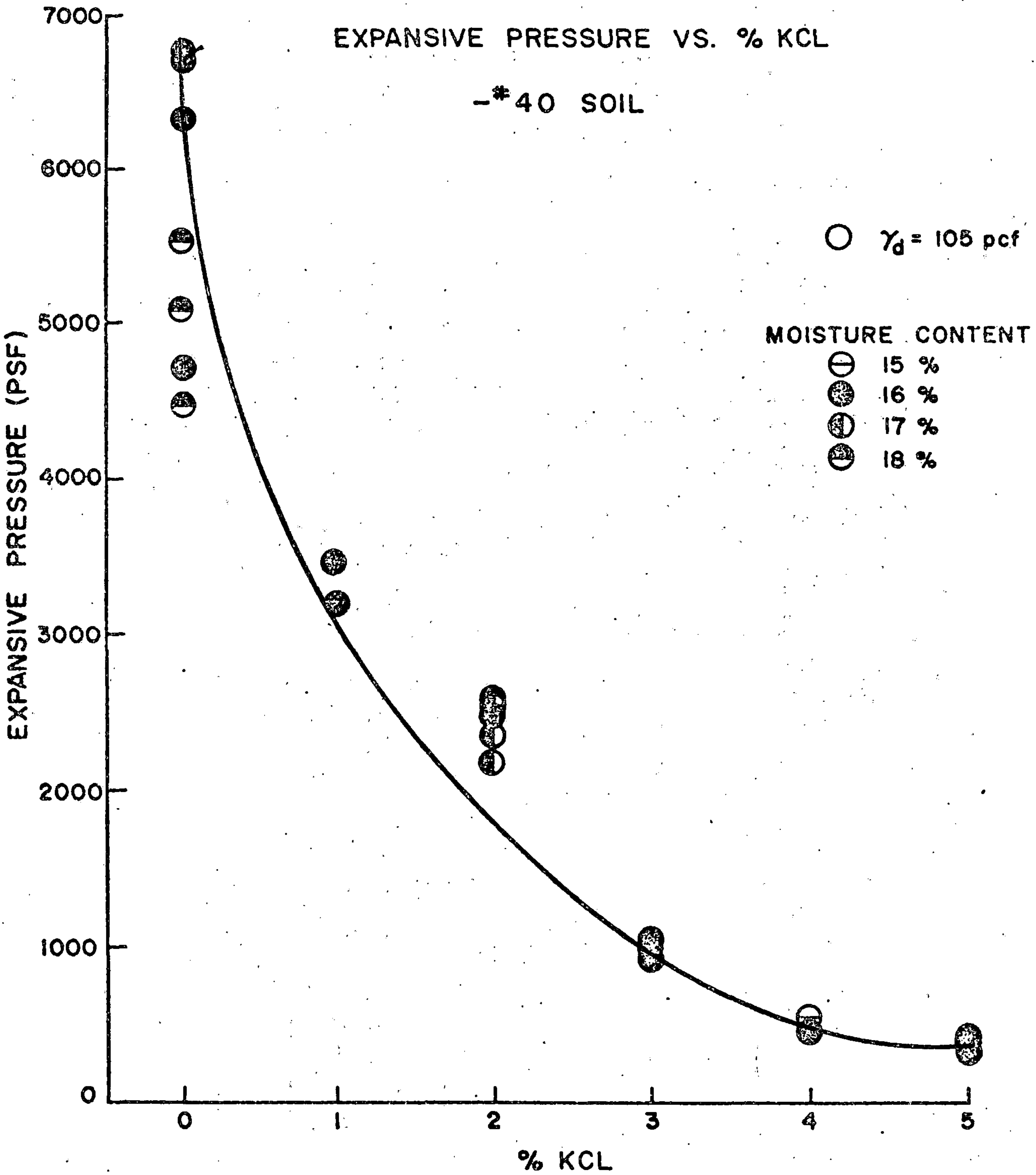

EXPANSIVE PRESSURE VS. % KCL
-#40 SOIL
γ_d = 105 pcf
MOISTURE CONTENT
15 %
16 %
17 %
18 %
EXPANSIVE PRESSURE (PSF)
7000
6000
5000
4000
3000
2000
1000
0
0
1
2
3
4
5
% KCL

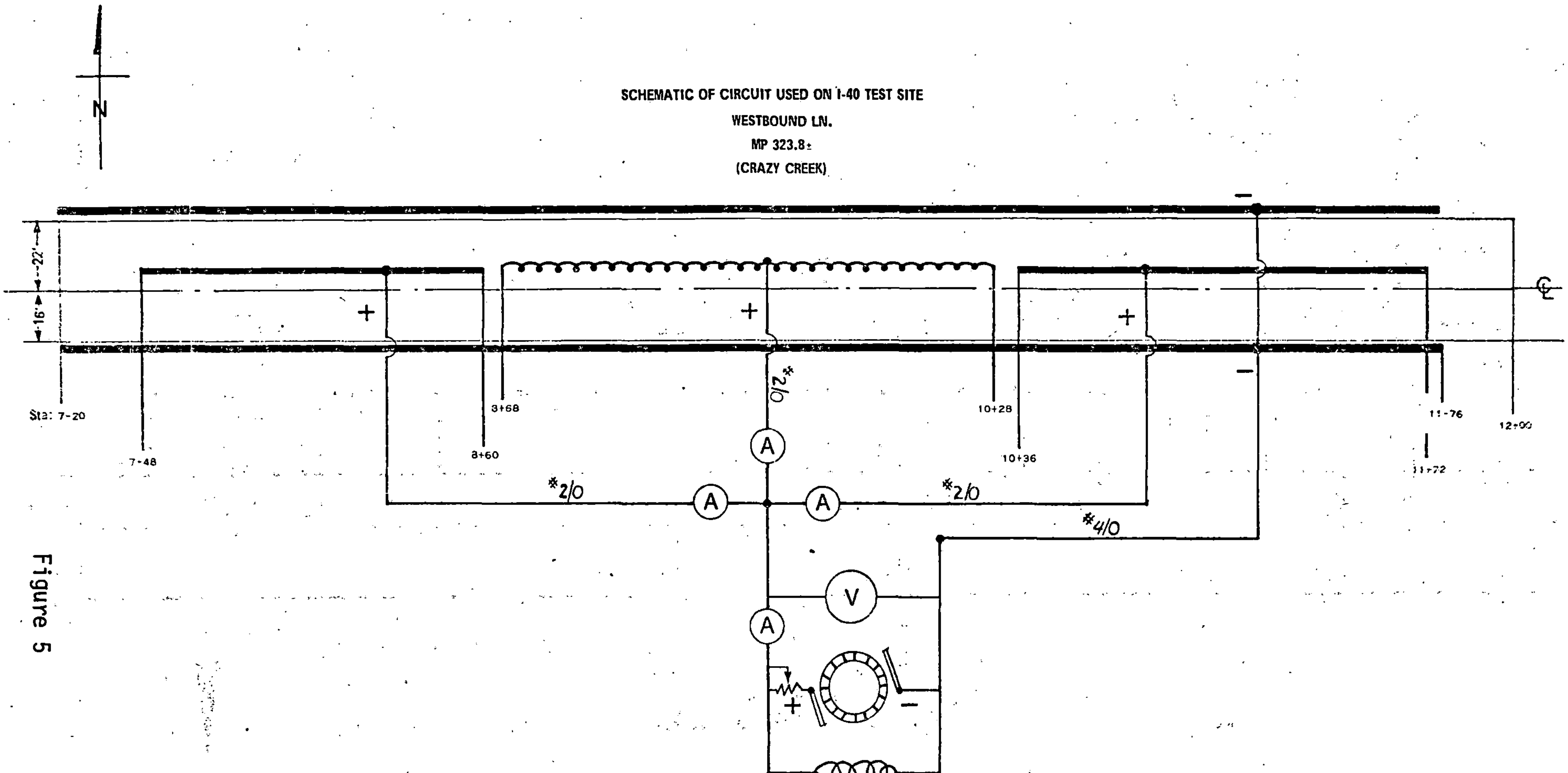
Figure 4

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)

Figure 5

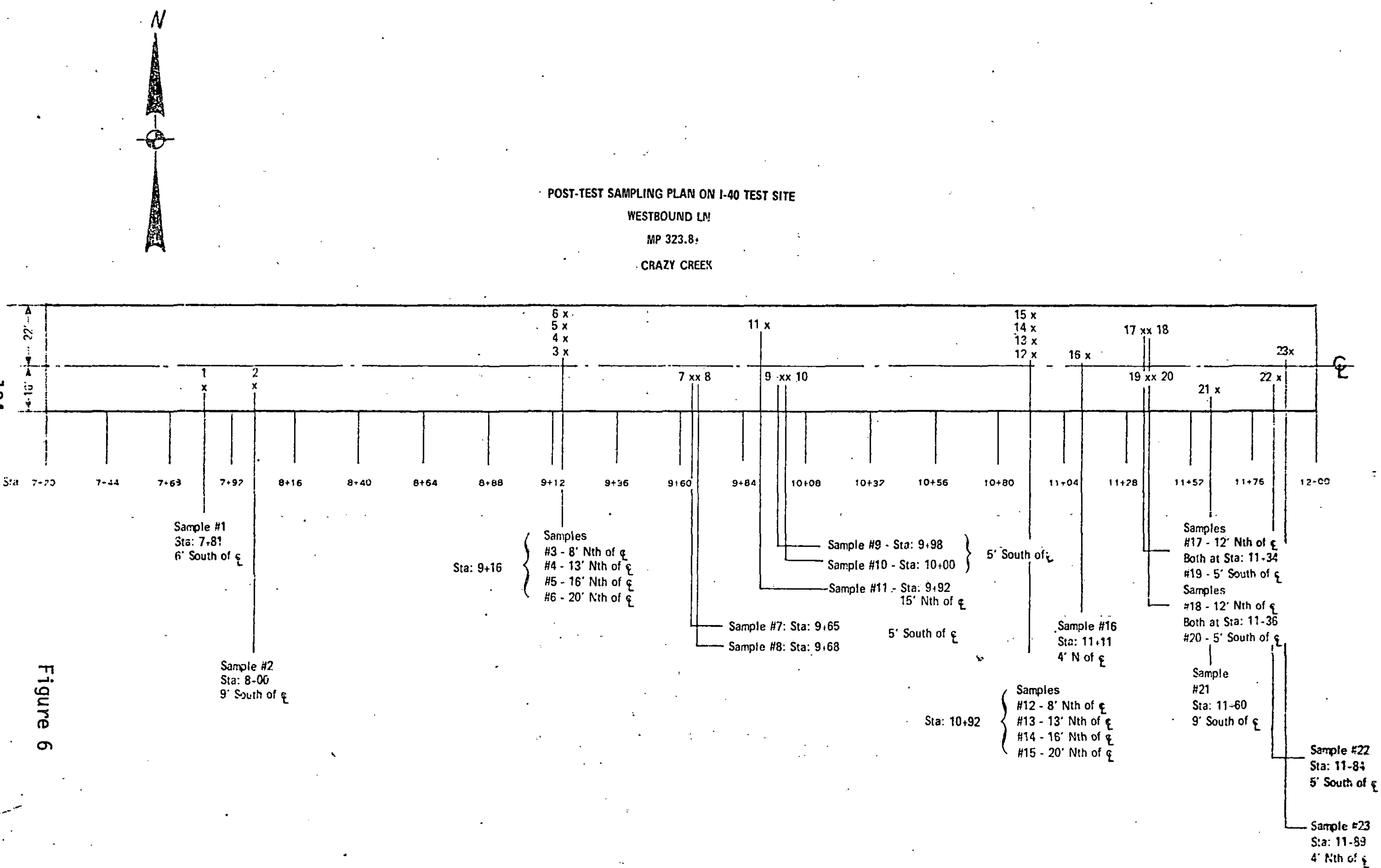
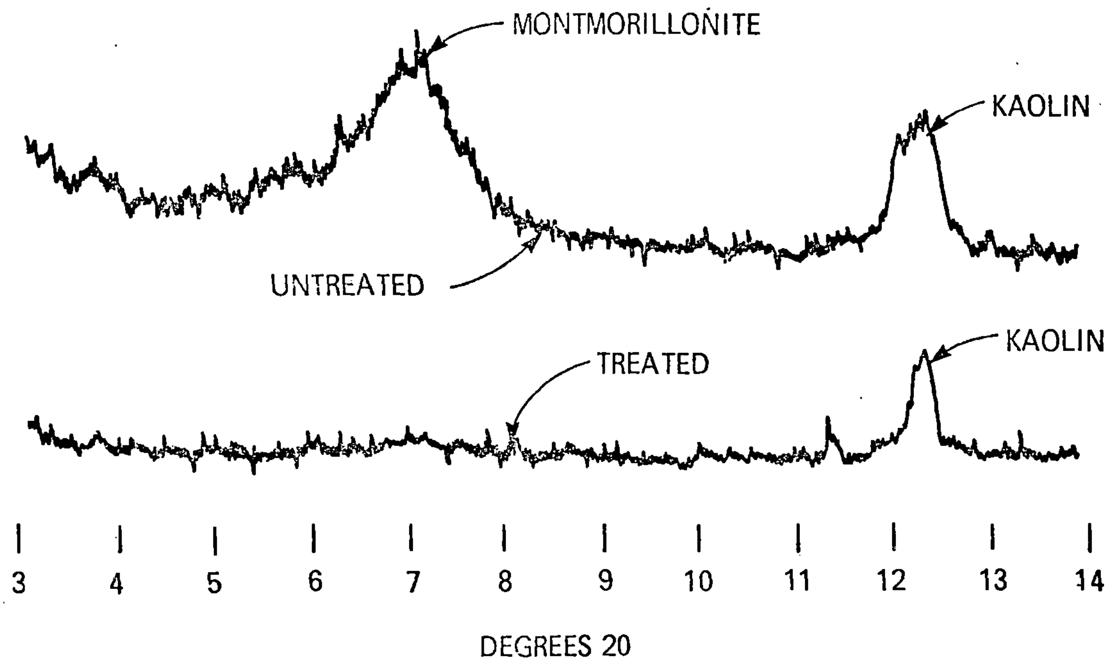

N
POST-TEST SAMPLING PLAN ON I-40 TEST SITE
WESTBOUND LN
MP 323.8±
CRAZY CREEK
22'
4-10'
℄
6 x
5 x
4 x
3 x
11 x
15 x
14 x
13 x
12 x
16 x
17 xx 18
23x
1 x
2 x
7 xx 8
9 xx 10
19 xx 20
21 x
22 x
Sta 7+20 7+44 7+68 7+92 8+16 8+40 8+64 8+88 9+12 9+36 9+60 9+84 10+08 10+32 10+56 10+80 11+04 11+28 11+52 11+76 12+00
Sample #1
Sta: 7+81
6' South of ℄
Sample #2
Sta: 8-00
9' South of ℄
Sta: 9+16
Samples
#3 - 8' Nth of ℄
#4 - 13' Nth of ℄
#5 - 16' Nth of ℄
#6 - 20' Nth of ℄
Sample #7: Sta: 9+65
Sample #8: Sta: 9+68
5' South of ℄
Sample #9 - Sta: 9+98
Sample #10 - Sta: 10+00
5' South of ℄
Sample #11 - Sta: 9+92
15' Nth of ℄
Sample #16
Sta: 11+11
4' N of ℄
Sta: 10+92
Samples
#12 - 8' Nth of ℄
#13 - 13' Nth of ℄
#14 - 16' Nth of ℄
#15 - 20' Nth of ℄
Samples
#17 - 12' Nth of ℄
Both at Sta: 11+34
#19 - 5' South of ℄
Samples
#18 - 12' Nth of ℄
Both at Sta: 11-36
#20 - 5' South of ℄
Sample
#21
Sta: 11-60
9' South of ℄
Sample #22
Sta: 11-84
5' South of ℄
Sample #23
Sta: 11-89
4' Nth of ℄

Figure 6

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 7

Figure 8

Post-test Material, Diffraction Pattern

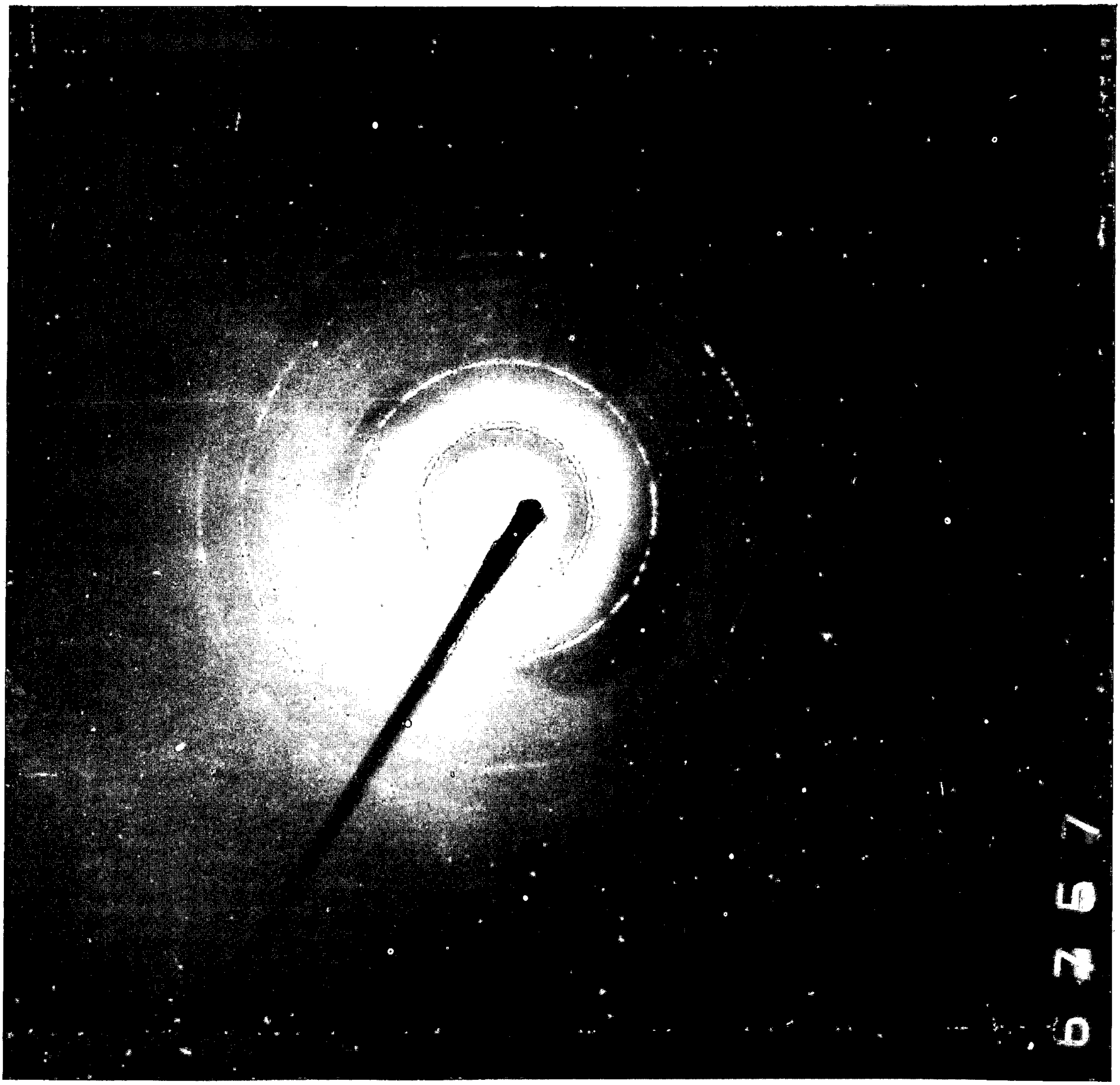


Figure 9

Post-test Material, Diffraction Pattern

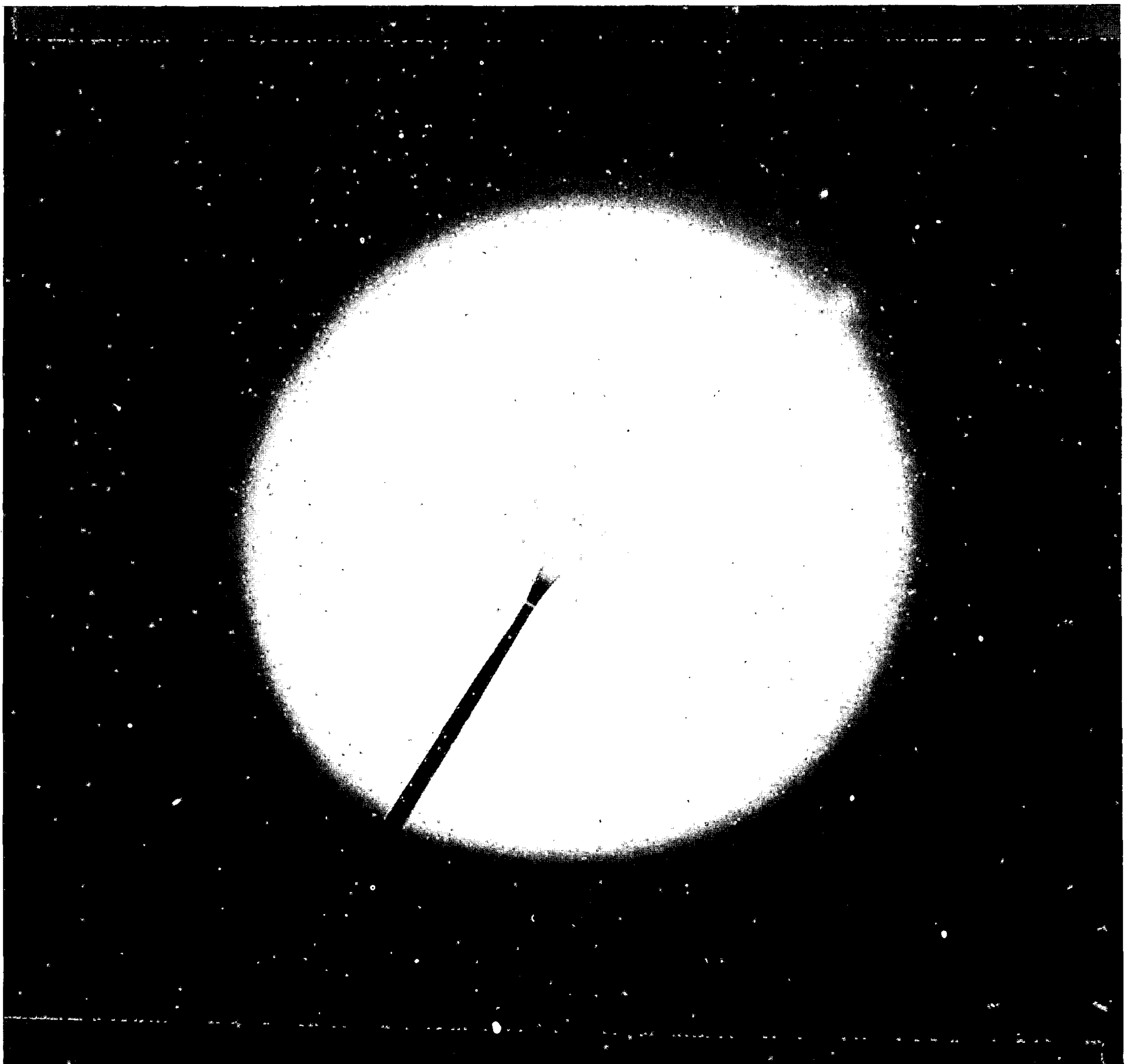


Figure 10

Pre-test Material, Diffraction Pattern

Figure 11

Pre-test Material, Diffraction Pattern

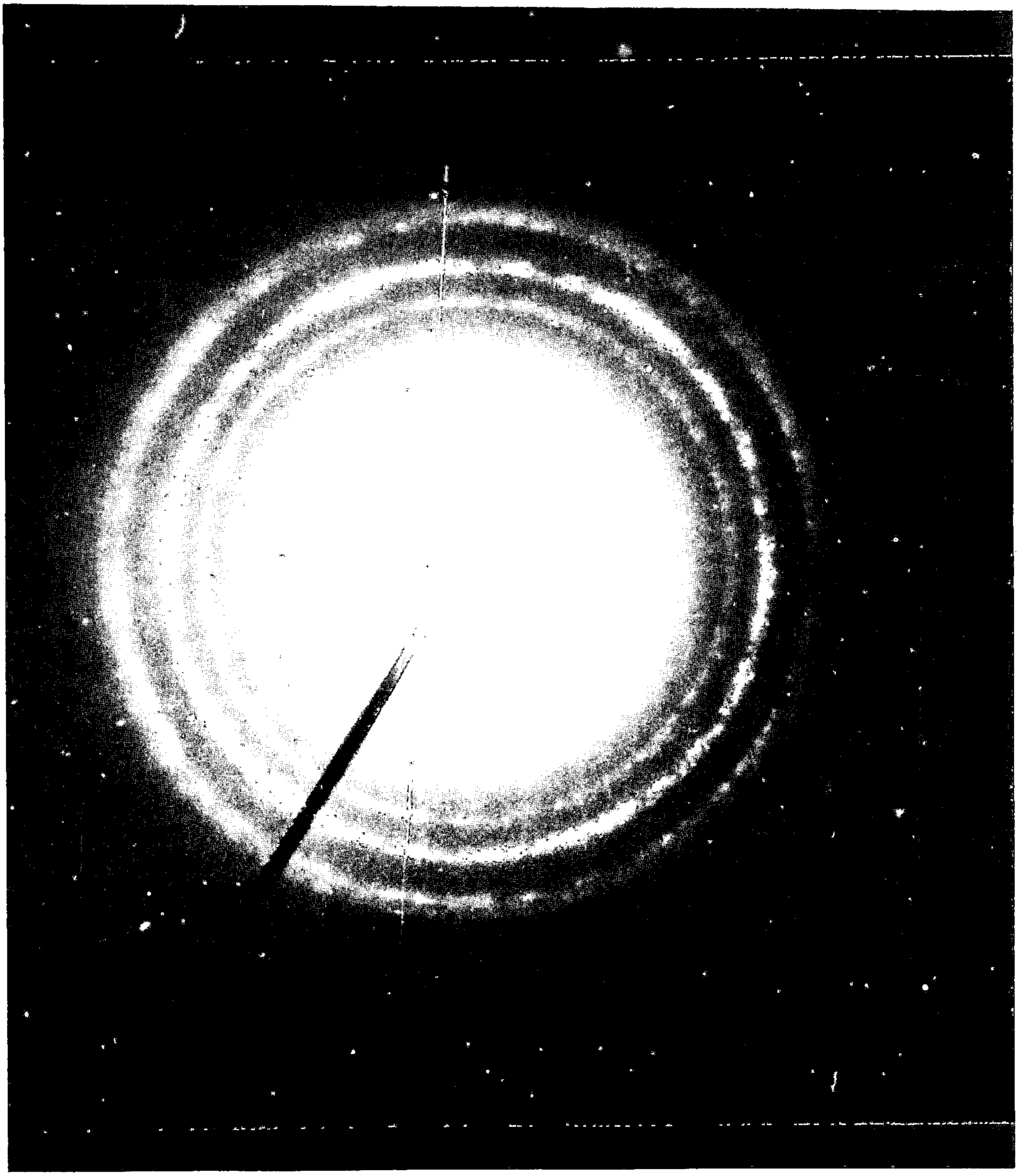


Figure 13

Wyoming Bentonite Electron Diffraction Pattern

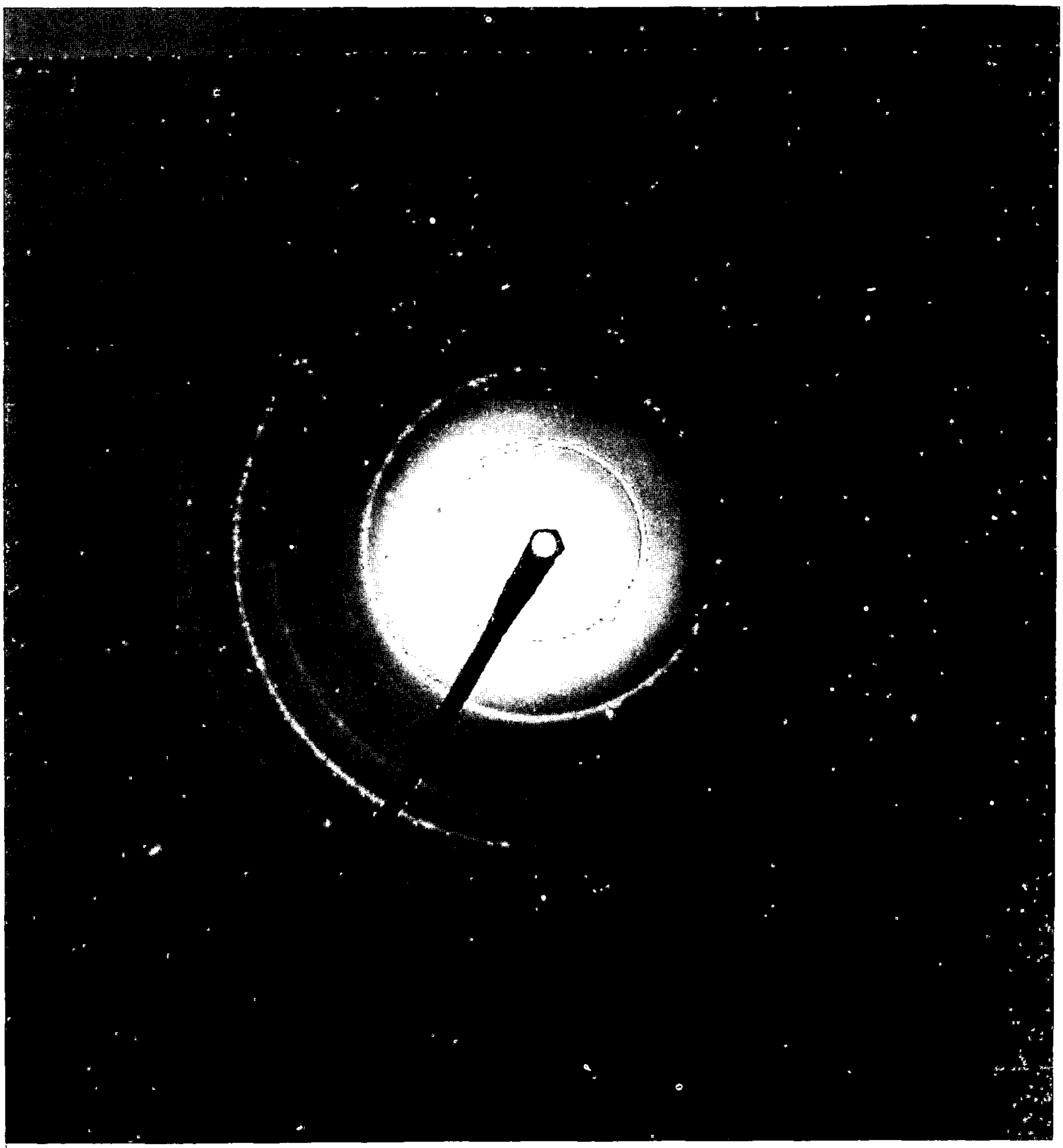


Figure 14

Old E.O. Site Diffraction Pattern

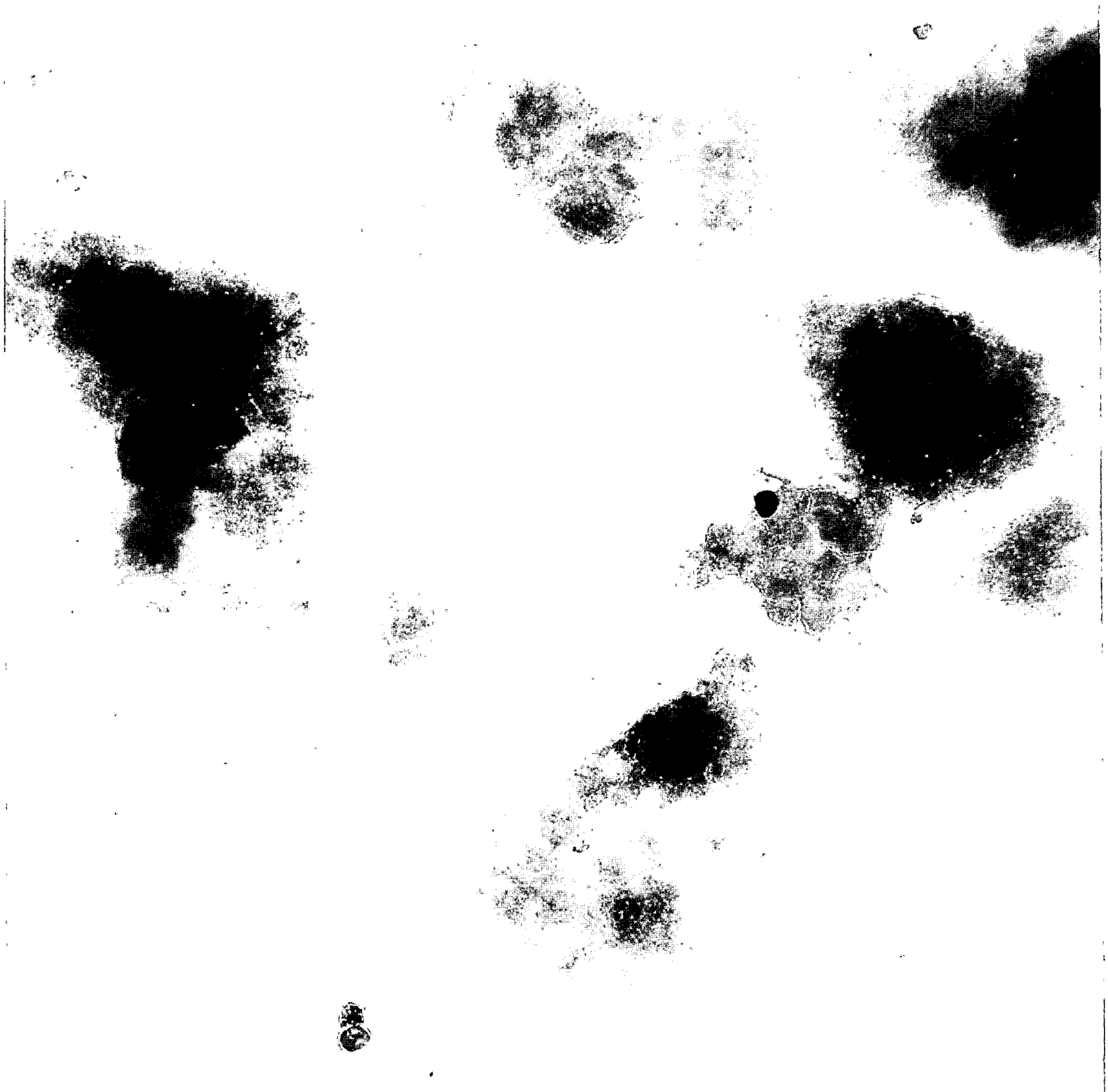


Figure 15

Electron Transmission Micrograph (Treated Material)

Figure 16

Electron Transmission Micrograph (Untreated Material)

ENVIRONMENTAL CONSIDERATIONS

By

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ROADBED STABILIZATION ; LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS

AND

THE FEDERAL RAILROAD ADMINISTRATION

August 21 and 22, 1975
Little Rock, Arkansas

ENVIRONMENTAL CONSIDERATIONS

Lime Slurry Pressure Injection techniques do present the potential for adverse environmental effects if reasonable care and precautions are not used in the implementation of this technique. The purpose of this presentation is to present the apparent adverse effects that can be a result of lime slurry injection and the means of avoiding these problems. The adverse effects can be categorized into three divisions. These are physiological, botanical and aquatic.

Adverse Environmental Effects:

The physiological effects include those potential effects to man as well as the aquatic biota. Injecting fluids into the ground does present the potential for the contamination of a well. Spillage of the slurry into local waterways can cause fish kills by either the presence of toxic materials or a pH adjustment in the waterway. The aquatic effects can include the fish kills mentioned above as well as the initiation of algae blooms. The addition of phosphate with a concurrent adjustment of pH in waterways could contribute to the occurrence of an algae bloom (5). Botanical effects consist of denuding of the right of way due to the pH alteration of the soil. Fish kills, algae blooms and vegetation destruction are highly visible effects and will lead to the most immediate reaction in the community.

Lime:

Lime contains some trace materials which are of concern. Analyses obtained from vendors lists the presence of arsenic and flouride. The current proposed safe drinking standards under Public Law 93-523 is 0.05 mg/l for arsenic and a maximum level of 1.4 to 2.4 mg/l for flouride depending upon water temperatures (4). The levels reported in lime analysis was low - 0.368 ppm for arsenic and 0.260 ppm for flouride. Analysis conducted on lime samples obtained from three vendors indicated that arsenic was not present in readily detectable amounts. Zinc and manganese were detected.

Analyses were made for arsenic, barium, cadmium, lead, selenium, silver, zinc and manganese. None of these materials would present a significant problem of ground water contamination at the present level of use of lime in the slurry.

The lime contains approximately 0.1% phosphate. In ppm this is equivalent to about 1000 ppm. The current limit accepted for the limitation of algae blooms in a waterway is 0.01 mg/l phosphorous. Apparently there is significant amounts of phosphorous in the slurry. The phosphate problem is compounded by the use of commercial detergents as wetting agents which are in excess of 50% phosphate builders. Spillage of lime slurries into surface waters can potentiate eutrophication of these waterways. The Arkansas State Standard is .001 mg/l as phosphorous in streams and less than 0.05 mg/l in lakes (1). Assuming a 6% lime slurry it would require approximately 40.0 gallons of dilution water per gallon of slurry to stay below the state lake standard with regard to phosphorous. Spillage of excess lime should be avoided.

Lime contains sulfates. Sulfates can be reduced in anaerobic environments to H_2S and cause objectionable odors in well water. The sulfates are reduced in the presence of organic substrates which are oxidized in the process and act as hydrogen acceptors. This is a problem only when there is present some other organic material to be oxidized by microbiological action in the ground water.

The polyvalent cations in the slurry will displace monovalent cations in the clay (10, 7). There will be slight increases in dissolved sodium and potassium in the ground water around the injection site. The hardness of these waters is likely to increase in the area surrounding the injection site. Current data on the epidemiological significance of moderately hard waters compared to soft waters suggests this will have a beneficial effect (8). In total the change in mineral content of well water adjacent to the site would be negligible.

The lime slurry consists of lime and a dispersant. The lime is a strong base (i.e.) it raises the pH of the water it is in contact with. The change in the pH that is significant is of some concern. Table (1) is the pH tolerances of some fish listed in the literature (6). It is apparent that depending on the starting point of the water and its acidity the amount of lime per liter (or gallon) that would be required to adjust the pH a given amount varies. The limits are also dependent on the species of fish present. Fish kills have occurred in streams adjacent to lime slurry pressure injection sights. Lime spilled into the pond or stream is not stabilizing the soil. Excessive pumping of the lime slurry to refusal and beyond and careless dumping of excess lime slurry are the causes of problems with fish kills. These are avoidable problems. Most states have financial penalties for discharges that result in fish kills.

Most soils are neutral to slightly acid in pH. The purpose of the lime injection technique is to increase the pH to 10.3 or above (10). The native plants will not thrive in soil of this pH. Thus you can expect some denuding of the right of way. Eventually vegetation cover of some type will return. Lime is not an expensive chemical. Careful limitation of the amount used is not treated as a high priority item. It should receive more attention because of the apparent and more visible effects it will have locally.

Additives:

The addition of surface active agents to slurries poses some additional problems besides nutrients. Care should be exercised in the selection of the additive. Quite a few surface active agents have undesirable physiological effects. Table 2 lists the toxic concentrations of some surfactants to common fish species (6). The use of any chemical should require an initial check of the Toxic Substance List compiled by the National Institute of Occupation Safety and Health for known carcinogenic, mutagenic, teratogenic or toxic effects.

Carcinogenic refers to cancer inducing agents. Mutagenic refers to agents that cause genetic mutations. Teratogenic agents are those substances which cause abnormal fetal development. Toxic materials are those which cause illness or death. The list of known carcinogenic materials is expanding rapidly. The concern over the New Orleans drinking water supply was based on the determination of trace amount of carcinogens in the water. An example of a common material which was found to be teratogenic is nitrilotriacetic acid (N.T.A.). It was initially used as a substitute for phosphate builders in detergents. After it was put in use teratogenic effects were demonstrated in animals given doses of N.T.A. Thus its use has been curtailed. Often common materials we use have known deleterious effects if administered orally at high doses. Analytical techniques have progressed to where a very minute amount of a material can be detected.

Currently there is public concern over the quality of drinking water. This is reflected in passage of the Safe Drinking Water Act. Public action groups will not hesitate to bring suit against contractors if there is suspicion that they have endangered local water supplies. Time spent on determining what is actually in the additives that are used could save a contractor from extended litigation.

Public Law 93-523, The Safe Drinking Water Act, requires that in three years permits will be required for subsurface chemical injection (2). The permits would be State administered and require accurate chemical description of materials being injected. This act contains penalties and public notification requirements of all water users of any contaminated water supplies, thus caution in the use of injecting chemicals in the vicinity of wells will be warranted. There is question as to whether the pressure injection technique will fall under the scope of this federal law. The question comes from the definition of chemical injection. In the law it is defined as subsurface implacement of fluids by well injection. Whether or not this will cover pressure injection grouting techniques is left to conjecture.

In June of 1974 Engineering News Record carried a story concerning the banning of soil grouting at Warita Airport, Japan (3). A local court ruled that construction on the project be halted until the safety of the grout being used was determined. In March, 1974, at Fukuoka, Japan, five residents suffered hallucinations after drinking well water that contained acrylamide from a nearby grouting site. American Cyanamid which markets acrylamide grout in this country specifically warns about the potential danger in the vicinity of drinking water supplies.

The company that received the court order was using a cement grout. It had no relationship to the previous incident. The public concern made it politically necessary to ban grouting. This incident indicates that a company should keep accurate records of the chemicals it uses in all of its grouting sites to defend itself in litigation.

Synergism:

There are well known synergistic effects of toxicants to fish. When a population is in stress its sensitivity to toxic material increases. The pH alteration that is concurrent with the discharge of the slurry into waterways will enhance the toxicity of the chemicals to the fish. Thus the risk of fish kills can be higher than was first apparent. This risk comes from enhancement of the toxicity of chemicals present in the waterway as well as those added by the slurry.

Summary:

The potential visible effects of lime slurry pressure injection on the environment are fish kills, algae blooms and destruction of vegetation. These can be avoided by limiting the amount of excess pumpage of lime and by careful disposal of excess lime from the slurry tanks. The physiological effects of pressure injection can be avoided by obtaining a complete chemical description of all additives used. These should be research in a toxic chemical list and questionable compounds avoided. Reasonable care in application and selection of injection material should be sufficient to avoid problems.

In conclusion lime slurry pressure injection soil stabilization can be employed with minimal environmental impact. Awareness of potential adverse effects and their causes can avert and should preclude any serious problems with this technique. Control applications will be the essential mechanism of averting problems.

TABLE I

pH TOLERANCES OF SOME FISH SPECIES

Fish Species	pH Limits Lower	Upper
Trout	5.4 to 6.0	7.9 to 9.2
Minnow	5.2	-
Gold Fish	4.2	9.5 to 10.6
Large Mouth Bass	-	9.6

TABLE 2

TOXICITY OF COMMON DETERGENTS TO FISH

Detergent	Species	Lethal Dose	Water	Exposure/Hours
Package Detergent	Minnow	41-85 p.p.m.	Soft	96
		15-87 p.p.m.	Hard	96
Alkyl Benzene Sulphonate	Minnow	4.5-23 p.p.m.	Soft	96
	Minnow	3.5-12 p.p.m.	Hard	96
Sodium Lauryl Sulphonate	Minnow	5.1 p.p.m.	Soft	96
		5.9	Hard	96
Sodium Tetrapropylene Benzene Sulphonate	Rainbow Trout	12	-	6
Detergent				
Sodium Pyrophosphate	Rainbow Trout	1120	-	27
Sodium Tripolyphosphate	Rainbow Trout	1120	-	-
Sodium Tripolyphosphate	Minnow	140	-	-

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10. Thompson, Marshal, "Influence of Soil Properties on Lime Soil Reactions", Public Works, August, 1965.

SELECTED CASE STUDIES
OF LIME INJECTED RAILROAD TRACK

By

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

Little Rock, Arkansas
August 21 and 22, 1975

SELECTED CASE STUDIES OF LIME INJECTED RAILROAD TRACK

To assist in evaluating the field performance of lime injection under railroad loading conditions, a case study program was initiated by the University of Arkansas Railroad Research Team. The purpose of the study was to collect quantitative and qualitative information about track condition, roadbed, injection procedures, and site conditions before and after lime stabilization. This information was acquired from the participating railroads and the lime injection contractors. Whenever possible, visits were made by the Railroad Research Team to the site before and after lime injection. Reports of the track condition before injection were requested from the responsible division engineer. The lime injection contractors have been extremely helpful in providing weekly reports of the location and quantity of lime used for most of the jobs initiated during the research contract. Soil samples and subsurface information have been obtained at selected sites utilizing contracted drilling rigs.

To date, studies have been conducted in 10 states with the cooperation of 12 railroads or industries. The Railroad Research Team plans to make subsurface investigations at sites in Mississippi, Louisiana, South Dakota, and Texas. The boring programs will be followed by laboratory testing with reports of the results to the cooperating railroads. The following case studies were selected as being representative of the roadbed conditions encountered at many of the sites.

Forrest City, Arkansas

The main east-west line of the Chicago, Rock Island and Pacific Railroad between Memphis, Tennessee and Little Rock crosses the flat prairie region of Eastern Arkansas. Near Forrest City the railroad is constructed on an earth embankment as it passes through the lowlands of the St. Francis River. The height of the fill embankment supporting the roadbed varies from 10 to 15 feet high in most of the 10 mile long problem area. Typical daily rail traffic consists of over 800 cars weighing nearly 50,000 tons.

Fill material for construction of the roadbed was probably obtained from the adjacent prairie soils. Borings made in the fill material indicate that the soil below the ballast layer consists chiefly of highly plastic gray clay, CH according to the Unified Soil Classification system. The location of the borings is shown on the Plan of Borings, Figure 1. A typical boring log is also shown on Figure 2. Occasional layers of cinder, sand, and sandy clay were encountered at several locations. The borings were drilled to a depth of 20 feet by Barrow-Agee Laboratories of Little Rock in December 1974.[1]

Stability problems (evident to the untrained eye) were observed at numerous locations. Track out of level and alignment was noticed, and slide areas where the embankment failed were also apparent. The lengths of the "soft spots" varied from approximately 50 feet to several hundred feet. The site in Figure 3 is an example of an unstable area near the Heth crossing.

A limited laboratory testing program was conducted at the University of Arkansas (Fayetteville and GIT) facilities. The purposes of these tests were to: 1) aid in the classification and identification of the soil samples, 2) measure the strength and volume change properties of the soils, and 3) determine what, if any, beneficial effects lime might have on the soil. Tests conducted under the direction of Dr. R. Welch at the Fayetteville campus measured the effects of time and percent lime on the unconfined compressive strength on selected samples. Changes of Atterberg limits, pH and bar shrinkage tests of clay soil were determined with the addition of varying amounts of lime.

Conclusions

Based on the results of the soil borings and the laboratory test program, the Railroad Research Team recommended to the railroad that a test section approximately 4000 feet long be injected with lime slurry. A sample set of specifications was recommended to the railroad. Some of the pertinent items recommended for the application were: 1) The soil should be injected to a depth of 14 feet, or until impenetrable material is encountered, 2) The total quantity of lime injected should be between 85 and 105 lbs. per lineal ft., and 3) Injection spacing should not exceed 5 feet between injections.

At the present time, the recommended test program is being considered by the railroad. In the event that the test section is injected, post injection borings will be made, and the treated roadbed will be evaluated.

Bentonite Spur

Belle Fourche, South Dakota

A section of the Chicago and Northwestern Railroad line between Belle Fourche, South Dakota and Colony, Wyoming experienced significant maintenance problems following annual wet seasons. Failure of the subgrade soils was probably repponsible for "sink holes" and difficulty maintaining cross level of the track on the Bentonite Spur. The spur was originally constructed as a "temporary" line to serve the new Bentonite plants, for a life expectancy of about 20 years. After over 25 years of continuous use, the condition of the roadbed has deteriorated to the condition shown in Figure 4. Because of the continued demand to use the spur, it was necessary to repair and upgrade the roadbed without stopping traffic.

In March 1973 soil borings were drilled by Francis-Meador-Gellhaus, Inc. of Rapid City, South Dakota at eight sites on the Bentonite Spur. A profile of typical soil conditions is shown in Figure 5. Twelve borings were drilled through the ballast and underlying clay soils into the weathered shale; the depth of penetration of the borings varied from 8 to 13 feet.[2] A description of the subsurface conditions as reported by Gelhaus follows. The spur is underlain by the Belle Fourche and Mowry shales of Cretaceous age between Colony and Belle Fourche. A layer of Bentonite clay 3 to 8 feet thick was found beneath the Mowry shale and above the Belle Fourche shale. This highly expansive clay is made up mostly of the clay mineral montmorillonite. Apparently, most of the railway embankments were constructed of the weathered shales and Bentonite Clay.

Surface drainage along much of the embankment was reported to be poor. Ponding was evident in cut areas due to sloughing of slope debris and clogging drainage ditches.

A series of laboratory tests were conducted by Gelhaus on the clay samples. The results indicated that lime treatment would be beneficial for some roadbed clays. A significant recommendation of the consultant's report was to improve the surface drainage and to drain the ballast.

In June 1974 the Woodbine Corporation of Fort Worth, Texas injected approximately 12,000 feet of unstable track on the Bentonite Spur with lime slurry (calcium hydroxide). The average depth of injection was approximately 10 feet as measured below the top of the railroad tie. The treated areas varied in extent from 100 feet to 1350 feet long.

A member of the Railroad Research Team returned to the Belle Fourche site to examine the condition of the roadbed and to observe additional lime injection stabilization. From the condition of the track treated in 1974 and the reduced maintenance required, it appears that the stabilization has improved the roadbed. The Railroad Research Team will continue to monitor the performance of the Bentonite Spur. The current plans are to drill post-injection soil borings in the areas treated in 1974.

Bogard to Sumner, Missouri

The right of way between M.P. 118 and M.P. 136 on the Burlington Northern line crosses moderately rolling terrain in Central Missouri. Cut and fill sections are numerous. Cuts of 20 to 30 feet deep exposing shale and limestone bedrock are common. Several fill embankments 15 to 20 feet high were constructed of locally available fill material (probably from adjacent cuts).

Drainage was generally good, however, in some cut areas where the ditches were not maintained, bullrushes were growing in standing water. Vegetation control was good, there being little evidence of weeds in the ballast.

Maintenance difficulties experienced in this area were primarily "soft spots" and several small embankment slides. "Soft spots" were identified by distortion of the track, pumping fine grained soils through the ballast, and movement of ballast around ties. The extent of this problem varied from one rail length to stretches several hundred feet long. There was no obvious correlation of problem areas with cut or fill, or even good and bad drainage.

Unstable embankments where slides occurred were observed at two or three locations. The track and ties were out of level and movement of ballast was apparent.

Corrective Measures

Attempts to stabilize the problem areas during the 20 year life of the line have included pumping Portland cement grout (about 10 years ago) and more recently lime injection. Embankments with unstable shoulders have been cribbed with ties or timber piles driven into the roadbed.

Recently, Roadway Stabilization, Inc. of Eighty Four, Pennsylvania has injected problem areas in the study section with lime slurry. The initial application was made in September 1973. Approximately 289 tons of dry lime were mixed with water and injected into soft or unstable roadbed. The total footage injected between mile posts 128.0 and 130.0 was approximately 5007 feet. The following spot treatment between mile posts 123.0 and 135.5 consisted of injecting 463 tons of lime into approximately 8902 feet or roadbed. This summer the injection work was resumed to treat the additional unstable roadbed. Typical field reports completed by the lime injection contractor's foreman and the railroad engineer's roadmaster are shown on Figures 5 and 6.

A hy-rail wheel inspection of the injection work in this area was made by the writer in June 1975, examining portions of the track treated in 1973 and 1974. A superficial examination of the areas treated 1 to 2 years ago indicates that the treatment was successful at most locations. Although it was not possible to quantitatively compare the performance of the track or roadbed before and after injection, there were few "soft spots" or sections obviously out of line and level.

Permission has been granted by the Burlington Northern Railroad to drill post injection soil borings in the roadbed treated in 1973 and 1974.

Valley Drive

A unique opportunity was afforded the Railroad Research Team to determine the effectiveness of lime injection at a site located in Little Rock, Arkansas. The site, referred to as "Valley Drive", is a portion of a subdivision access road which was stabilized with lime slurry injection in July 1973. Because of economic factors, the area was not fully developed and approximately 1800 feet of the stabilized road was not surfaced.

After permission was obtained from the owners, a drilling and sampling program was initiated to obtain samples of the treated soil in the road and samples from similar but untreated soil adjacent to the road. Although no attempt was made to obtain samples at specific lime injection points, the boring pattern was planned to give complete coverage of the treated access road. The location of the borings is depicted on Figure 8.

Valley Drive is located in the southeast part of Little Rock. Outcroppings of the Midway formation are common in the area. Foundation engineers and earth contractors in Little Rock are familiar with the expansive nature of clay soils found in the Midway formation. Paved areas and building foundations frequently require special treatment.

The purpose of injecting the section of Valley Drive was to economically stabilize the expansive subgrade soil. Approximately 180 tons of hydrated lime was mixed with water and injected in the roadbed to a depth of 7 feet by the Woodbine Corporation.

The east end of Valley Drive is constructed on relatively flat, open land. Soil conditions disclosed in the ten foot deep borings were relatively consistent. A layer of firm to stiff tan or brown silty clay was found at the surface varying in thickness from two to four feet thick. The weathered surface soil was of moderate plasticity and is classified as CL, according to the Unified Soil Classification System. Some coarse sand and gravel was encountered and occasionally white seams of material resembling calcium carbonate were also observed. A typical boring log is shown on Figure 9.

Below 3 or 4 feet depth, stiff gray and red or tan sily clay with well defined blocky and slickensided structure was disclosed to 10 feet depth, the maximum depth of the borings. This highly plastic clay, over consolidated due to weathering, is often referred to as "Midway Clay." The blocky structure of the soil showed a distinct change with depth, i.e. at 5 ft. depth the size of the fractured particles was small, about .125 in. in diameter. Below 7 or 8 feet, the diameter of the blocky particles varied from 0.25 to 0.50 in.

A soil test program was designed to compare the soil shear strength and compressibility of the treated and untreated samples. The University of Arkansas Railroad Research Team is currently performing standard soil classification tests, triaxial compression tests, and swell-consolidation tests on the undisturbed samples. The results of the strength and volume change tests should provide some quantitative information to compare the effects of lime injection on unstable soils.

Conclusions

The behavior of unstable soils treated with lime injection in three different states was observed and evaluated. Although all of the information necessary for a single complete case study was not available at this time, several important conclusions were reached about roadbed behavior.

The roadbed subgrade failures observed were a result of either low initial soil shear strength, or change in shear strength and compressibility due to saturation of the soil with water. Significant changes in soil moisture content in the roadbed appear to be responsible for most of the subgrade failures observed. Poor surface drainage (clogged ditches, flooded areas, etc.) and water perched in the ballast above impervious clays were observed at several sites. Subgrade failures frequently occured after long wet periods in winter or spring in areas where the roadbed becomes saturated and the soil looses shear strength.

The use of injected lime slurry to stabilize "soft spots" and other problem areas in rail roadbeds has had some success. The greatest success appears to be at locations where moisture and saturation have reduced the roadbed strength (Belle Fourche, S. D. and Bogard, Mo.). At these locations changes in the soil properties due to lime injection may have improved the behavior of the ballast/soil structure.

The case study being conducted at Valley Drive should provide data on the behavior of undisturbed natural soil. Based on observations of nearby structures constructed on lime stabilized soil, it is expected that the primary beneficial effect will be to control the volume change potential of the soil.

Other case studies still in progress, but not reported here, will hopefully answer questions about the effect of lime injection on soils with low initial strength and high organic content. Further studies in the area of soil shear strength of lime slurry injected soils are in order.

Acknowledgements

The writer wishes to thank the representatives of Burlington Northern Railway, Chicago and Northwestern Railroad, Chicago, Rock Island and Pacific Railroad, Roadway Stabilization, Inc. and Woodbine Corporation for their help in providing information, site access, and cooperation with the Railroad Research Team of the University of Arkansas.

The research program was conducted under sponsorship of the Federal Railway Administration, Department of Transportation, Contract DOT-OS-40107.

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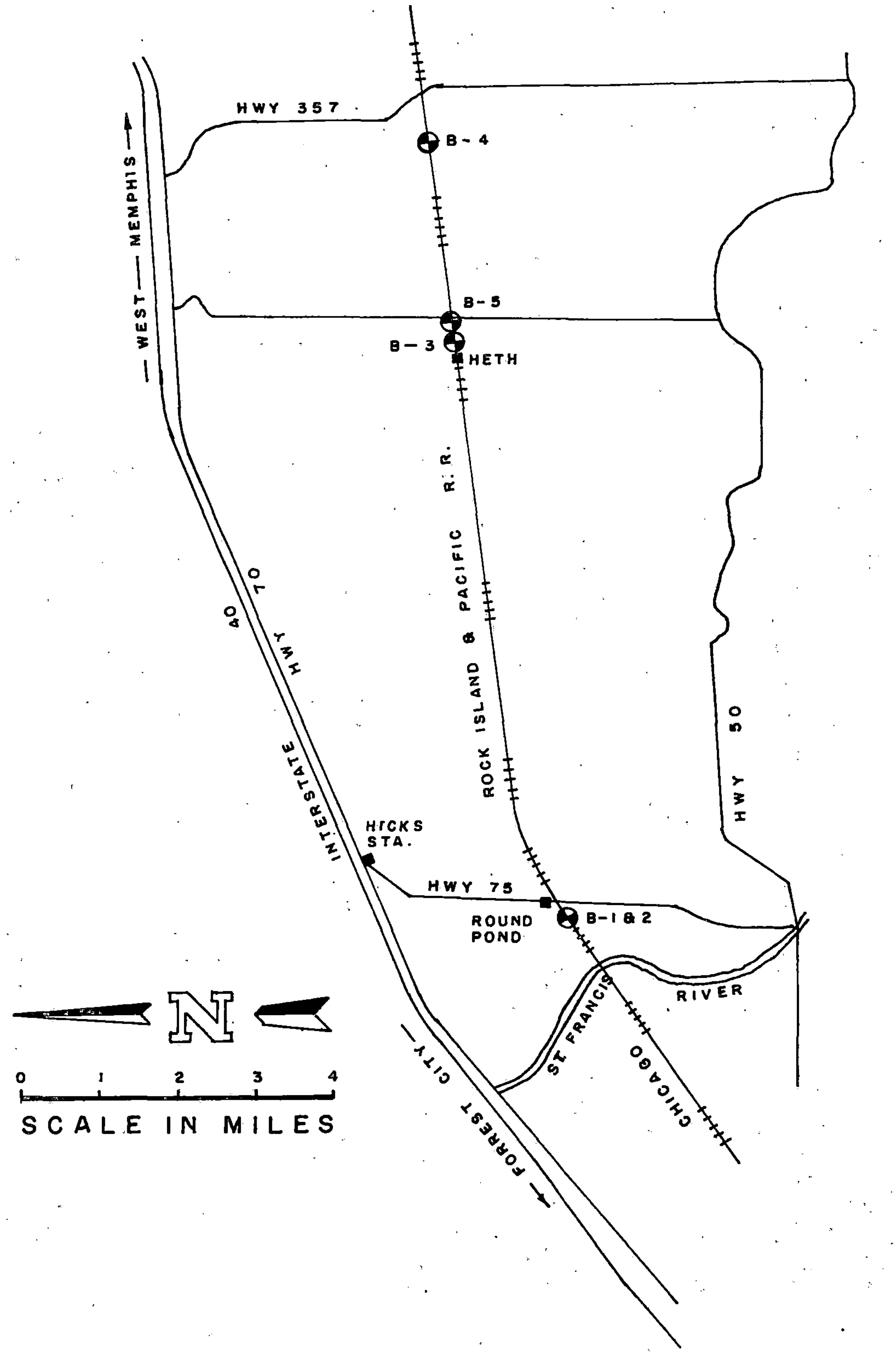


PLAN OF BORINGS

CHICAGO, ROCK ISLAND & PACIFIC RAILROAD
FORREST CITY TO WEST MEMPHIS, ARKANSAS

Figure 1

PROJECT ROCK ISLAND RAILROAD BED, LITTLE ROCK TO MEMPHIS

FOR CHICAGO, ROCK ISLAND AND PACIFIC RAILROAD

CHICAGO, ILLINOIS 60605 ATTN: MR. W. J. CRUSE

ELEVATION - DATE 12-13-74 WEATHER CLEAR & MILD

WATER LEVEL 4.0 AT DRILLING FIELD ENGINEER -

WATER LEVEL 4.0 AT COMPLETION DRILLER SAM BILLINGSLEY

WATER LEVEL - AT 24 HOURS

Sample No.	Sample Depth Feet From	To	Stratum Depth Feet From	To	Soil Classification	N	W	Qu	PPR
			0.0	2.0	Ballast				
			2.0	5.5	SB-2 w/sand				
1	7.0	9.0	5.5	9.0	Gray clay		39		
2	9.0	11.0	9.0	12.0	Gray clay		47		
3	12.0	14.0	12.0	20.0	Gray clay w/layers of		25		
4	14.0	16.0			SB-2		19		
5	17.0	19.0					51		

ALL SYMBOLS AND ABBREVIATIONS USED ARE DESCRIBED IN THE STANDARD LEGEND SHEET

REMARKS: SHELBY TUBE SAMPLE #1 7.0' - 9.0'
SHELBY TUBE SAMPLE #2 9.0' - 11.0'
SHELBY TUBE SAMPLE #3 12.0' - 14.0'
SHELBY TUBE SAMPLE #4 14.0' - 16.0'
SHELBY TUBE SAMPLE #5 17.0' - 19.0'

Figure 2

TEST BORING RECORD(1)

BORING NO. 3 JOB NO. LRB-1581

Track Roadbed at Heth

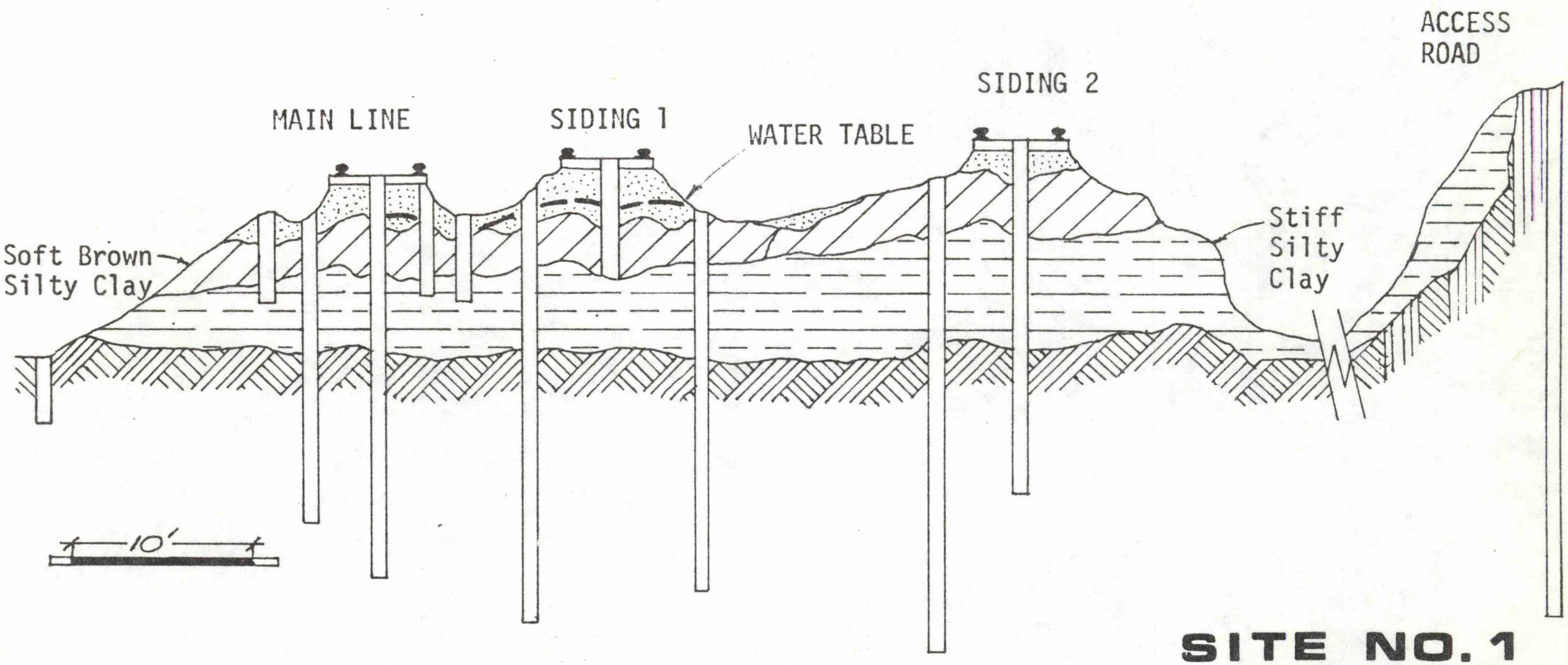
Embankment Near Ark. Highway 149 Crossing

Figure 3

Jordan Ditcher Repairing Roadbed After Injection Near Belle Fourche, S. D.

Bentonite Spur Roadbed Shortly After Injection

Figure 4



TYPICAL SOILS PROFILE[2]

Figure 5

University of Ar sas
P. O. Box 3017
Little Rock, Arkansas 72203

Job # ______
Foreman Jim Rounkles

LIME STABILIZATION CONTRACTOR'S
WEEKLY WORK REPORT
W.E. June 28, 1975

R. R. Name Burlington Northern — Region Chicago
R. R. Division Engineer D. F. Merrill — Location Chicago
R. R. Inspector or Flagman R. M. Evans — Location Brookfield
Job Location: 4th Sub. Division (Between Bogard + Stet — State Missouri

DAY / DATE	MON	TUES	WED	THURS	FRI	SAT	SUN
Temperature Daily (high and low)	70° 87°	87° 70	85° 70°	85° 70°	85° 70°	90° 70°	90° 70°
Precipitation Daily (inches of rainfall)	0	0	1/2	0	0	0	0
Location of Area Worked (mile post, etc.)	144.60 to 144.70	144.70 144.95	144.95 145.05	145.05 145.10	145.10 145.50		
Track Injected (feet)	429	602	468	137	617		
Injection Spacing (A, B or C)*	C	C	C	C	C		
Injection Depth (feet)	14	14'	14	14	14		
Injection Pressure (psi)	90°	90°	90°	90°	90°		
Lime Delivered Per Day (tons)	26	24	24	14	28		
Lime Water Ratio (lbs. per gallon)	3 to 1	3 to 1	3 to 1	3 to 1	3 to 1		
Customer Delays (hours)	1	1 1/4	1 1/2	7	1 1/2		
On Track Work Time (hours)	6	8 3/4	8 1/2	3	8 1/2		
Total Charged to R. R. (hours)	7	10	10	10	10		
Site Description (cut, fill, level, etc.)	level	fill	fill	cut	cut		
Soil Description (general terms)							

Lime Supplier and Location Linwood Stone Product Davenport Iowa
Type of Surfactant ______ Ratio ______
Equipment Data - Contractor's Injector Unit Number 5 Haul Truck Unit Number 501
Method of Mixing Lime and Water Blender - Pump Agitator
Any Unusual Conditions ______

*A. Every Tie
B. Every 2nd Tie
C. Every 3rd Tie

Jim Rounkles
Signature, Roadway Stabilization

Figure 6

P. O. Box 30?
Little Rock, Arkansas 72203

Date June 28, 1975

LIME STABILIZATION RESEARCH REPORT

WEEKLY WORK REPORT
W.E. June 28, 1975

R. R. Name Burlington Northern Division Hannibal
Job Location: Needles to Maxwell State Missouri
Contractor's Name Roadway Stabilization Foreman J. Rowland for R. S.
R. M. Evans for B. N
Location of Area Worked
(mile post, etc.) MP144.64 to MP 145.23

Why was this particular track area selected for LSPI? Unstable Roadbed

Subgrade soil classification, type or description. (Use standard classification nomenclature, i.e. Unified, ASSHO, etc.) Standard classification Unknown

Yearly gross tons on this track 1972 7,643,871, 1973 8,827,499
Heaviest monthly traffic in tons ______ Month? ______
Weight of Rail 112#, ~~welded or~~ bolted, ballast type? Slag
Maximum Time Card Speed Limit of this track? 60
No orders on now - previous orders of 25 and 35 MPH had been on this location
Slow orders in effect before injection 0 after injection 0
Type of maintenance work performed past three months? (M.P. to M.P.) 144.64-145.23
Alignment and surface correction by section gangs.
Estimated Man Hours 190
Type of maintenance work performed past year? (M.P. to M.P.) Surface correction unit with tamper and liner

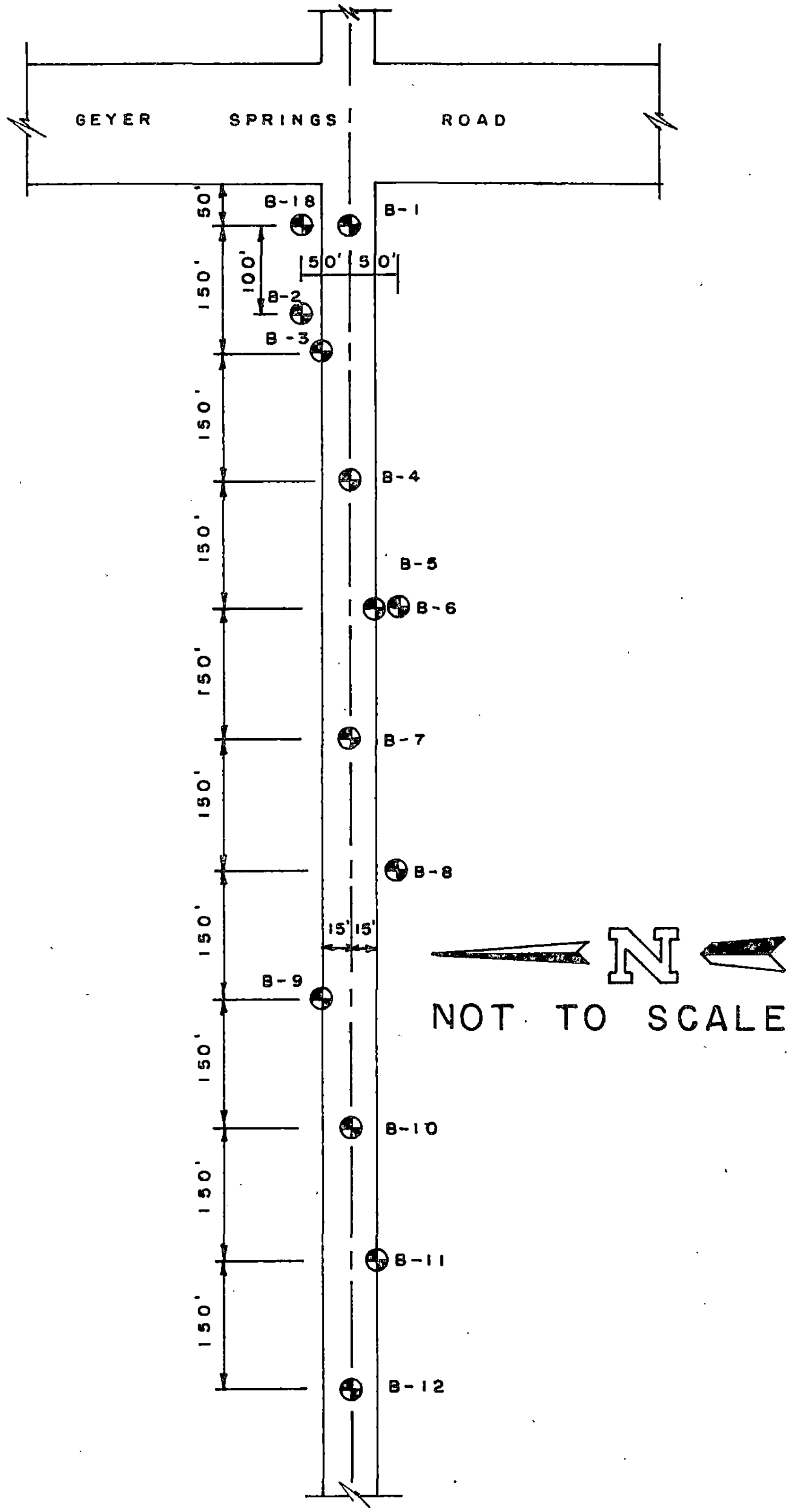
Estimated Man Hours 430
Grouting or stabilization history of this track area None

Will track be reworked after injection As needed New Track? no
Reballasted? no Resurfaced? no
Any Unusual Conditions: None

Address and Phone No. of R. R. Engineer
Brookfield, Mo
314-221-1480 Ext 56

H. E. Jackson
Signature, R. R. Engineer

Figure 7



PLAN OF BORINGS

VALLEY DRIVE STUDY – LITTLE ROCK, ARKANSAS

Figure 8

LOG OF BORING NO. 9

VALLEY DRIVE
LITTLE ROCK, ARKANSAS

TYPE: Auger

LOCATION:

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL:	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT: 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT: 10 20 30 40 50 60 70	% RECOVERY
0–10			Stiff tan silty clay with some coarse sand and occasional gravel -gray and red with less coarse sand and occasional white pockets -blocky and slickensided below 4 ft -with reddish tan partings and no white below 8 ft				

COMPLETION DEPTH: 10 ft
DATE: 6-13-75

DEPTH TO WATER IN BORING:

DATE:

Figure 9

EXPLORATION AND TESTING

FOR LIME INJECTION PROJECTS

By

Robert C. Welch
Associate Professor of Civil Engineering
University of Arkansas
Fayetteville, Arkansas

ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

Little Rock, Arkansas
August 21 and 22, 1975

EXPLORATION AND TESTING FOR LIME INJECTION PROJECTS

The current cost for lime injection treatment of roadbeds is approximately $20,000 per mile. Before spending this kind of money, we would like to have two questions answered: (1) Will lime slurry pressure injection (LSPI) improve the stability of the roadbed? (2) How much improvement can be expected? The first question can usually be answered, but we do not now have the means to quantitatively assess the improvement brought about by LSPI. The exploration and tests useful in answering the first question are described herein. A series of instrumented test sections are planned to obtain the answer to the second question.

LSPI will improve the stability of roadbeds if: (1) Lime can be distributed throughout the soil mass and (2) the soil is lime reactive. The stability is improved by reducing volume changes occurring with seasonal moisture changes and by increasing the shear strength. Calcium ions are adsorbed on the surface of the clay particles, satisfying the negative charges, and reducing the affinity of the clay for water. If a clay lump is encapsulated by a layer of lime, the barrier created by the lime treated zone reduces seasonal changes in moisture content and thus reduces swelling or shrinking. The water in the slurry also provides some beneficial effect with respect to volume change, especially if the soil is dry. Experience has shown that shrinkage upon drying to the equilibrium moisture is much more uniform than swelling. The beneficial effects due to wetting the soil will be small, however, if the natural moisture content is greater than the plastic limit. The shear strength increase is due to a pozzolanic reaction taking place between the lime and the soil. The pozzolanic reaction is time and temperature dependent, but the adsorption of calcium ions occurs very quickly. The membrane effect created by the strengthened layer significantly improves the stability of the soil mass.

The distribution of pressure-injected lime slurry throughout a soil mass is usually dependent upon a pre-existing network of cracks, seams, fissures, slickensides, etc. Compaction planes sometimes provide a path for the lime slurry. The best means to determine whether or not this type of soil structure exists is by visual examination of an undisturbed soil sample. If the structure is not readily apparent, observation during air drying or dropping the sample on the floor or other smooth hard surface will aid in identification.

Lime reactivity tests will fall into two categories. The first category is measurement of the effects of the adsorption of cations on the surface of clay particles. Probably the best indication of this effect is the reduction of plasticity. Liquid limit and plastic limit tests are performed on the natural soil and on soil containing lime. A reduction in plasticity index indicates lime reactivity. Swell tests or the potential volume change (PVC) test developed by the Federal Housing Authority are also useful.

The second category of tests is the measurement of pozzolanic activity. The pozzolanic reaction is time and temperature dependent, and results in a cementation of the soil mass. The increase in strength due to this cementation is usually measured by unconfined compression tests for intimate mixes of soil and lime. A convenient and easy procedure to determine the relative magnitude of the strength increase is the use of 1.4 in. diameter specimens molded in the Harvard miniature apparatus. Specimens at various lime contents could be cured at room temperature (70° F) for about 15 days or at an elevated temperature (not higher than 140° F) for a shorter period of time. Other tests, such as the cohesiometer test, indirect tensile test, etc. could be used. Remember that we are simply trying to find out if a pozzolanic reaction will occur and the relative strength of that reaction. We cannot use the values obtained in design or analysis.

The Eades and Grim quick test is useful in determining the optimum percent of lime to use for both the plasticity tests and the compressive strength tests. The optimum lime percentage is that which gives a pH of 12.4 after one hour of curing in slurry form.

To summarize, the following sequence of tests would give a reasonably good evaluation of whether or not LSPI would improve a roadbed.

1. Determine natural moisture content and density.

2. Examine structure of soil for seams, fissures, slickensides, etc.

3. Perform grain-size analysis to find percentages of sand, silt, and clay.

4. Use Eades-Grim quick test to find optimum lime content.

5. Find liquid limit and plastic limit for natural soil and soil with optimum lime content.

6. Test unconfined compression specimens of natural soil and soil with optimum lime content. Specimens require curing.

The following comments are made to aid in evaluating the test results.

Evaluation of Test Results

1. An expansive clay soil (one which shrinks and swells with moisture change) can be identified from the results of the liquid limit and plastic limit tests. The difference between the liquid limit and the plastic limit is the plasticity index (PI). The expansive properties of clays generally correlate with the PI as shown below.

PI	Expansive Properties
less than 15	Non-expansive
15 - 25	Slightly Expansive
25 - 35	Expansive
over 35	Highly Expansive

2. If an expansive soil is present and exhibits a moisture content less than the plastic limit, a significant volume change may occur after injection due to the water in the lime slurry. This is not undesirable, since the adsorption of calcium ions will tend to prevent further moisture change. If further change were to occur, it is better to be on the wet side because shrinkage occurs much more uniformly than swelling.

3. If the soil contains seams, fissures, slickensides or similar structural features, the chances of distribution of lime slurry throughout the soil mass are reasonably good. If these features are not present, it does not mean that lime injection will not work, but the probability of success is certainly less.

4. In order for the pozzolanic reaction to significantly increase strength, a clay content greater than 10% is usually required. Some silts, however, will react with lime. Sands almost never show any significant pozzolanic reaction.

5. The lime content given by the Eades and Grim quick test should be used only for laboratory analyses or for intimate mixes of lime and soil. It should not be used to determine the amount of lime to be injected.

6. By comparing the plasticity index of the soil-lime mixture with that of the natural soil, the effect of calcium ion adsorption can be seen. If the PI is reduced to below 15, then a beneficial effect upon volume change properties can be anticipated. Many clay-lime mixtures exhibit non-plastic behavior.

7. Pozzolanic cementation, measured by compression tests, is time and temperature dependent. Accelerated curing, say 3 days at 140^{o} F with a relative humidity of 100%, can be used. Caution should be used with accelerated cures because high temperatures may cause reactions which will not occur at ambient temperatures. Curing at room temperature (70^{o} F) for say, 15 days, is recommended until experience shows similar effects take place under both ambient and elevated temperatures.

8. If a pozzolanic reaction does not take place between the lime and soil, an alternative is to inject a lime-pozzolan mixture. Fly ash (from the burning of coal) is a readily available pozzolan.

If the soil structure will admit lime and the tests show a significant reduction of plasticity and an increase in strength, then the chances of improvement of the roadbed by LSPI are good.

Some comments on the identification and investigation of problem areas seem in order. Existing problem areas are obviously well known to the maintenance staff. It is desirable, however, to be able to locate potential problem areas so that preventive maintenance or at least close observation can be done. Soil maps or geologic maps are a valuable source of preliminary information regarding soil types. After preliminary identification of problem areas from maps or other sources, confirmation should be obtained by sampling suspect areas and performing laboratory tests. A boring spacing of approximately 600 ft is suggested with intermediate borings as necessary to define the limits of the problem area. Depth of the borings should be about 20 ft and undisturbed samples of the soil should be obtained.

If existing and potential problem areas are identified, tested and treated, significant savings in both maintenance and operating costs will accrue. In this presentation, a brief description of the exploration and testing required to evaluate the applicability of the LSPI method of treatment have been given.

LIME INJECTION PRODUCTION EQUIPMENT
AND TECHNIQUES

By

Paul J. Wright
Vice President
Woodbine Corporation
Fort Worth, Texas

ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS
AND
THE FEDERAL RAILROAD ADMINISTRATION

August 21 and 22, 1975
Little Rock, Arkansas

INTRODUCTION

Historically the greatest portion of railroad maintenance dollars has been spent on top of the roadbed - for rail, ties and ballast and all the related maintenance functions of these components. Keeping the track in surface has usually been a function of adding more ballast, tamping and occasionally cribbing out the fouled ballast when it becomes filled with clay that has pumped up from below.

Excess moisture is the primary cause of subgrade instability. Everyone in the railroad business recognizes the importance of good drainage but you don't have to walk out much track to realize that improper drainage is, in fact, a reality with many railroads. Drainage is many times lost by plugged up culverts or land work done by adjoining property owners. Side ditches are easily blocked by squeezes and slides and keeping them open and properly drained is a major problem because there are so many areas where these shear failures occur along the roadway. In many areas good drainage is not possible because there is no place for the water to go.

Also, I think that everyone realizes that a lot of the maintenance done in these problem areas is on a fire fighting basis and usually before the complete job can be done there is another fire someplace else.

LIME SLURRY PRESSURE INJECTION - A VIABLE APPROACH TO SUBGRADE STABILITY

Our approach to subgrade instability is through Lime Slurry Pressure Injection (LSPI). Our contention is that when the subgrade is bad - any surficial work that is done is merely buying time. Other techniques that have been used by railroads such as cement grouting and driving poles have not produced permanent results. Most of the areas we have lime injected had previously been driven or grouted or both.

Although there are many aspects of lime injection that are not yet proven, particularly with regard to the actual mechanics of what is taking place - I think we can safely list these five items as the principal benefits to be expected from LSPI.

1. Dewatering: We know from experience on many jobs that lime actually cuts off the flow of subsurface water.

2. Uniformity of moisture content: The principal benefit in many instances is in stabilizing the moisture content of the soil. As the lime flows through the soil it forms impervious moisture barriers which tend to impede the movement of moisture through the soil.

3. Less seasonal effect: As a result of stabilizing the moisture content of the soil there is less degradation from seasonal moisture. Long dry spells or long wet spells do not have the same devastating results on the roadway.

4. Reduces swelling: We now have substantial test data to know that swell is reduced even in the clays between the lime seams and not just at the interface.

5. Increase strength: Tests have also shown that there is a corresponding increase in strength and as the moisture content of the soil is stabilized there is no longer the substantial loss of strength due to saturation.

We have learned more about LSPI in the last year than in all the previous six years combined. Most of our early efforts involved developing and improving equipment and techniques whereas we are now more involved with private as well as funded research, development of new methods of testing, new applications, as well as the use of different materials and additives.

WOODBINE CORPORATION - SPECIALISTS IN SOIL PROBLEMS

Before starting my slide presentation I would like to briefly acquaint you with our company. Woodbine Corporation was founded in 1968 by Joe D. Teague, Gene Cain and myself for the sole purpose of engaging in various methods of soil stabilization including the then very new technique of LSPI. Since that time we have stabilized many million square feet of soil. Present applications for our system include building sites, streets, runways, railroad beds, levees, dewatering projects and releveling projects using pozzolanic grouts. We are also engaged in conventional lime stabilization and cement grouting. We consider ourselves as specialists in soil related structural problems. Woodbine Corporation has participated in several federally funded research projects, including:

1. Robnett, Q.L., Jamison, G.F., and Thompson, M.R., "Technical Data Base for Stabilization of Deep Soil Layers," Technical Report, No. AFWL-TR-70-84, Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico, April, 1971.

2. Improvement of Problem Track Subsoil by the LSPI Method, University of Arkansas, Dr. James R. Blacklock, DOT, FRA, Office of R & D.

3. HUD Project, Dr. Arthur Poor, University of Texas at Arlington, Arlington, Texas.

We have sponsored several privately funded projects to expand our data base including two in Little Rock, Arkansas. We have a continuing program of participation with several engineers to further develop testing techniques for LSPI. Pre and post injection test data is being stored in a computer bank in Little Rock, by Systems Research, Inc. and includes data from projects in Texas, Oklahoma, Louisiana, Arkansas and Mississippi.

DEVELOPMENT OF LSPI APPLICATIONS

The first application of LSPI was remedial stabilization of existing structures which had suffered foundation distress. This goes back about 13 years ago in Louisiana where the first work was done by hand methods. The techniques as well as equipment were very crude compared to today's standards. The theory of this type of injection is to form a seal or barrier around the perimeter of an existing structure to allow the soil below the structure to achieve a uniform moisture content. This same basic concept is still used today although there have been considerable improvements.

Some of the soils engineers in the D/FW area began to specify LSPI for building sites in an effort to reduce the volume change potential of expansive clay soils.

Some of our first efforts involved using a pre-drilled hole, sealing it at the surface with a packer and injecting the hole under pressure with lime slurry.

We later began to develop injection rigs that hydraulically forced the injection pipes into the ground without requiring a pre-drilled hole. These machines were much more efficient and resulted in better quality and more production at a lower cost.

Today the same basic injection rigs are used for remedial stabilization of buildings. These machines produce a much better job by taking the injections deeper and getting more material in the ground.

In August, 1971, we contracted with FRISCO to install some test sections in North Texas, using LSPI stabilization. To our knowledge this was the first railroad injection work that utilized machine equipment and hydrated lime. Some limited work had been done in Louisiana using manual (hand) injections and by-product (waste) lime and the results were not satisfactory. The use of by-product or waste lime, particularly for injection, is a total waste of time and money.

This first railroad work was completed with our regular rubber tired forklifts. The logistics proved to be very bad for railroad work and led to the development of our present high-rail equipment.

FRISCO reported after about one year of observing the test sections that their maintenance had been reduced by 75% on all the track we stabilized except for some deep ballast pockets. In these pockets our 10' deep injections did not even penetrate through the ballast into the clay. That first contract with FRISCO was followed up with several more in the North Texas area.

When we're talking about results of the lime injection work that has been completed to date it is important to remember that we have only spot treated the very worst high maintenance areas of track. We generally walk out the sections of track with an engineer and the roadmaster and make a list of the locations to be stabilized.

As a rule several soil samples are taken so that lime compatibility tests can be run to determine that the soil does react with lime. We have used several techniques but the one we favor the most is to remold the samples, then after curing they are soaked in water for 24 hours and tested with a pocket penetrometer. Comparison of the control sample with the lime sample gives us an idea of how reactive the soil is with hydrated lime. This in no way is intended to replace the need of consulting with independent soil testing laboratories but is done as a safeguard so we will not contract for work where injection would not be appropriate.

WOODBINE CORPORATION'S PRESENT SYSTEM OF RAILROAD STABILIZATION

One or two bulk mixing tanks are placed on the job as close as possible to the area to be injected. The mixing tanks should be located near water and on an all-weather road surface so that bulk lime deliveries can be made regardless of weather.

Our mixing tanks are 10' x 30'. This size allows us to mix an entire 20 ton load of lime at one time, assuring a uniform mixture each load. All of the slurry tanks are equipped with mechanical agitators to keep the lime in suspension.

In some instances, the bulk slurry tank can be positioned next to the track so that slurry can be pumped directly to the on-track slurry haul truck.

When the bulk slurry mixing tanks have to be located away from the track, a 4,000 gallon transport is used to deliver the slurry to the on-track equipment. Slurry is transferred from the transport to the on-track haul truck which then delivers the slurry to the injection truck.

The injection truck is a self contained unit equipped with a 2,000 gallon slurry tank, high pressure pump, engine and three injectors capable of making 10' deep penetrations. When the logistics of this sequence are carefully planned, the daily production will usually be 500-600 track feet per day with one mixing tank, or 600-800 track feet per day with two mixing tanks.

The average daily production is also influenced by the amount of rail traffic and how much actual on-track time is allowed the contractor. Because of the great variable in traffic, most of this work is generally contracted for on an hourly rate basis rather than by the track foot.

RAILROAD STABILIZATION CREW

A railroad injection crew normally consists of four men, although some jobs can be adequately manned with three people.

The lead man is well experienced in lime injection and has been trained to look for unusual problems or signs that indicate something is wrong or something additional needs to be done. The lead man is responsible for customer coordination, ordering lime, accepting deliveries, keeping all field records and submitting the necessary job reports.

One or two men handle the slurry mixing and hauling from the bulk mixing tank to the injection truck.

One operator is required for the injection rig. From his seat at the rear of the truck he can see all the area around each injector and as each injection is complete he can advance the truck for the next injection.

INJECTION PROCEDURES

The injection points are machined parts that are attached to the end of the heavy wall, heat treated, 1 5/8" OD injection pipe. These points contain a hole pattern that distributes the slurry in a 360 degree radius from the injection hole.

The injectors are pushed into the ground hydraulically, usually to a depth of 10'. Studies have shown that the zone of significant season moisture change is about seven feet below the surface. We believe that 10' is appropriate for most track work with the exception of high fills which may require additional depth. We presently have a new injection system in work which will allow depths of 20' or greater. We intend to apply for patents on this system which should be available for work in two to three months.

The injectors are spaced on 5' centers. One is positioned in the center of the rails and one five foot out on each side. Injections are made every five feet, although in some areas where the subgrade is extra bad it is recommended that every other crib be injected instead of every third crib.

Injections are made incrementally on the way down, injecting to refusal at each increment. In most soils an 18"-24" interval is required between each level of injection in order to get a seal and be able to pump more slurry into the soil.

HOW LIME EFFECTS THE SUBGRADE

As previously mentioned, the chief culprit of most track subgrade instability is water. Excess water in the subgrade usually manifests itself in problems we call soft track, pumping, squeezes, slides, slope failures, and out of face track.

Water penetrates the subgrade from surface rains, by capillary rise from below and from lateral movement, expecially in areas of side hill cuts. Water is easily trapped at grade changes and improper drainage results in excessive soaking of the subgrade.

Conversely in long dry spells the elevated roadbed is vulnerable to excessive drying. The resulting surficial cracks then make the roadbed especially vulnerable to rapid soaking from heavy rains and the immediate result can be a derailment.

Probably the chief benefit of LSPI is in stabilizing the moisture content of the soil. Although we are not quite sure of all the chemistry that takes place when we pump lime slurry into the soil, we do know that in heavy clay soils the lime flows through the available fractures in the soil forming a network of lime seams. These lime seams become impervious moisture barriers that impede the movement of moisture through the soil. These thin lime seams actually thicken or grow by calcium ion exchange to a total thickness of about 3" over a period of one year. These lime seams add strength to the fractured areas of the soil mass and keep more water from getting into the subgrade. Many times during injection, clear water is observed flowing out of the roadbed at the bottom of the fill as it is replaced by the lime surry.

Until about six months ago we thought these lime seams plus pre-swelling the clays were the principal and perhaps the only benefit to be derived from lime injection. Recent tests that have been completed on many of our jobs offer proof that the soil between the lime seams is also affected. Swell is reduced and strength is increased. This is a statement that couldn't have been made a few months ago.

We knew that we were pre-swelling the clays when injecting a building site for example, and tests would show that moistures increased on an average of about 2-3% with one lime injection. These moisture increases were recorded on the soil between the lime seams and not at the interface. Even though the lime particles themselves can't penetrate the heavy clays, the water in the slurry apparently contains enough calcium ions to physically affect the soil. Tests show that the PH of the slurry supernatant is from about 11.9 to 12.3. We think the water moves into the clays when dry by unsaturated flow (suction), and in the saturated clays by dispersion.

The top line in Table 1 shows the amount of free swell in this soil in its natural (untreated) state. The bottom line shows the reduction in swell by treating it in the lab with .5% hydrated lime. This is a reduction in swell of approximately 2 1/2 times. The middle line shows how much the swell was reduced by using only the water (supernatant) from the lime slurry.

The soil test samples are injected with a hypodermic needle to try and simulate the field injection. This technique was developed by John Woodard, manager of Barrow-Agee Labs in Little Rock, Arkansas. After injection they are then tested either for swell, strength or consolidation.

After achieving these results in the laboratory we were then interested in determining how these would compare with actual field samples following injection. Table 2 shows that the predicted differential movement for a single injection was 0.215 feet and the actual movement as measured in the field 0.24 feet for Bldg. #1 and 0.102 feet predicted compared to 0.10 feet actual for Bldg. #2. These tests were just completed on a project in Oklahoma and it is evident on this project that the lab tests correlate very close with the actual field results. Although we do not yet have a tremendous amount of data it does appear that these laboratory tests can be used to predict field results. Also you will notice on Table 2 where a second injection was made on this project. Based on some very recent tests, it is our opinion that some building sites have to be injected a second or maybe even a third time in order to physically get enough lime into the ground to affect the desired engineering changes. Heretofore, if a site was injected once and there was unsatisfactory movement to the structure it was considered that lime injection failed or apparently just doesn't work. We have now realized that due to the relatively small amount of lime slurry that goes into the ground (approximately .5% to 1%, compared to 5% to 6% usually required for conventional lime stabilization), some sites simply have to be injected more than once. We have also been able to attach some significance to staging the second injection, allowing at least 48 hours after the initial injection. In my opinion the use of multiple injections on some sites is one of the most significant breakthroughs we have made.

I would suggest here today that the same thing may likely be true of some sections of track. There will be certain sections, perhaps, where conditions are bad enough to require additional injections. Hopefully the test procedures we are working on now will eventually allow us to predict in advance which sections of track may require multiple injections.

A VISUAL FIELD TEST

In addition to lab tests we just completed a field test which provides an excellent method of visually determining flow or movement of the slurry through the soil. On this test section it was necessary to drill through 6" of soil cement and 6" of lime stabilized material which is topped with about 16" of ballast. The track which is relatively new has failed badly. Both stabilized layers are cracked and mud is pumping up into the ballast.

In order to install this test section it was necessary to drill through the 12" of stabilized material.

A tie was removed and using a backhoe, a 5' deep trench was excavated across the track. One trench was excavated across a section that had already been injected and one in a section not yet injected.

In the trench that was in the area already injected, it was necessary to use phenolphthalein to locate the various seams where the lime had flowed through the soil. Although the soil mass was a fill, it was a very heavy, uniform looking clay with few visible fractures and it appeared saturated.

When the injection truck moved to within 7 1/2' of the other excavation, lime slurry began to flow into the trench, first at the interface of the stabilized layer and then at various places all up and down the wall of the trench.

SUMMARY

Lime Slurry Pressure Injection is a relatively new technique not yet fully proven to everyone's satisfaction. To many who hesitate to deviate from the old ways - its acceptance may be very slow - or never. Others have realized that if these recurring high maintenance problems are ever to be dealt with successfully it is mandatory to try new ideas and techniques that appear to offer promise. The railroad industry is in a fortunate position to have the benefit of a system that has thirteen years of growth and improvement, and contractors who have been willing as entrepreneurs to develop and fabricate equipment to do this work even though the railroads have not yet committed themselves to much more than what I would call extended test sections. Before closing I would like to offer a word of advice or caution to the railroads relative to the selection of a contractor to perform LSPI stabilization. When you are buying a service where the end result of the job is not visible to the eye it becomes very important that you know something about the capabilities and experience of that contractor.

I would recommend that you check the potential contractor's references. His past performance in railroad work - can he provide legitimate pre and post injection data to substantiate what he can do? - What about his equipment and techniques - did he recommend any type of testing to you?

These are a few of the things I would encourage you to find out before you let a contract - because there are no standards to meet except those imposed by the railroads in their selection and qualification process.

Lime Slurry Pressure Injection is not a panacea for all your track problems. But for many of them it is the perfect answer. An economically feasible, permanent method to improve subgrade stability with a minimum disturbance to rail traffic. For railroads, Lime Slurry Pressure Injection stabilization is an idea whose time has come.

Ⓘ CONT
△ LIME
+ SUPR

TABLE 1

SYSTEMS RESEARCH, INC
LITTLE ROCK ARKANSAS

J1287 60675

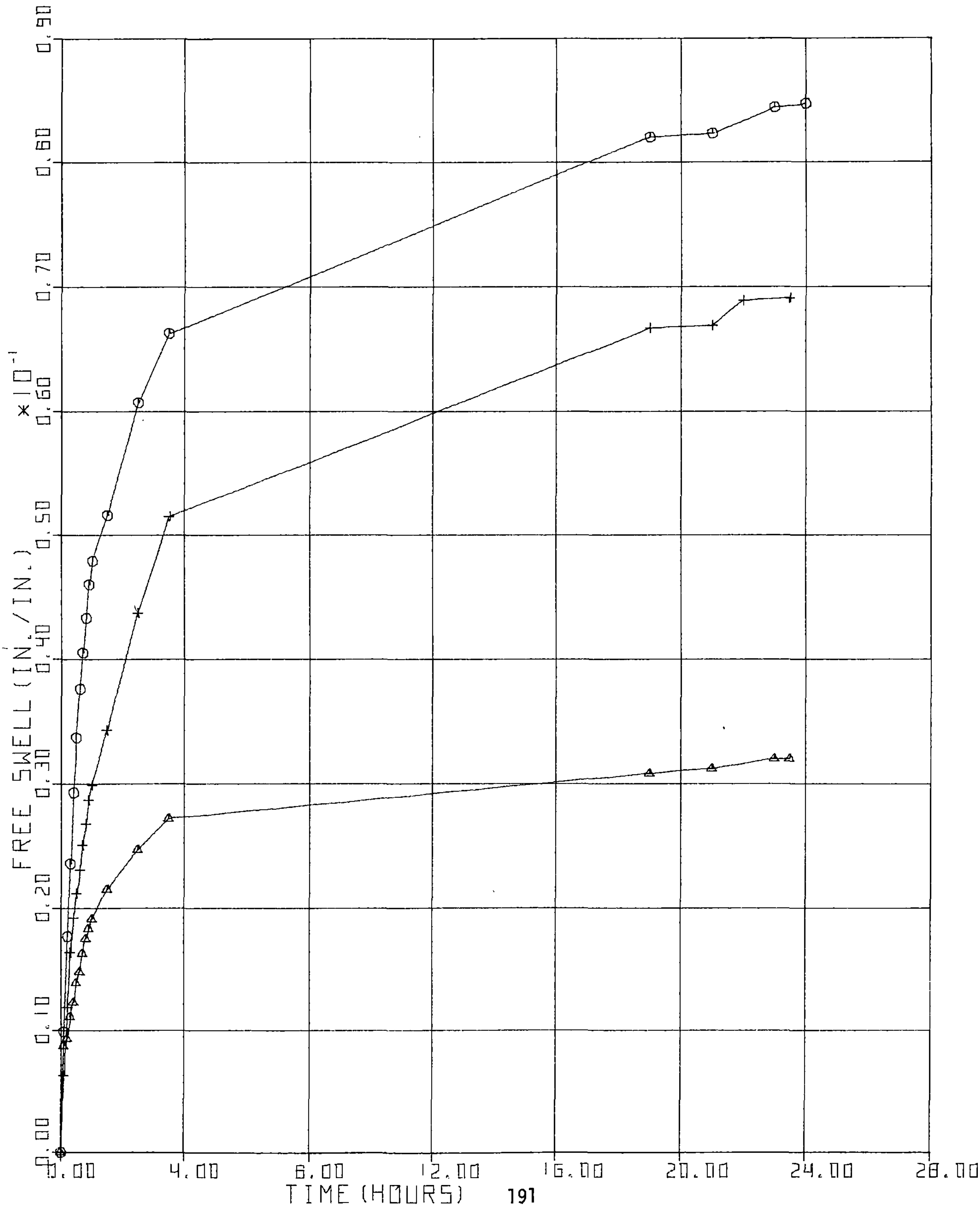


TABLE 2

mcnutt - schneller, inc.

engineers & planners

little rock, arkansas

DIFFERENTIAL MOVEMENTS

	95%≤ COMPUTED	*N EQUALIZED	MEASURED
BUILDING #1			
FIRST INJECTION:			0.24'
NATURAL INITIAL TESTS	0.230'	0.230'	
NATURAL PREINJECTION TESTS		ANOMALY	
SINGLE INJECTION INITIAL TESTS	0.215'	0.215'	
SECOND INJECTION			0.07'
NATURAL INITIAL TESTS	0.080'	0.080'	
DOUBLE INJECTION INITIAL TESTS	0.054'	0.054'	
POST INJECTION TESTS	0.084'	0.068'	
BUILDING #9			
FIRST INJECTION:			0.10'
NATURAL INITIAL TESTS	0.109'	0.109'	
NATURAL PREINJECTION TESTS	0.155'	0.107'	
SINGLE INJECTION INITIAL TESTS	0.102'	0.102'	
SECOND INJECTION:			0.04'
NATURAL INITIAL TESTS	0.082'	0.082'	
NATURAL PREINJECTION TESTS	0.117'	0.081'	
DOUBLE INJECTION INITIAL TESTS	0.055'	0.055'	
POST INJECTION TESTS	0.068'	0.047'	

* FOR COMPARISON ONLY

Luncheon Speech

LIME OUTLOOK

Robert S. Boynton, Executive Director
National Lime Association, Washington, D.C.

Lime Supply and Requirements

The lime industry has just weathered a strange situation: For nearly two years - up to four months ago - there has been at least off and on, a lime shortage or very tight supply situation. Usually the reverse is true - an abundance of lime, an over-capacity problem and highly competitive conditions.

During much of 1973 and '74 shortages were almost as widespread as during World War II - slow deliveries on steel and 1 to 2-year delays in delivery of fabricated steel capital equipment; most non-ferrous metals, like copper, aluminum were in very short supply; paper; most heavy industrial chemicals, including lime; etc.

While a buyer's market for lime returned in April, 1975, aided by the current recession, basic factors still exist that could impede most industries, including lime.

Factors Causing Shortfall of Lime

There were quite a few diverse reasons for the lime shortage.

1. EPA Air Pollution Standards - Lime plants were compelled by the EPA and states to clean up their particulate emissions and install the most advanced air pollution control systems, i.e., baghouses, electrostatic precipitators, high energy wet scrubbers, etc. even though dust from lime plants is not remotely toxic, only a nuisance dust. The installation cost of the capitol equipment and cost of operation and maintenance is so high that some old, marginal plants preferred to shut down permanently rather than to retrofit pollution controls to their old obsolete plants. 12 plants were closed. In some cases, the cost of pollution control systems was more than double the value of their old, fully depreciated plants. However, 6-7% of U.S. lime production capacity was lost.

2. High Lime Demand - Both 1973 and 1974 set all-time records in both lime production and shipments. Increased demand came from a few well-established markets, such as oxygen steel manufacture, soils stabilization and potable water treatment. Also new developing environmental markets in pollution abatement increased for treating sewage wastewater, acid mine discharges, and desulfurizing stack gases from coal-burning plants.

3. Other Problems that caused losses in production were strikes; periodic shortages of fuel (gas curtailments, coal strikes, etc.); shortage of rail cars to ship lime; floods; curtailed lime production due to air pollution episodes; and zoning problems affecting land use; etc.

However, in spite of this, new lime plant facilities have come on stream during the past six months, strengthening the lime industry's ability to supply future demand. There is also no shortage of the raw material - limestone - needed to make lime, at least, in the forseeable future.

Fuel Situation

Unquestionably the No. 1 problem for lime for many years ahead is fuel and energy for lime kilns - scarcity of fuel, quality of fuel and soaring costs.

Lime is one of the most fuel-intensive industries, consuming an average of 8 million Btu to make a ton of lime. Gas, coal and oil prices have soared 200 to 500% -- and some lime producers will have to absorb further increases. Thus, if gas rates are \$1.50/Mcf, 8 x \$1.50 = \$12/ton just for energy in a ton of lime. The concensus of opinion is that fuel costs will soon level out on an average of \$2.00/million Btu's, making the energy factor in a ton of lime - \$16/ton. As a result, lime prices have risen more in the past 2 1/2 years than in the preceding 25 years combined.

Lime Statistics

Lime production in 1974 was about a record 21 1/2 million tons, a fraction less than 1/5 the size of portland cement in tonnage. It is still larger than any other chemicals, except sulfuric acid. Of this lime tonnage, 7 million tons are captively produced and 14 1/2 million are commercial lime (shipments). Of this latter commercial ton figure, the following are the principal lime use categories by approximate percents:

Use	%
Steel manufacture (as flux)	45
Non-Ferrous metals	10
Potable Water Treatment	10
Pulp and Paper	7
Soils Stabilization	7
Refractories	7
Chemicals manufacture	5
Air and Water pollution control	4
Building lime (mortar, plaster)	3
Glass and miscellaneous uses	3

Possibly no other materials have as many or as diversified uses as lime.

Conclusion

The lime industry is most intrigued with the development of lime pressure injection and feel it has a bright future if it is not over-promoted and is applied based on a rational construction specification. This conference has been a "shot in the arm" to this construction process and should help to stabilize and further the development of pressure injection. Since no other stabilization use for lime is as demanding as its use in pressure injection, I urge engineers to specify and contractors to use only a good quality of virgin lime. Although waste limes have been used with some success in conventional soils stabilization applications, it is too risky to use for pressure injection. Bad failures have even occurred in conventional stabilization work. Remember there is no uniformity in quality of waste lime.

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ANALYSIS OF LIME INJECTED ROADBEDS

By

James R. Blacklock
Program Director of Railroad Research
Graduate Institute of Technology
Little Rock, Arkansas

ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS

AND

THE FEDERAL RAILROAD ADMINISTRATION

August 21 and 22, 1975
Little Rock, Arkansas

SUMMARY

The accurate determination of the stresses, strains, deformations and modes of failure of the train-track structural system is a difficult task for the railroad civil engineer. The new concepts of static and dynamic repeated load spectrums, load history dependent soil material parameters and modern structural analysis techniques are necessary for a proper solution to this complex problem. The railroad track system, composed of soil, ballast, tie, joint and rail poses a design and analysis problem that is unique to the railroad industry. Although the modern methods of structural analysis such as the digital computer oriented finite element method have been perfected for seemingly more complex problems such as those found in high rise buildings, large earth dams, bridges and airplanes; a considerable amount of research, test and application experience will be required prior to the final development of the method as a railroad engineer's routine analysis tool.

It is only recently that the powerful finite element method has been applied to the static stress analysis solution of the very simplest of the train-track system configurations and loading environments. These efforts conducted by graduate engineering students at the Universities of Illinois and Arkansas were supported by University, industry, AAR and FRA research funds. The acute necessity for the development of an industry structural analysis capability to help cope with the severe track roadbed problems in the U. S. requires that an accelerated research and development program be initiated in the near future to provide adequate static and dynamic analysis capability for future design and rebuilding of the American railroad track system.

In this paper a brief history of track analysis efforts of the past 100 years and an introduction of the finite element method are presented. The accomplishments for finite element analysis of dams and embankments and their similarity to railroad systems are noted, and a brief review of one of the graduate student track analysis papers is presented. The new study for the application of the finite element method to analyze lime injected or chemically treated roadbed soils at the University of Arkansas is discussed. The development of an analysis capability for this complex nonhomogeneous, multilayered soil/structural system is one example of the special applications which will be possible with the finite element method of analysis. To assist the engineers who have only been recently introduced to this method, a short bibliography of finite element method conference papers and textbooks is presented.

INTRODUCTION

The analysis of the track structural system to obtain stresses, strains and displacements presents a difficult problem to the railroad civil engineer. To begin the engineer must obtain the stress-dependent engineering properties of a loaded soil mass which exhibits considerable variation in its profile. These properties must include the dynamic or impact repeated loading effects of various possible train load spectrums. After the engineer has obtained these items, he must have at his disposal a method of analysis which can be utilized for the solution of complex, layered, three-dimensional, nonlinear, nonhomogeneous structural systems acted upon by static and dynamic repeated loads. A failure criteria must be devised which will predict in advance the failure mechanisms of the rails, ties, connections, ballast and the soil mass. Each of these is an integral part of the total problem and to make matters more complicated the train imposed loads are integrated with the track system through rail contact, rail profile, plastic and elastic moduli and spring rate of the combined properties of all the structural components.

The solution of this problem rivals the complexity and difficulty of the most complicated civil and aerospace structural configurations. The train-track system analysis problem has been generally ignored whereas the complexity of the huge civil and aerospace structural problems have long since been recognized and properly treated. The railroads can profit from the years of research engineering in these related industries and the resulting development of the sophisticated computer based technical tools for testing, analysis and design. This then is the key to the technical solution of the track analysis problem. The railroads must utilize advanced techniques, hardware and equipment, and employ engineering specialists to develop structural analyses capabilities.

In this paper the finite element method of analysis for the static solution of stresses, strains, and displacements in a multilayered, three-dimensional rail, tie, ballast and soil track system will be discussed. The proper engineering solution for the nonlinear static structural analysis of the layered roadbed is required to completely understand the action of the total track system. Simple example problems will illustrate the utilization of a two-dimensional, plane strain, isotropic nonlinear finite element computer program.

The analysis of the lime injected track system using the finite element method of analysis will be discussed. This method is particularly well suited for analysis of the chemically stabilized layered structural system. The use of an accurate analysis method to determine the engineering effect of the injection process will enable the railroad engineers to determine on a rational basis the design parameters associated with the economic and successful application of the lime injection method of soil stabilization.

ANALYSIS BACKGROUND

As long ago as 1870 the railroad experimental activities of Baron von Weber, Director of the State Railways of Saxony, were reported in the periodical London Engineering concerning the subject "Stability of the Permanent Way." The topics investigated included rail design and material studies, ties and chair supports and ballast materials. Experiments were made to obtain answers to the five following questions:

1. What is the resistance offered by a well-bedded sleeper of average size against lateral displacement in the ballast?

2. What is the resistance of the whole structure against displacement at one point, and what is the influence of the ballast and bedding, on and in which the structure rests, upon this resistance?

3. How far does the filling against the ends of the sleepers increase this resistance?

4. To what extent is the resistance to lateral displacement increased by the load on the structure?

5. How far does the application of piles or stones, etc., increase this resistance?

The results and conclusions from the experiment are described in detail in Reference 1. This study is particularly interesting in light of the fact that the many railroad engineering problems of 1870 remain today. Also, it is interesting to note that the track structural system design of 1870 is with few exceptions the same that is in use today. The new AAR, FRA and railroad industry research programs are still searching for rational solutions to some of these 100 year old problems associated with the stability of the permanent way.

One notable track analysis advancement occurred in 1914-1925 as a result of the work of the Special Committee to Report on Stresses in Railroad Track which had as its chairman Arthur N. Talbot. The work of the committee reported in detail in References 2 and 3 has been the basis for the engineering analysis of track systems for the past 50 years. That this effort whould have served so excellently for so long is I believe mainly due to the time and resources alloted to the solution of the problem. To quote from the 1917 Talbot Report, pages 1194 and 1195:

"From the beginning the Committee has realized that the problem assigned to it is very complicated, involving many difficulties and uncertainties. It has felt that an adequate report on stresses in railroad track must be based on experimental data derived from extensive tests on standard railroad track. It has also realized that, because of the complexity of the action of track under load and the variability of the conditions which may be found in track and load, adequate experimental work would involve long, painstaking, and repeated tests under many conditions, and also that considerable time would be required for reducing the data thus obtained and interpreting the results. It believed that results of value could be obtained only after prolonged work on the problem. Experience has shown that the anticipated difficulties were not over-estimated. It was found necessary to expend considerable effort and time on the development of instruments for use in the tests and on methods for conducting tests. The problem was studied, and the methods were developed in the light of the information gained during the work. The experimental work undertaken thus far has included the measurement of track depressions, ballast and roadway pressure, and fiber stress in rail, for both static and moving loads, and laboratory tests on the distribution of pressure through ballast."

--- "It is apparent, then, that railroad track has not been developed in the manner followed in the development and expansion of most engineering structures and structural parts, where the scientific study of forces and stresses and the use of analysis and experiment have contributed in a marked way to improvements and growth. Instead, the present standards of track have been evolved from previous practice through a process involving extension and trail, and judgment and experience. That track has attained its present state of excellence is a tribute to the sense, insight, and judgment of the many men who have contributed to its growth and development. It is not surprising then, that the Committee found it necessary to devote considerable effort to studying the fundamental principles of the mechanics of track action, this being done principally through experimental work on track which may be described as ordinary track in good condition."

These men obtained the best answers for the solution of the 1920 train-track problems using 1920 engineering teachnology, and it served the railroad industry well for several years. Today we can better analyze the problem with our more advanced theoretical knowledge, digital computers, new laboratory testing equipment and new instrumentation for determining inplace field soil measurements. We must apply current engineering technology in the economic and operational environment of 1975 railroad industry for the accurate analysis of the modern train-track system.

CURRENT ANALYSIS REQUIREMENTS

When President Paul Reistrup of Amtrak says "Track is Amtrak's main problem" he is certainly talking about existing track that is in need of rehabilitation; however, it is important to note that it is not only the existing older roadbeds constructed with early 20th century technology that are presenting problems to the railroad industry. Today as never before the industry requires new and better structural analysis techniques for static and dynamics analysis for stresses, strains and deformations of train-track systems. The Black Mesa and Lake Powell line in Arizona is of recent vintage construction incorporating modern track design and construction methods, materials and hardware. The track is again the main problem for this line is experiencing track problems. There has been considerable speculation by the industry as to the cause of the problems, and even some modifications have been tried with little success. "The report now under study is expected to concentrate on the total system design: size of the rail, space of the ties, weight of the cars, and degree of ballast compaction," Modern Railroad, August 1975. It is thus evident on this new line that (1) the track system was not designed and constructed properly, (2) the existing analytical tools for analysis did not accurately determine the operational static and dynamic stresses, strains and deformations and predict modes of failure and (3) new accurate methods of analysis correctly utilized and construction properly placed would have saved many thousands of dollars. This example has been pointed out only to emphasize that there is a real need for structural methods of analysis research and development work for new track construction as well as rehabilitation of existing overloaded and deteriorating tracks.

FINITE ELEMENT ANALYSIS HISTORY

The finite element method of analysis gained widespread use in the U. S. in the aerospace industry beginning in 1956 with the publication of the now famous paper by Turner, Clough, Martin and Topp, research engineers working for the Boeing Company of Renton, Washington, Reference 4. In the ensuing period of research and expansion, the understanding, use and capabilities of the finite element method of analysis were advanced principally in support of the aerospace industry. Large companies established teams of engineering specialists in a rapid race to out do one another to obtain a better method of analysis. The analysis method development progress in this period up until approximately 1970 was extremely rapid. A parallel competitive effort in the computer industry helped to promote a goodly portion of this progress. It was a very exiciting and profitable time for the engineers involved.

In 1965 the 1st Conference on Matrix methods in structural mechanics was held at Wright Patterson AFB, Ohio, Reference 5. Those in attendance included J. H. Argyris of the University of London, Fraeijs de Veubeke of Belgium, Ray Clough of University California, Paul Denke of Douglas Aircraft, Dick Gallagher of Bell Aerosystems, O. C. Zienkiewicz of the University of Wales, Bob Melosh of Philco-Ford, Harrold Martin of the University of Washington, E. C. Pestel of Germany, Ted Pian of MIT, J. S. Przemieniecki, Conference Director, of the Air Force Institute of Technology, Lucian Schmit of Case Institute of Technology, M. J. Turner of the Boeing Company, and many others from around the world. There was, in fact, a veritable list of who's who in engineering structural analysis in attendance. This set the stage for many later technical meetings and gave a data base for the publication of many papers and books. The first textbook on finite element analysis in the U. S. was published in 1966 by the late Professor Harold C. Martin of the team of Turner, Clough, Martin and Topp. Today there are issued several new books on this subject each year. A list of recent finite element textbooks is presented in the Bibliography at the end of this report.

To note an interesting example of the use of finite element analysis in the 60's, the wing-body intersection of the Boeing 747 airplane analyzed by the method was completed in 1967 and reported in Reference 6. The analysis was performed on the CDC 6600 computer using both the force and direct stiffness method of finite element analysis. The structural problem was analyzed by studying each major portion of the intersection structure. In Figures 1 and 2 are shown the structural system, the substructures and the finite element idealization of the substructures. The total problem contained 4,266 nodes, 12,549 elements and utilized the solution of 13,870 simultaneous equations. This analysis was completed in a 10-month period and represented the largest of several similar analyses completed at the various aircraft companies during the 60's using the finite element method of analysis.

In this same period of time NASA contracted for a multipurpose structural analysis computer program to be called NASTRAN. This program was the first of the extremely large structural analysis programs. NASTRAN became operational in 1970 and is currently available. Another large multi-purpose structural analysis computer program is operated by McDonald-Douglas Automation Company, and it includes ICES and STRUDL. The McDonald program is available for a nominal monthly cost through utilization of a remote computer terminal.

Certainly, one could write a lengthy book about this period of structural advance and the people, companies, universities, and government organizations that made it all happen. Today we can use the finite element method of analysis for the analysis of dams, buildings, towers, bridges, vehicles, foundations, ships and machinery. The method has outstanding capabilities for the analysis of these total integrated systems. It can also be used to analyze contract stress in wheel flanges, stress concentrations around bolt holes, and critical fracture problems. From the structural engineer's point of view the finite element method of analysis is the singular best technology available today for economic and accurate analysis of the complex train-track structural system.

RAILROAD ANALYSIS DEVELOPMENT TASKS

There are no standard engineering equations available to analyze the complex train-track structural system. It is irregular at the surface, infinite at the lower boundaries and composed of many decidedly different structural materials. It is a nonhomogeneous, nonlinear, layered, three-dimensional continous structural system and very difficult to analyze. There have been only a few attempts to solve the problem using the finite element method of analysis and very few technical papers issued which have contributed to the state-of-knowledge in the engineering community with specific application to the finite element analysis of the train-track structure. Notwithstanding that the technological advances of the 60's are at our disposal for the analysis of this problem, it is unlike other problems that have previously been solved, and therefore it does not readily yield accurate answers to the application of the existing computer analysis techniques. Several years of development research will be necessary prior to the use of the finite element method of analysis for the efficient solution of the track structural problem as a routine engineering function.

It is evident that the total development task has been greatly reduced through benefit of the technical knowledge gained in the aerospace and civil industries. The railroads can achieve satisfactory competence with only a fraction of the research commitment of these industries. This fraction, however, represents a large dollar comment, especially for an industry that has not tended to pursue progress in this area in the past. It will be necessary for the AAR and FRA through their federal research programs to augment the industry's efforts in this area and to encourage support from engineering specialists and engineering organizations who can assist the railroad industry in making rapid progress in this area.

STRUCTURAL MODELING

Structural modeling of the train-track system will first require that the total structure be idealized into substructures and the the substructures be simulated or modeled by individual finite elements. In concept this is easy to grasp when the structural system is composed of individual parts small enough to be each modeled as one finite element. The ideal of modeling a continum or continuous body by a series of small elements requires that one divide up the structure by a suitable grid into a number of discrete elements interconnected at their nodal points, hence the name finite element method of analysis.

As a simple example of structural modeling we can look at the analysis of a cantilevered beam, Figures 3 and 4. The beam is modeled for finite element analysis by dividing it into finite element configurations, see Figures 5 thru 8. Using a standard "available" finite element computer program with triangular, rectangular and quadrilateral elements the beam is then analyzed for static inplane loads to obtain stresses, strains and displacements. The finite element solution of this simple problem for several different grid simulations and three different finite elements shows the comparative degree of accuracy and suitability of each grid and each element for this particular problem, Reference 7. The resulting calculated tip deflections for a 1000 lb. load are plotted in Figure 9. The variability of the answers illustrates the need for engineering training and understanding in the application of the finite element method prior to production utilization. Because this was a simple problem, it could have been solved by the usual equations of engineering analysis.

Next we can consider the analysis of a cantilevered beam composed of different materials and designed of an irregular configuration. Again this problem could be readily solved by the finite element method, however, this time with out aid of a computer program we would need to resort to trial and error design and testing. The usual equations of engineering analysis are not adequate to solve this more complex problem.

EARTH DAM EMBANKMENT ANALYSIS

The finite element method has been found to be well adapted for the static analysis of earth dam embankments. In 1966, Professor Ray W. Clough of Berkley presented the first paper on this subject at the ASCE Soil Mechanics and Foundation Division Conference on Stability and Performance of Slopes and Embankments, Reference 8. The paper evaluated the effect of incremental dam construction, studied the effect of foundation flexibility on embankment stresses and deformation, and considered the soil non-linearity in the analysis of a specific embankment configuration. Because the analysis of stresses and deformation in earth embankments is an exceedingly complex problem, it was necessary to make a number of assumptions and limitations in order to render the problem tractable. Clough considered the most important assumption to be that the actual three-dimensional system could be represented as a two-dimensional plane strain problem, considering a central cross section normal to the axis of the dam. For the purpose of the study the 100 ft. high embankment areas was taken as standard. The finite element idealization used in the analysis of the dam is shown in Figure 10. The vertical stress distribution determined in the analysis is presented in Figure 11. The study determined that incremental analysis procedure reliably predicted stresses and that foundation deformations can have a sufficient effect. The analysis of the Olter Brook Dam demonstrated for the first time that the finite element method using nonlinear materials properties can predict with remarkable accuracy the deformations actually observed during the construction process.

In 1972, Kulhaway and Duncan presented a paper on the finite element analysis of the Oroville Dam, Reference 9. The 80,000,000 cu. yd. Oroville embankment has a base width of 3,500 ft. and maximum height of 770 ft. The dam was designed and constructed in major zones with an inclined impervious core, the transition, the shell and the 128 ft. high concrete core block shown in Figure 12. The finite element analysis was conducted using non-linear, stress-dependent, stress-strain relationships for the tangent modulus and the tangent Poisson's ratio of the embankment soils, Reference 10.

The embankment was instrumented extensively, and the data thus obtained was used to evaluate the accuracy of the calculated stresses and deformations predicted by the finite element method analysis. The results of the analysis, compared to the results obtained from the instrumentation, are presented in the paper. The studies described indicate that the results of the nonlinear finite element analysis, conducted using properties measured under appropriate laboratory test conditions and incremental analysis procedures, are in good agreement with the actual behavior of the Oroville Dam.

In 1973 Lefebvre, Duncan and Wilson presented a paper to show the accuracy of two-dimensional finite element analysis of dams in V-shaped valleys, Reference 11. This was done by comparing the results of plane-strain analyses of the transverse section, and the results of both plane strain and plane stress analyses of the longitudinal sections, with the results of three-dimensional analyses of dams. The three-dimensional analyses were conducted on a three-dimensional finite element computer program, "SOLID SAP," developed by Wilson, Reference 12. Wilson's program is capable of running time efficiencies comparable to those for two-dimensional analysis. The three-dimensional finite element program offers a sufficient improvement for accurate analysis of railroad track embankment problems.

RAILROAD FINITE ELEMENT ANALYSIS

Little interest has been generated for the application of the finite element method for static analysis of the railroad track structural system. The first paper which specifically addressed the topic was written by J. R. Lundgren, a graduate student at the University of Illinois in 1970, Reference 14. In Lundgren's research a small computer program was generated for the elastic finite element solution of the track longitudinal cross-section. The three-dimensional effect of the transverse stress distribution was not taken into account. Following Lundgren's paper there has been little activity. Two finite element analysis entries are listed in the latest edition of RRIS, one of a vibration analysis of a railway coach and the other refers to Dr. G. C. Martin's current AAR Track-Train Dynamics program. The Track-Train Dynamics Bibliography, February, 1973 does not contain any finite element references.

Lundgren's paper is a well written masters thesis, and it offers a good beginning point. At least two other works are in progress, one by B. J. McAlister of the University of Arkansas, Reference 13, and one by S. Tayabji of the University of Illinois, both are graduate students sponsored by FRA research funds.

In McAlister's paper the nonlinear finite element method of analysis is utilized. A two dimensional finite element simulation approximation of the train-track structure is analyzed first as a transverse section, Figure 13 and then as a longitudinal section, Figure 14. The effect of the two phase analysis is to generate stresses, strains and deflections which are representative of those under the track, which is in reality a three-dimensional solid system. The computer program utilized was written by Sogge and Richard of the University of Arizona, Reference 15, for the analysis of stresses, strains and displacements around a buried pipe line. The Sogge and Richard computer program, SSI, utilizes the Duncan and Chang nonlinear stress-dependent, stress-strain relationships for the tangent modulus and the tangent Poisson's ratio of the embankment soils, Reference 10. McAlister modified the SSI computer program with the addition of an automatic node generator to simulate the track problem. Actual railroad soil properties obtained in the University of Arkansas test laboratories were utilized for the stress dependent material representation. Design parameters for the ballast, ties and rails were approximated. The nonlinear analysis of the subsoil was emphasized.

The drastic shortage of test data made it difficult for the authors to validate their method of analysis. Also, since the Lundgren paper solves the two-dimensional elastic problem and the McAlister paper solves the simulated three-dimensional nonlinear problem, the solutions can not be readily compared. The typical stress distributions from the McAlister paper for the transverse and longitudinal stresses in a loaded track system are shown in Figures 12 and 13 respectively.

It should be noted that McAlister's roadbed simulation for the transverse section utilized only 195 finite elements interconnected by 118 nodes and that the basic finite element used was a constant stress triangle. A considerable improvement in the accuracy of the solution will be possible with a finer mesh grid in the areas of high stress gradient and a deeper and wider section to account for the elastic foundation. In addition, the use of higher order finite elements or a three-dimensional program would also be expected to improve the accuracy of the solution. The determination of the nonlinear material parameters for both the ballast and the subsoil should be included for each layer.

ANALYSIS OF LIME INJECTED ROADBEDS

The finite element method of analysis will provide a suitable engineering analysis tool for the rational static and dynamic stress analysis of layered lime injected railroad track subsoil systems. A portion of the improvement associated with injection of lime slurry is as a result of strength increased adjacent to lime seams within the soil mass. The increased strength and stiffness of a lime-injected soil mass is a function of the total area of lime seams contained within its volume. By determining the cross-sectional area and thickness of the lime seams based upon gallons of slurry injected per each surface square foot and including the strength and stiffness properties of the soil-lime interface, it will be possible to develop a rational finite element method of analysis capable of predicting the behavior of lime-injected roadbeds subjected to static and dynamic loadings. The development of a special finite element for lime-soil interface will be necessary in order to proceed with this approach. Thus far this work is in the planning stages only. An extensive laboratory testing program will be required in conjunction with the theoretical analysis methods development program. The finite element analysis of the lime injected roadbed will enable the engineers to predict the changes in the elastic and plastic moduli and the spring rates of the track system caused by the injection of lime. It will provide the capability for stress analysis and failure prediction necessary for the lime injection stabilization of new and existing track roadway systems.

CONCLUSIONS

The finite element method of analysis is the best method currently available to predict the stresses, strains and deformations of the train-track structural system. It is the only method that can include the effects of nonlinear material parameters, multiple load cases, temperature effects and geometric discontinuities to obtain an accurate failure analysis of the total integrated structure. The basic analysis method and computer programs developed for the aerospace and civil engineering problems will require additional studies prior to routine railroad applications. Current research efforts are insufficient to meet the needs of the railroad industry. The future research programs must be conducted jointly by the railroad companies, the universities, and FRA and AAR research organizations to achieve balanced technical capabilities for the continued growth and rapid utilization of the research results.

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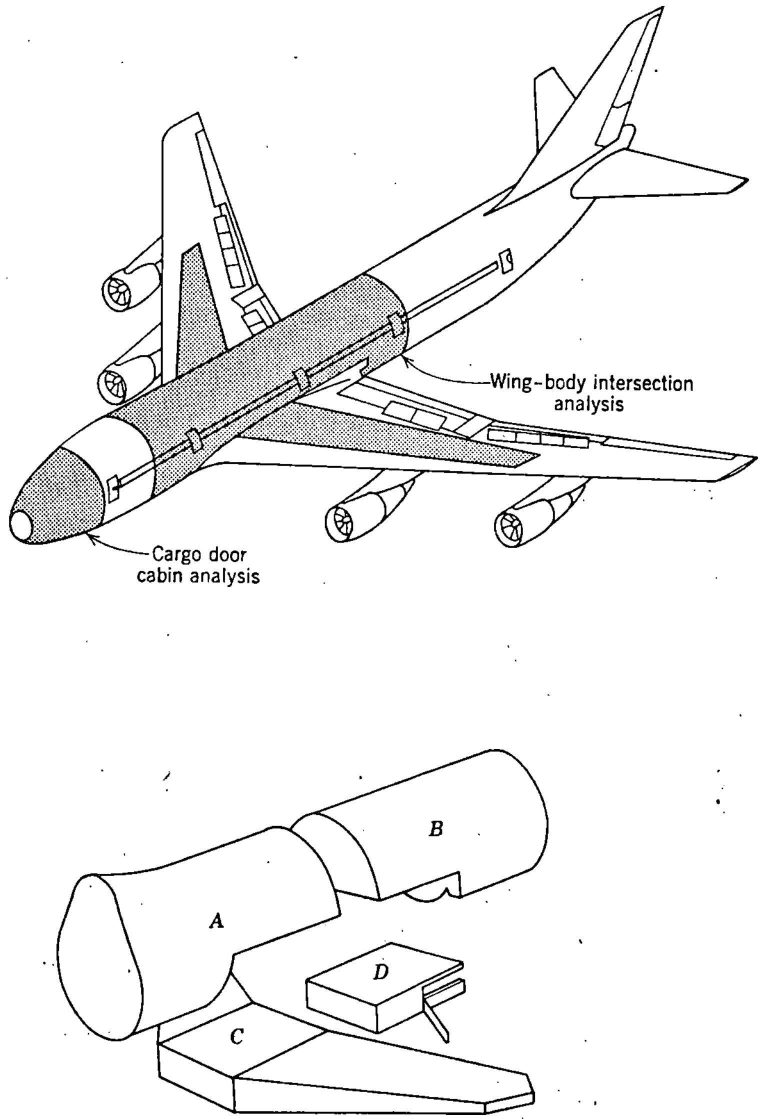


Figure 1   747 Airplane, Analyzed by the Finite Element Method

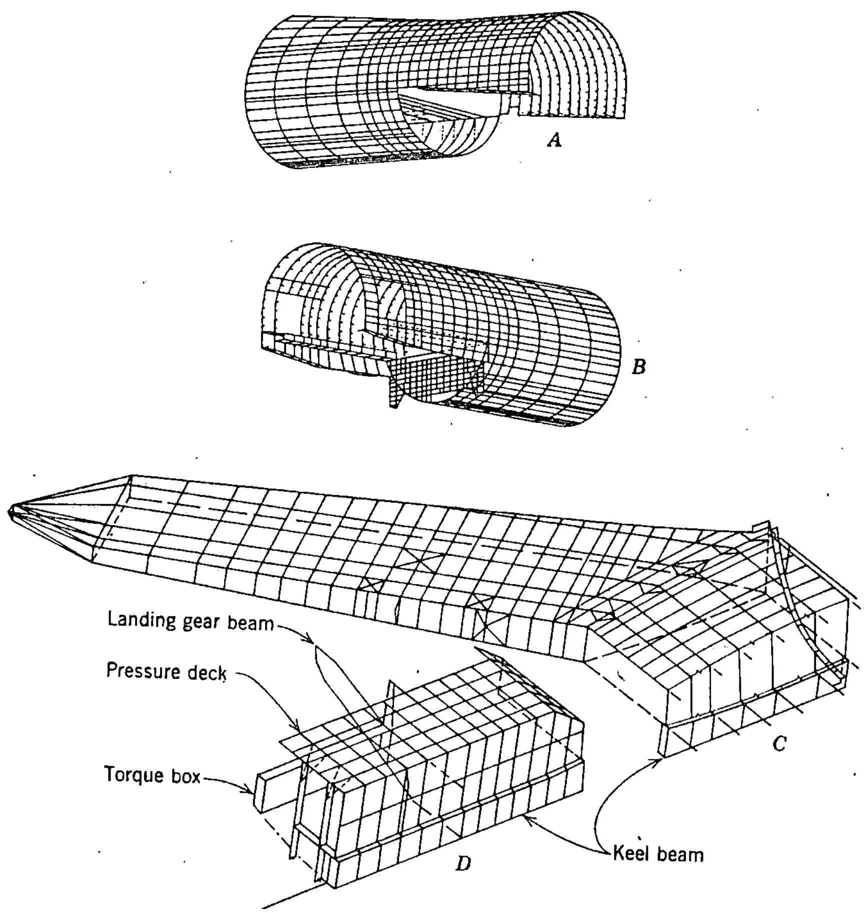


Figure 2 747 Airplane Finite Element Simulations

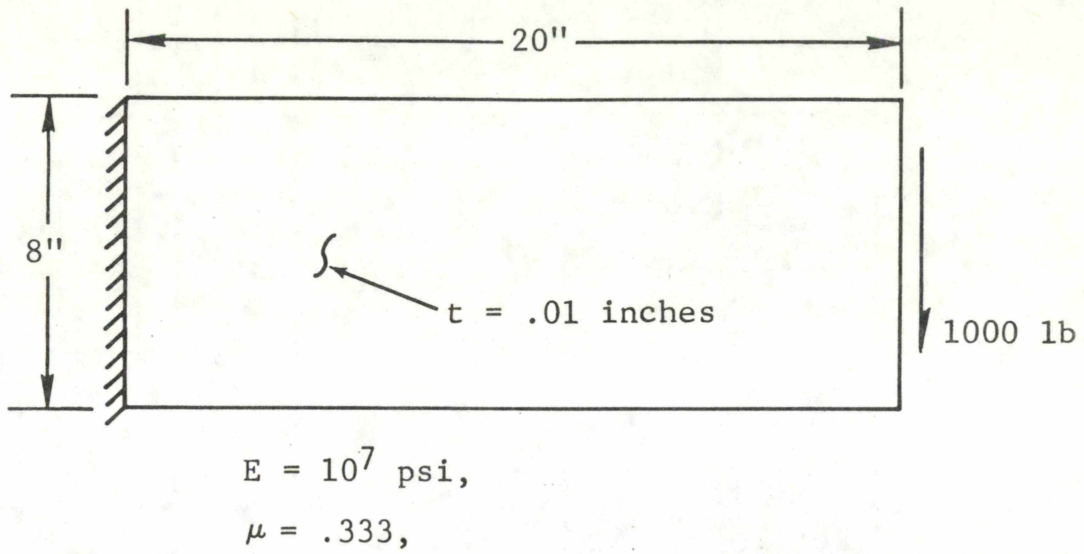


Figure 3 Cantilever Plate Problem, Models I Through IV

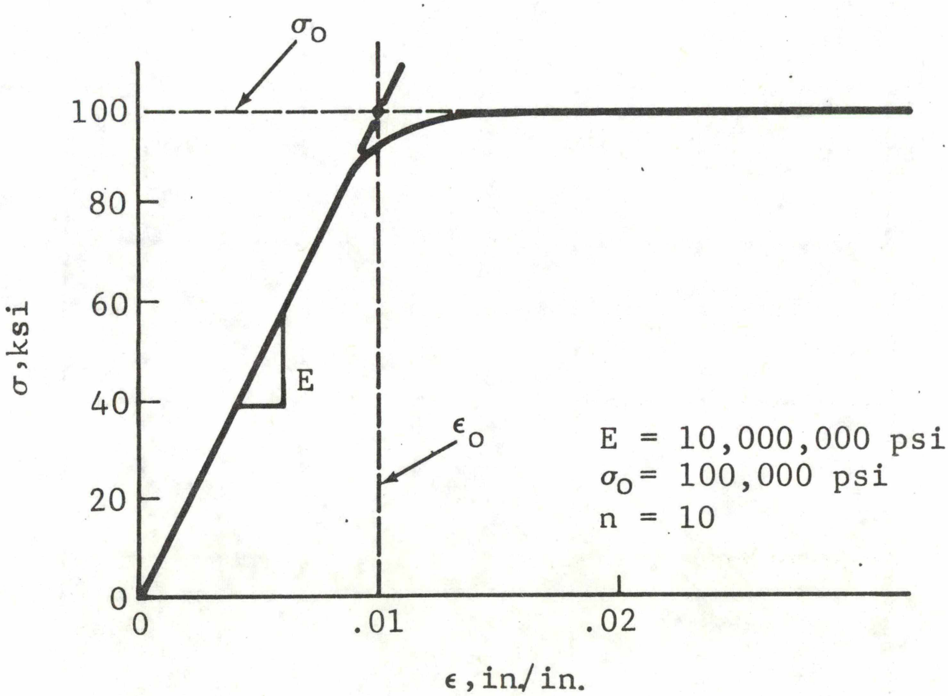


Figure 4 Nonlinear Stress-Strain Curve

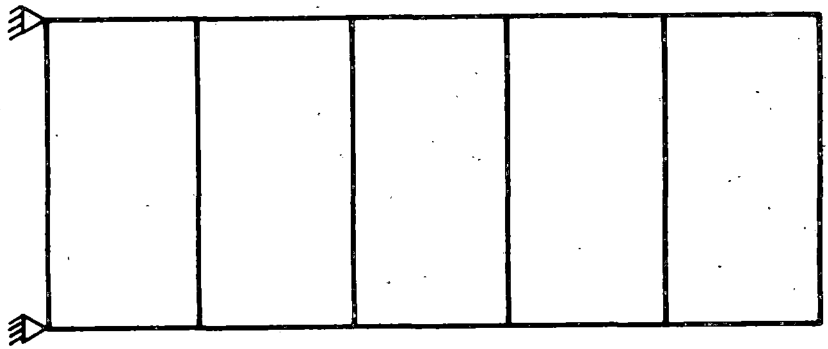


Figure 5 Model I Simulation, 5-Grid Plate

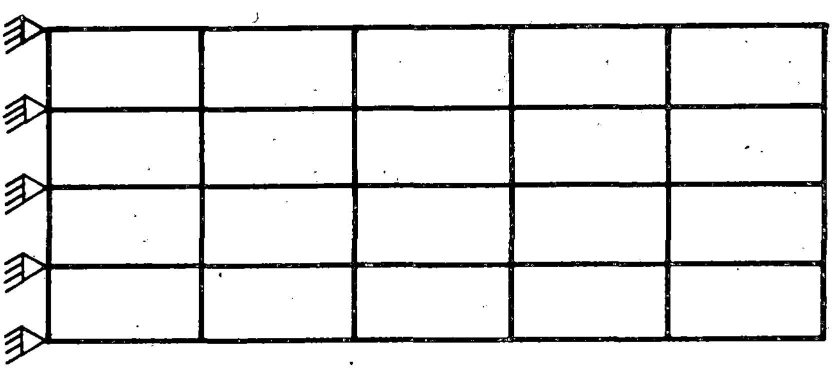


Figure 7 Model III Simulation, 20-Grid Plate

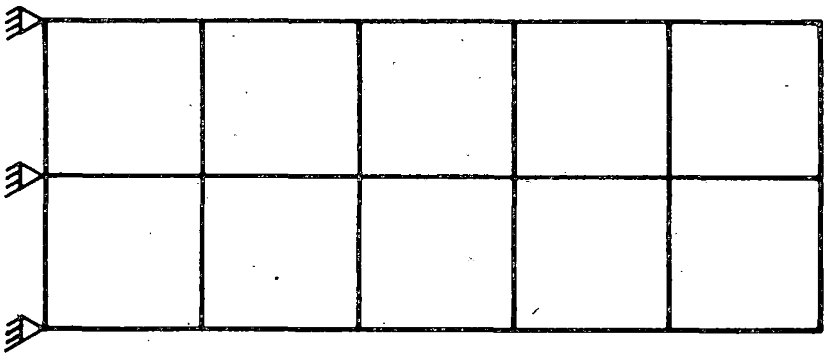


Figure 6 Model II Simulation, 10-Grid Plate

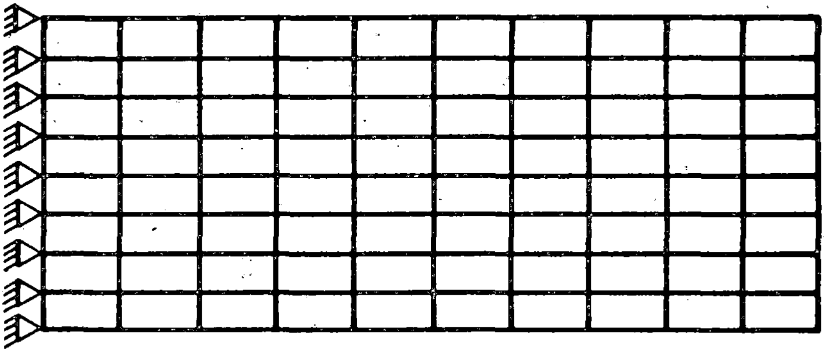


Figure 8 Model IV Simulation, 80-Grid Plate

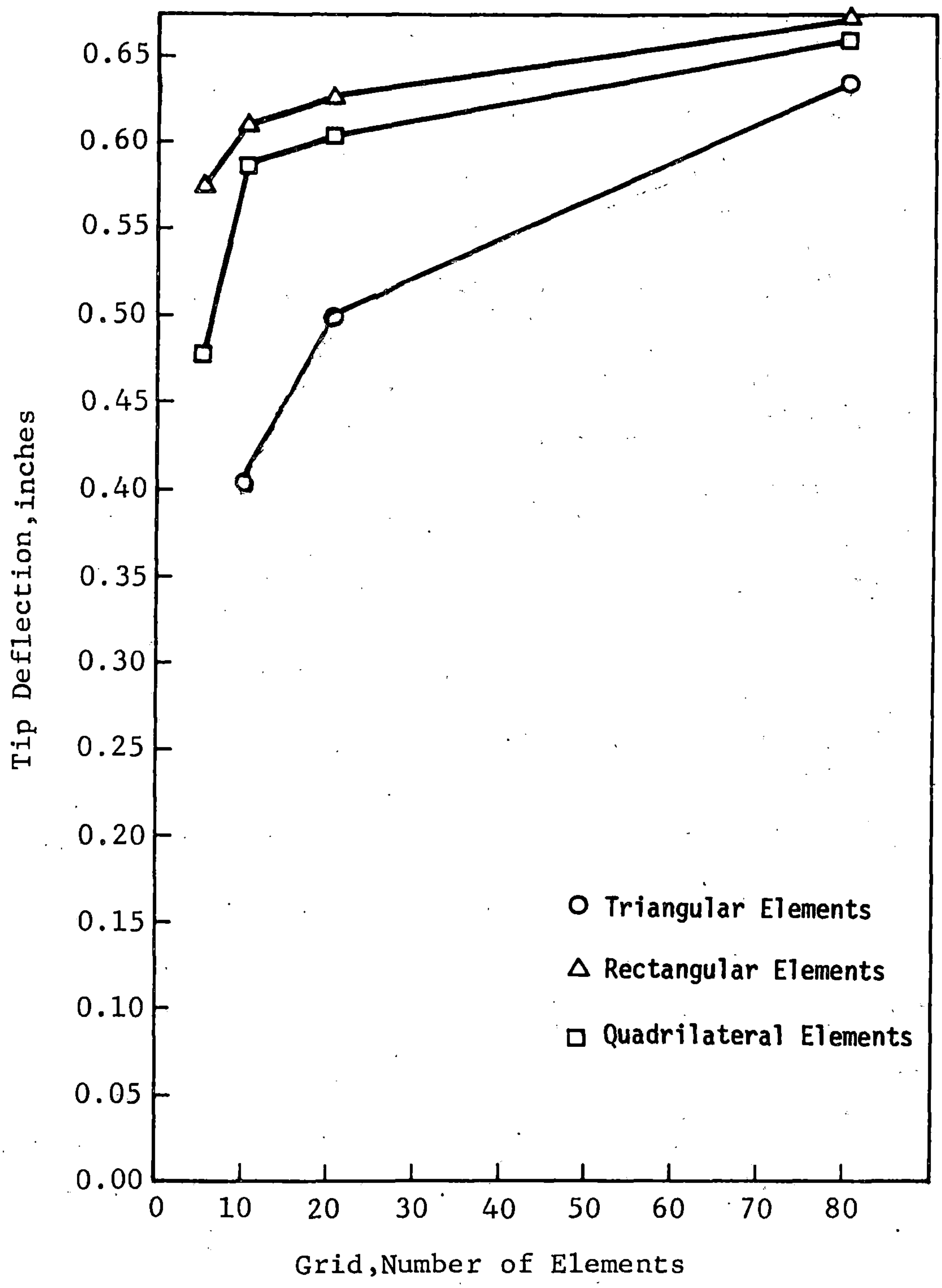


Figure 9 Elastic Cantilever Plate Tip Deflection vs Grid Size

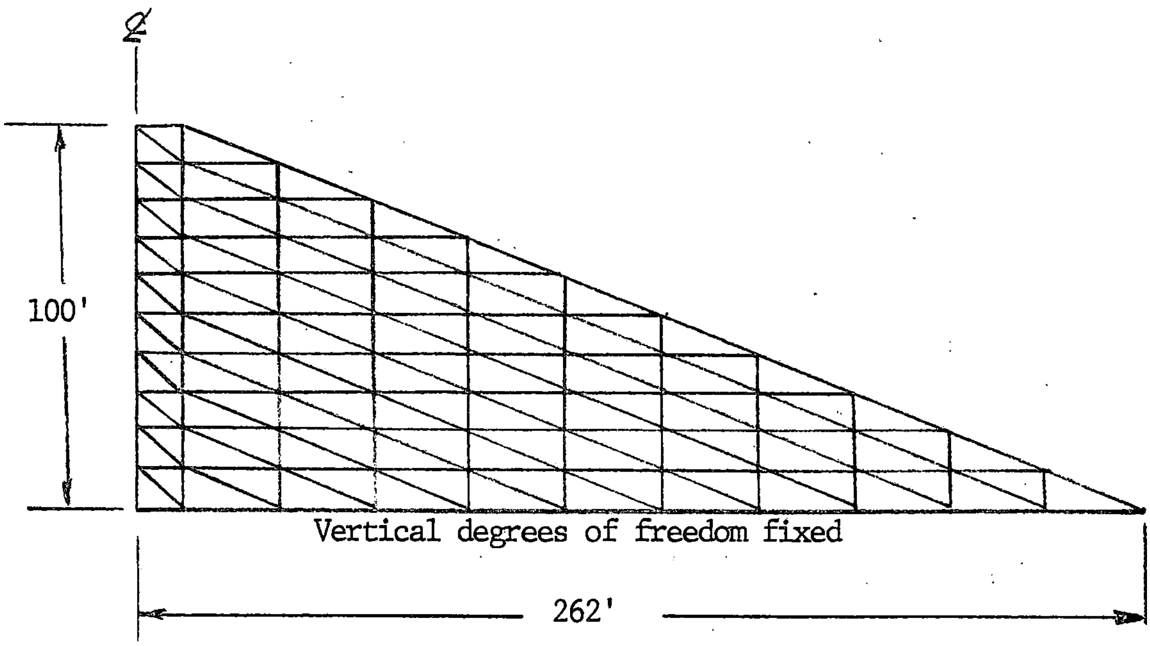


Figure 10 Finite Element Idealization of Earth Dam

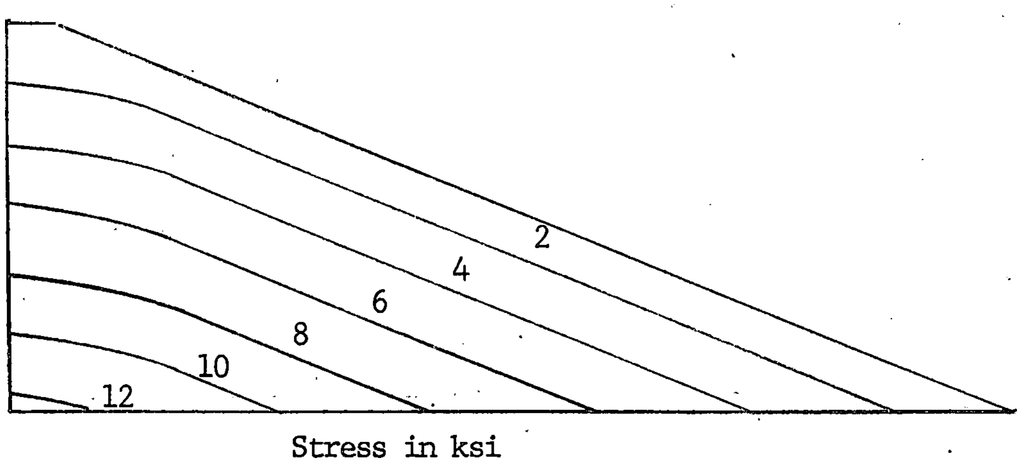
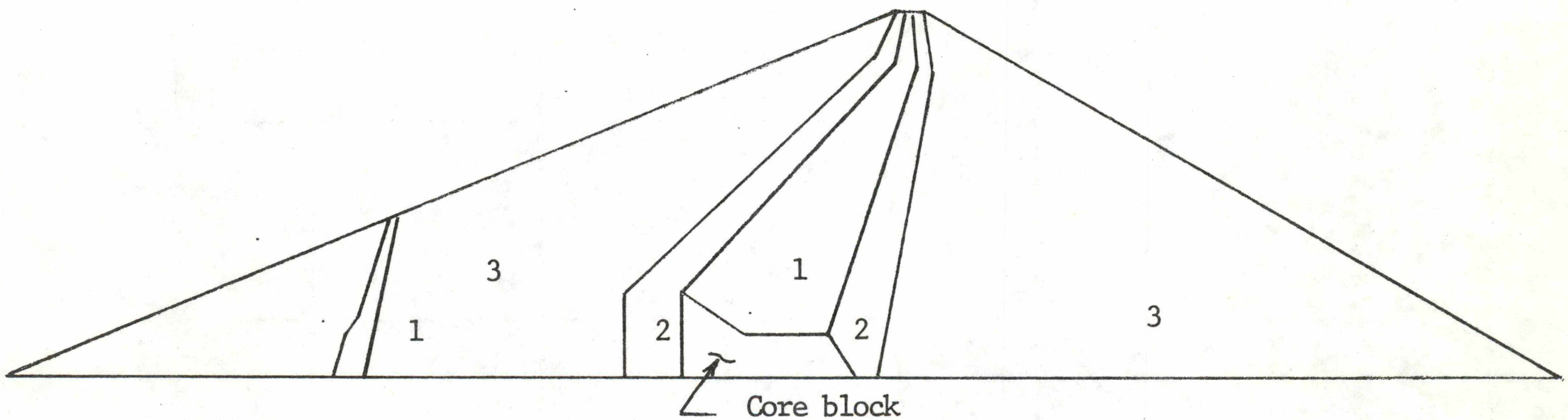


Figure 11 Vertical Normal Stress Contours Due to Dead Weight Loads



Zone 1 Impervious (clayey sandy gravel)
2 Transition (amphibolite gravel, fine)
3 Pervious (amphibolite gravel, course)

Figure 12 Oroville Dam Maximum Section

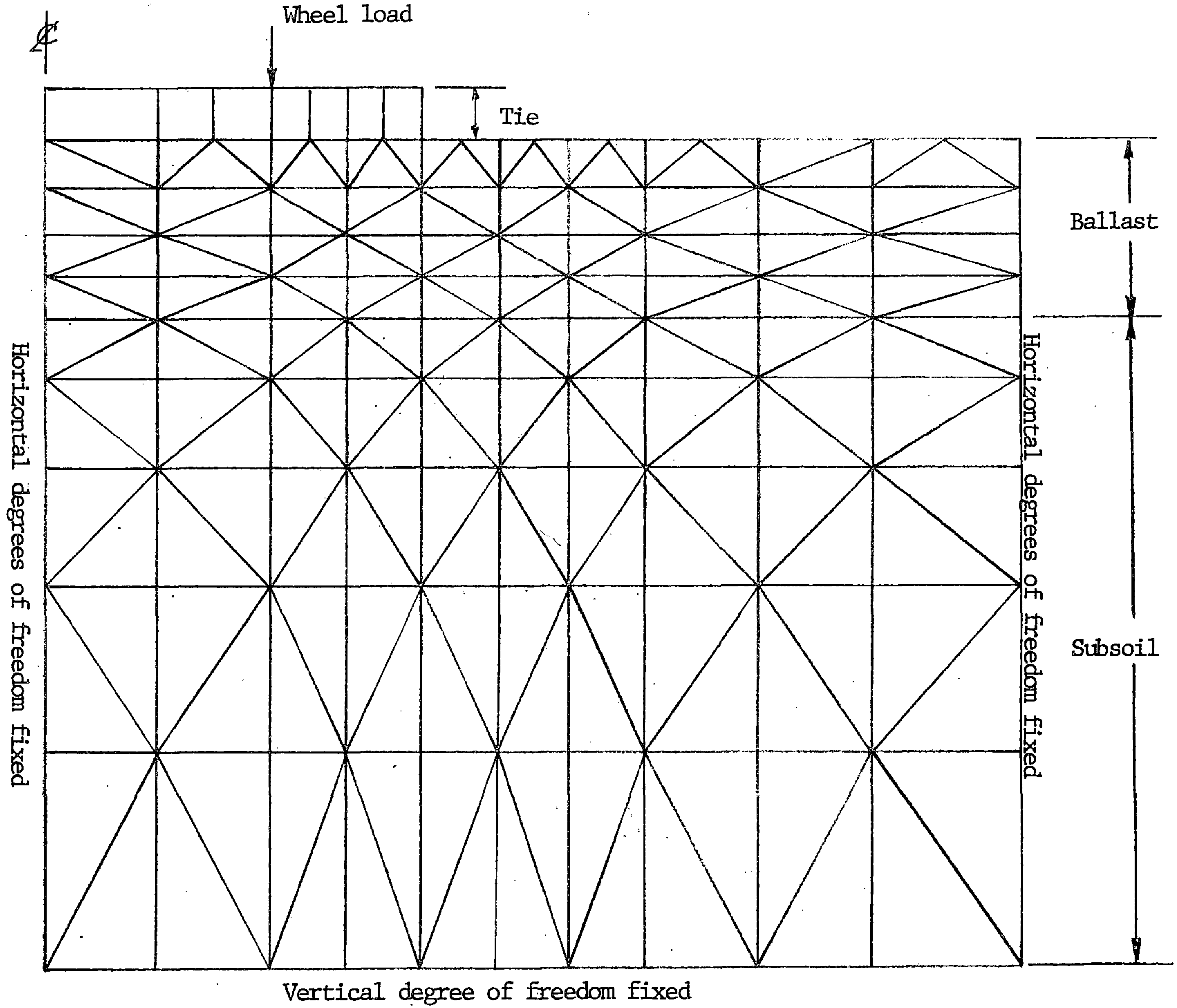


Figure 13 Railroad Subgrade - Transverse Simulation

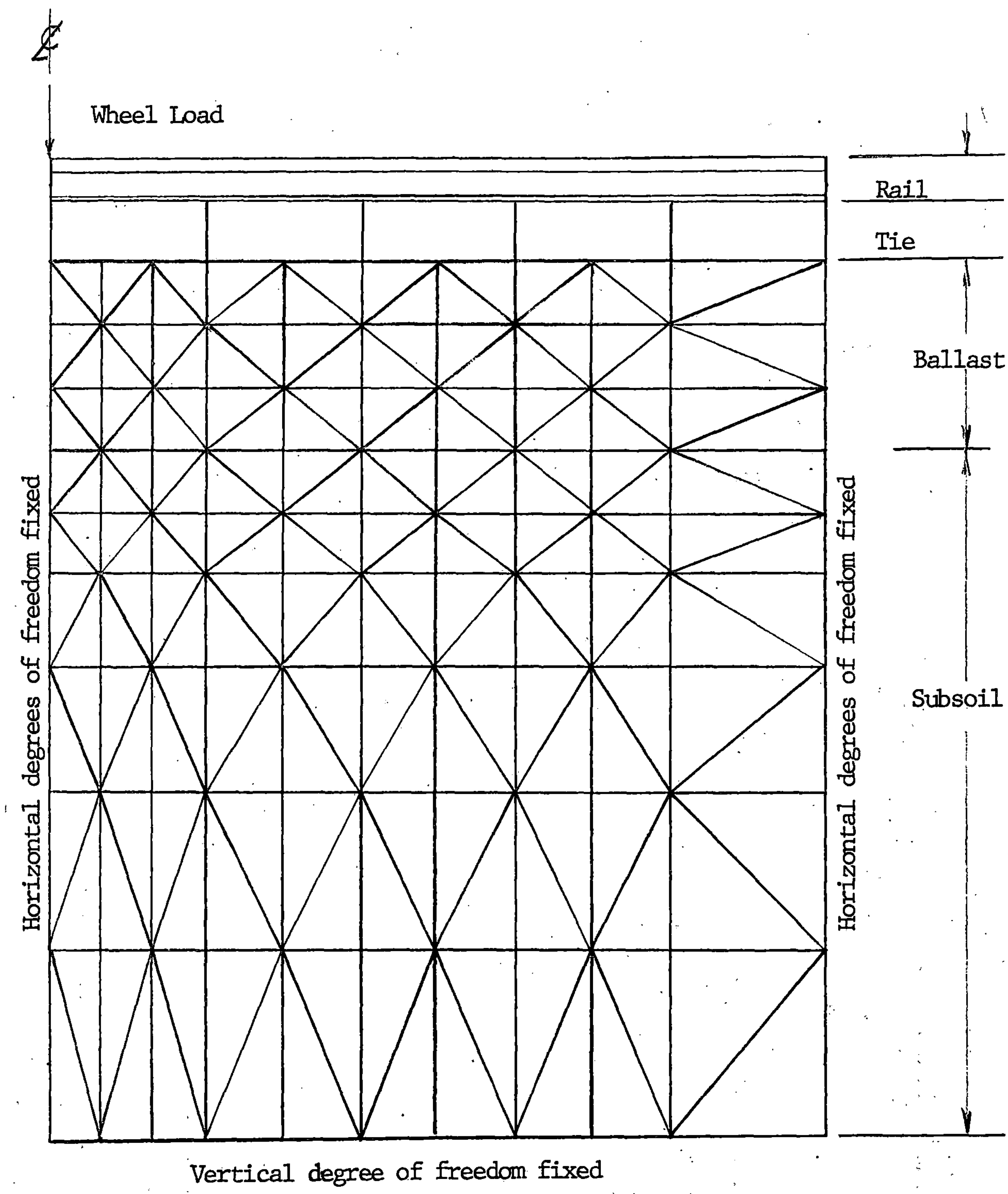


Figure 14 Railroad Subgrade - Longitudinal Simulation

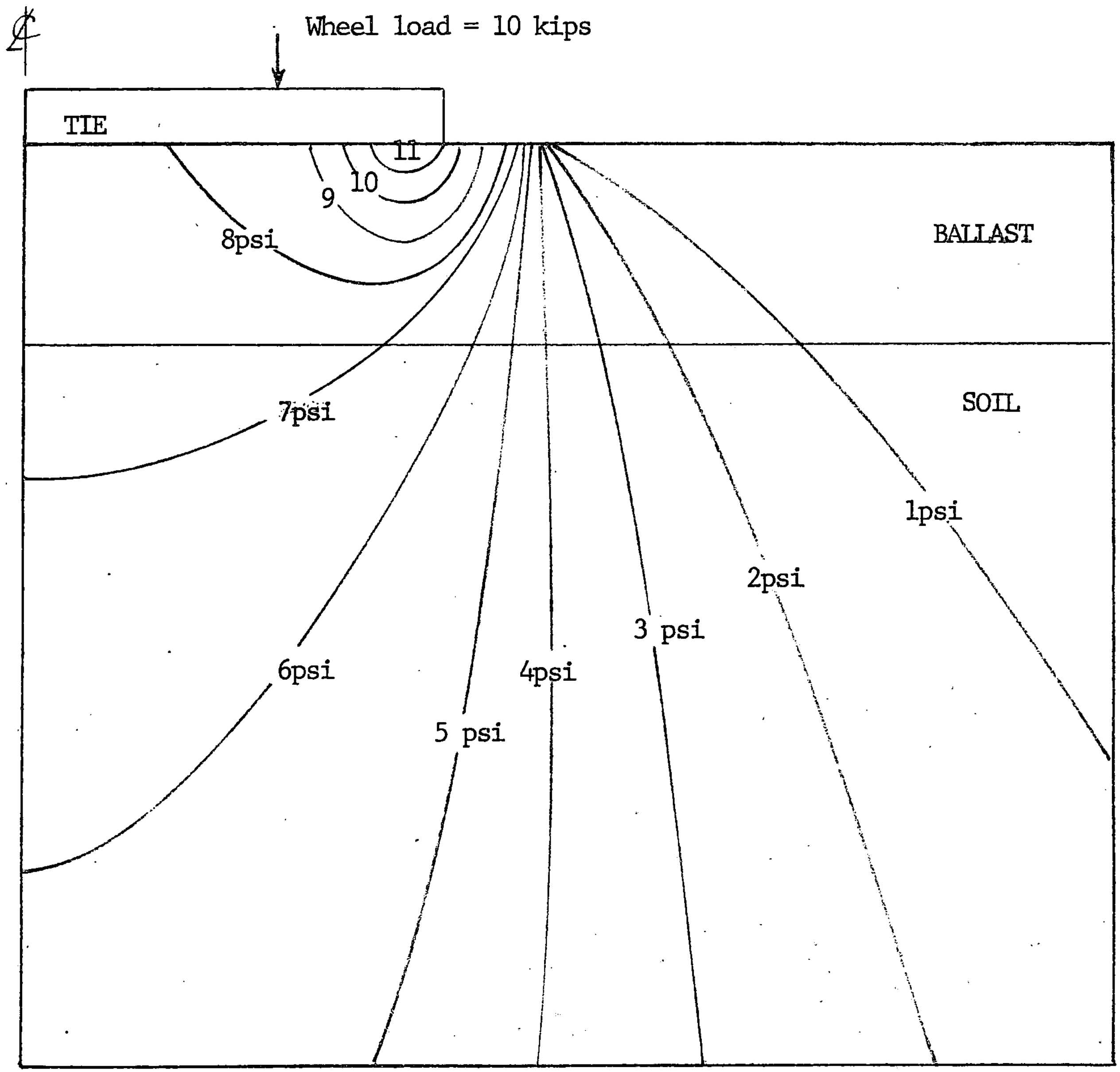


Figure 15 Roadbed Live Load Stress Contours - Transverse Section

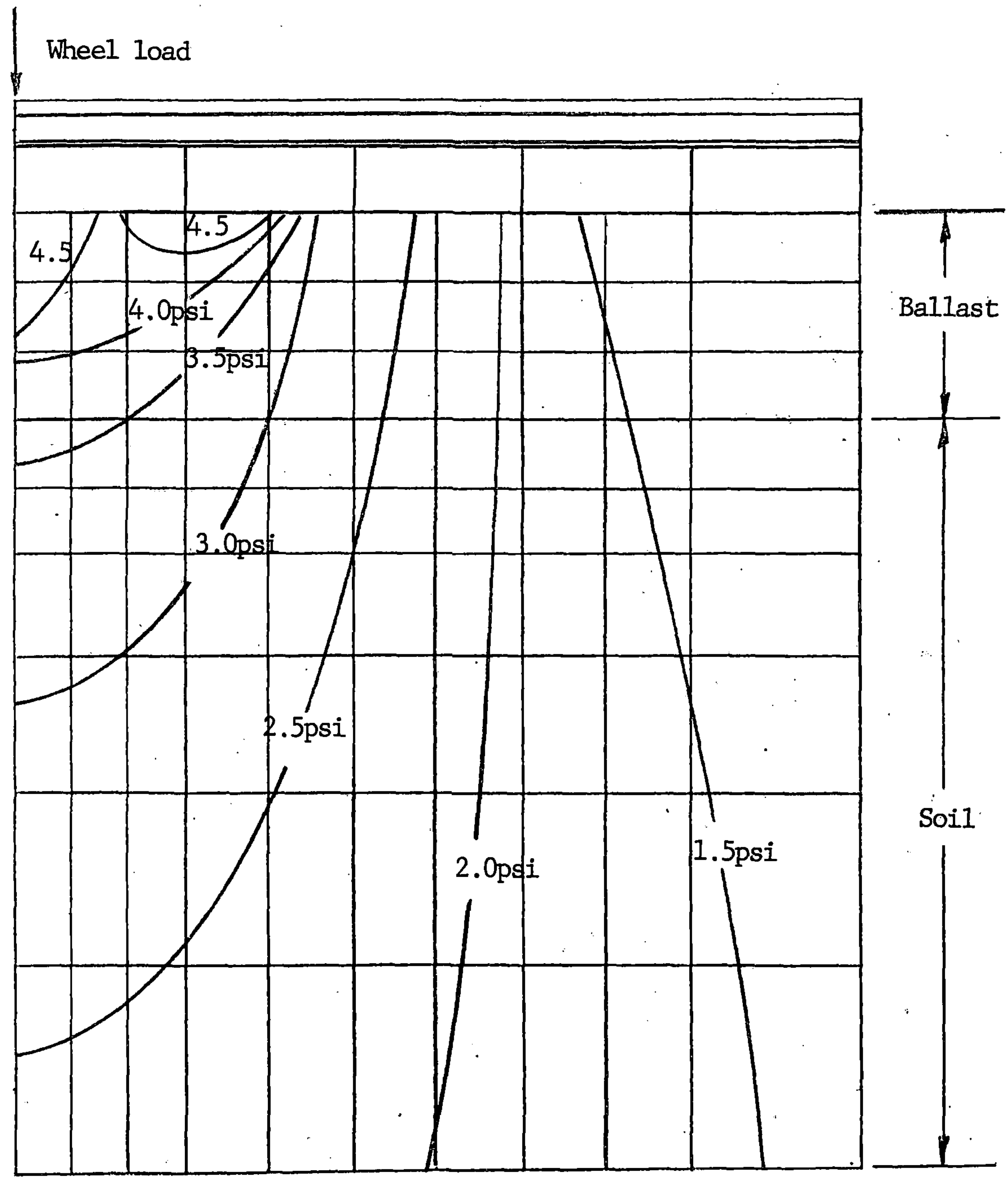


Figure 16 Vertical Stress Contours - Longitudinal Section

REPEATED LOAD TESTING ON
FOUR LIME STABILIZED OKLAHOMA SHALES

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

THE UNIVERSITY OF ARKANSAS

AND

THE FEDERAL RAILROAD ADMINISTRATION

Little Rock, Arkansas
August 21 and 22, 1975

REPEATED LOAD TESTING ON

FOUR LIME STABILIZED OKLAHOMA SHALES

ABSTRACT

The current methods of highway pavement design do not take into account the effects of climatic and environmental factors existing during the construction of a subgrade and the effects of traffic induced stresses on it after construction.

In this study, an attempt was made to evaluate the effects of simulated weathering and traffic induced stresses on four Oklahoma shales treated with the addition of lime. The amount of lime added ranged between zero and six percent. Shale or shale lime mix samples were subjected to wet-dry weathering cycles ranging in number from zero to fifty and tested under cyclic loading in the form of repetitive split tensile strength tests.

The lime treatment was effective in modifying the characteristics of clay shales containing predominantly montmorillonite clay minerals. For these shales, the sample behavior became progressively more brittle and the effect of weathering became less apparent with the increasing amount of lime.

REPEATED LOAD TESTING ON
FOUR LIME STABILIZED OKLAHOMA SHALES

INTRODUCTION

During grading operations, soil is excavated from its original location and transported to the fill site where it is spread in thin layers and compacted to maximum dry density at optimum moisture content. In certain cases, the subgrade soil must be stabilized. After completion of grading operations, the road is left unpaved for a certain period of time. During this period climatic and environmental factors act on the subgrade soil. High temperatures and low relative humidity conditions cause drying of the compacted soil. On the other hand, precipitation and frost action tend to increase subgrade moisture content. In the southern United States, freezing of subgrades is not a major problem, thus, increases in moisture content are due primarily to precipitation and high relative humidity conditions.

A number of wet dry cycles precede the paving operations. After the placement of the pavement structure, the moisture content levels in the subgrade soil vary within small ranges; however, the pavement is now subjected to the action of traffic loads. Under the influence of climatic and traffic factors, changes occur in the subgrade material. These changes are expected to be primarily physical in nature and their effects would be manifested by variations in the plasticity and strength characteristics of the soil material, which in turn affect the stability, durability and performance of pavement.

The scope of this study was two fold: First, to employ laboratory experiments which simulate more realistically the events that occur in the early life of a subgrade material, and second, to evaluate the effects of weathering and repetitive loading on four raw and lime stabilized shales of Oklahoma.

SELECTION OF MATERIALS

On the basis of their geologic, physiographic and geographic locations, 24 shales were obtained from various parts of Oklahoma. After obtaining routine engineering and geologic information the shales were divided into various groups, shales in each group having similar characteristics. From these groups four shales, each belonging to a different group, were selected for this study. Locations of these shales are shown in Fig. 1, and their relevant geologic and engineering information are depicted in Tables 1 and 2 respectively. As will be noticed, the

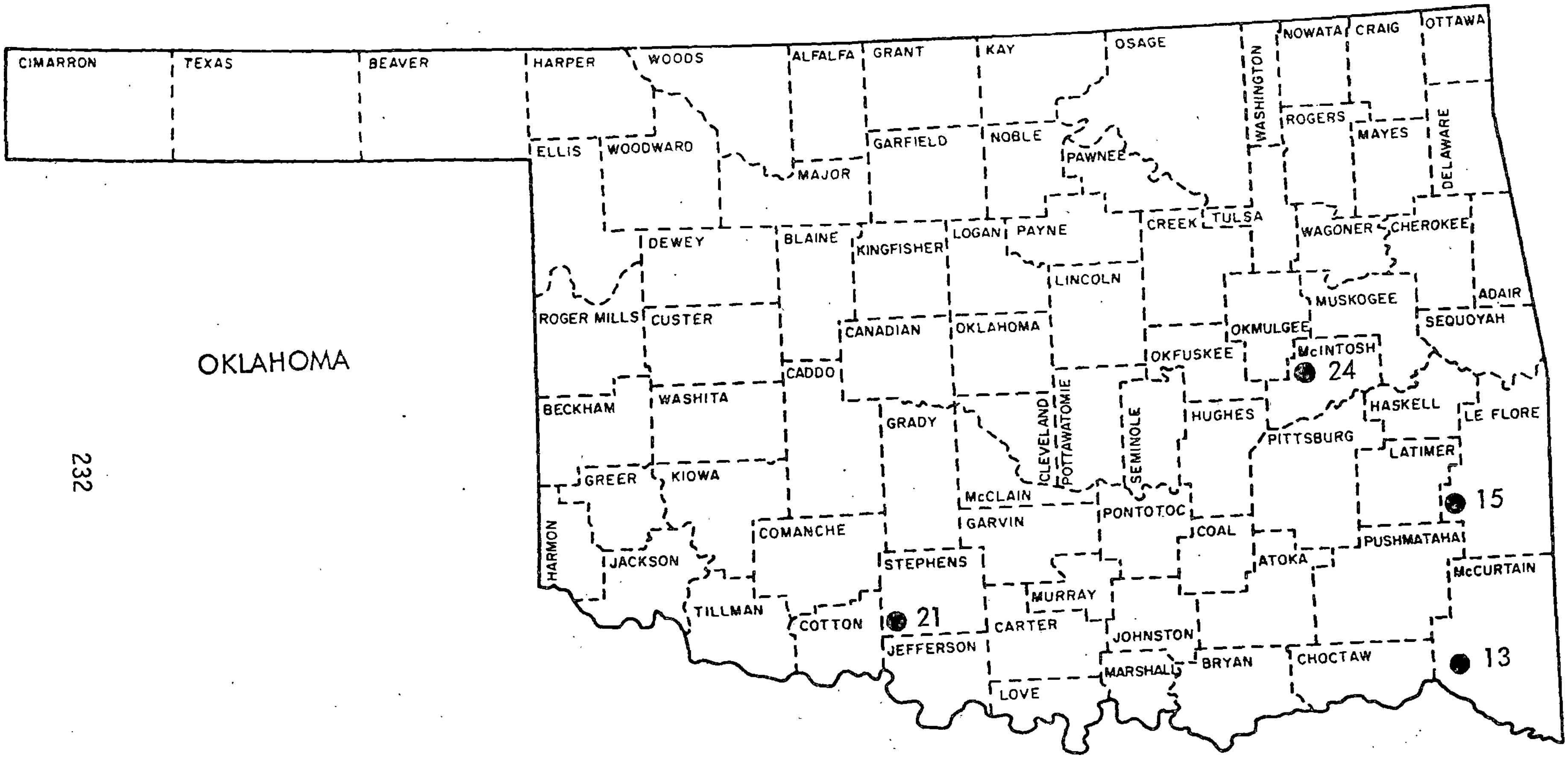

Figure 1. Location and identification of selected shale samples.

TABLE 1. GEOLOGIC INFROMATION ON SHALES[a]

Shale Number	County	Geologic System	Physiographic Region	Geologic Unit
13	McCurtain	Cretaceous	Red River	Washita
15	LeFlore	Mississippian	Ouachita Moutain	Stanley
21	Stephens	Permian	Red Beds Plains	Claypool
24	McIntosh	Pennsylvanian	Prairie Plains	Senora

[a]From Reference - Sheerar, 1932

TABLE 2. ENGINEERING PROPERTY DATA OF SHALES[a]

Shale No.	Grain Size Analysis,%			Specific Gravity	pH	UCS[b] psi	Volume Change %	AASHTO[c] Class.
	Silt	5μ Clay	2μ Clay					
13	35	59	48	2.73	5.1	22.5	10.3	A-7(6)
15	9	18	14	2.77	7.9	2.2	0.2	A-1-b
21	49	39	25	2.79	8.5	31.7	6.8	A-6
24	44	23	14	2.73	7.6	26.1	4.4	A-4

[a]From Reference - Laguros, 1972

[b]Unconfined Compressive Strength: (1 psi. = 6.89 kN/m^2)

[c]American Association of State Highway and Transportation Officials Classification.

AASHTO (American Association of State Highway and Transportation Officials) classification for these shales varies from A-1-b to A-7(6).

Air dried shale material passing U. S. Standard sieve No. 10 was used for the investigations of this study. The shales were stabilized using hydrated lime, U.S.P. (powder) $Ca(OH)_2$ manufactured by Mallinckrodt Chemical Works of St. Louis, Missouri. Distilled water prepared in the laboratory was used for all tests.

TESTING PROCEDURE

The lime retention point for shales 13, 21 and 24 was found to be between 2 and 4 percent of lime on dry soil weight basis (Annamalai et al., 1970). Shale 15 has AASHTO classification of A-1-b and is not considered suitable for lime stabilization. It was, however, included here to provide a reference point. For the shales 13, 21 and 24 it was noticed that significant changes take place in maximum dry density, optimum moisture content, plasticity characteristics and unconfined compressive strength between zero percent and three percent lime additions. Thus, the percentages of lime adopted for stabilization were 0, 1, 2, 4 and 6.

For all shales and shale lime mixes moisture density relationships were determined according to ASTM Designation D 698-64 T with the exception that the Harvard miniature compaction apparatus was used.

Samples for the repeated load test were compacted statically at maximum dry density and optimum moisture content. A prepared sample had a diameter of 1.35 in (34.3 mm) and a height of 2.95 in (75.0 mm).

Application of Wet-Dry Cycles

Based on the data for the ten year period 1960 - 1969 from 16 locations in Oklahoma the range of wet-dry cycles was found to be between 30 and 45 (Laguros, 1972). Since the time lapse between construction of subgrade and placement of pavement on it is usually in the range of six months, the numbers of wet-dry cycles adopted for this study were 0, 5, 15, 30 and 50.

All samples were wrapped in plastic sheets and placed in 100 percent relative humidity environment for one week. After this curing, samples were subjected to wet-dry cycles. Half of the total number of samples were tested in "humid" state and half in "dry" state. The samples subjected to zero wet-dry cycles and to be tested in humid state were taken out of the humidifier and directly used for strength testing. All other samples were placed in an oven set at 140°F (60°C) which corresponds to the maximum temperature in open areas for Oklahoma. The samples were oven dried for 12 hours. They were then transferred back to the 100 percent relative humidity atmosphere at 72°F (22.2°C) and kept in it for 24 hours. This drying and humidifying was continued until the required number of wet-dry cycles was completed. At the end of the cycle, the state of sample was termed "humid" if it had moisture content in equilibriation with the humidifier environment. On the other hand, if the sample had moisture content in equilibriation with the oven environment its state was termed as "dry". For the samples subjected to zero wet-dry cycles the moisture content in the humid state will be approximately equal to their molding or optimum moisture content.

Initially, it was proposed to soak the samples in water, but no sample could stand this severe treatment. Hence, the idea of soaking the samples was substituted by storing them in 100 percent relative humidity environment.

At the end of required wet-dry cycles the samples were weighed and their dimensions measured. After this they were subjected to cyclic split tensile strength testing.

Application of Cyclic Loads

The system for measuring loads and deformations consisted of load cell, displacement strain gage, Sanborn strip chart recorder and Atomic strip chart recorder.

The range of frequency of load application was between 6 APM (applications per minute) and 40 APM. No useful information was obtained on trial samples tested under frequency lower than 6 APM. For frequencies higher than 40 APM, the information obtained was not considered to be reliable. Since the dwell time was chosen to be 1.0 second, at 60 APM and at higher frequency settings the time period of load application became less than the dwell time and a continuous, rather than a wave curve, was obtained on the Sanborn strip chart.

Due to the limitations of the Sanborn strip chart recorder, a load of 40 lb (178 N) was adopted as the minimum level of load application. Both the loads and the frequencies were applied in increasing order. First, the 40 lb (178 N) load was applied 100 times at 6 APM. Then this load was applied 100 times at each of the higher frequencies. This procedure was repeated for the loads of 80, 120 and 160 lb (356, 534 and 712 N). The testing was continued until failure occurred. Initiation of any cracks visible to the eye was noted on the strip chart as soon as the crack became noticeable. In the case of certain samples, due to machine limitations, testing had to be terminated at 160 lb (712 N) load and 24 APM frequency.

DISCUSSION

On drying, some samples showed cracks, visible especially at the ends of the samples. The cracking was less apparent in the samples containing lime. No cracking was obvious in the samples containing lime at or above 2 percent treatment level. This observation confirms the beneficial aspects of lime stabilization.

The strength related characteristics of samples in this investigation were effected primarily by two major factors:

1. application of wet-dry cycles,
2. stabilization of shale due to the addition of lime.

Increasing the number of wet-dry cycles implies more extensive destruction of bonds among the particles constituting the sample mass. On the other hand, the increase in the number of wet-dry cycles prolongs the time element and pozzolanic reaction between shale and lime results in increased bond formation. This observation may help explain the wide range of values related to strength characteristics.

The load and displacement graphs produced by the recording equipment were continuous and showed the expected wave pattern. For most visco-elastic materials, the maximum strain lags behind the maximum stress amplitude in time (Seed et al., 1967). In this investigation, however, the stress applied was due to the movement of the lower platen of the machine which brought the sample in contact with the load cell assembly, and thus, the load and the diametral compression had their maxima at the same moment and no lag was noticed in their amplitudes. In split tensile strength tests since the stresses are primarily dependent upon the diametral strain (Kumar et al., 1975), it was decided to plot the diametral strain against the load and the frequency of load application. A graph of this type is shown in Fig. 2.

Three zones can be identified in these graphs (Fig. 3):

1. Initial zone: This zone is limited to the first 50 cycles of the 40 lb (178 N) load applied at the frequency of 6 APM. It is during this time that the seating of the sample occurs and which in certain instances seems to have caused appreciable disturbance. Small flattening of the sample ends was, in general, sufficient to prevent any further sample disturbance. It was not found possible to take this disturbance into account since its exact nature could not be established.

2. Intermediate zone: It is the zone after the initial zone and before the sample failure takes place.

3. Failure zone: The zone in which cracking and failure of sample occurs.

A number of samples showed less than the minimum measurable strength in that they broke at the application of first load. No identifiable zones could be established for such samples. For certain other samples, the failure zone and the initial zone were the same as the samples failed before 50 applications of 40 lb (178 N) load at 6 APM (Fig. 4, Curve A). For such samples it was difficult to assign any value for the diametral strain at failure. In a few cases, the failure of samples occurred just outside the initial zone (Fig. 4, Curve B), and thus for the samples of this category there was no intermediate zone. Some samples, especially those in the dry state, did not fail at all within the range of testing (Fig. 4, Curve C). For the samples of this type there was no failure zone. For most of the samples, however, the three zones could be identified in spite of the fact that distinction between the intermediate and the failure zone was not always very obvious.

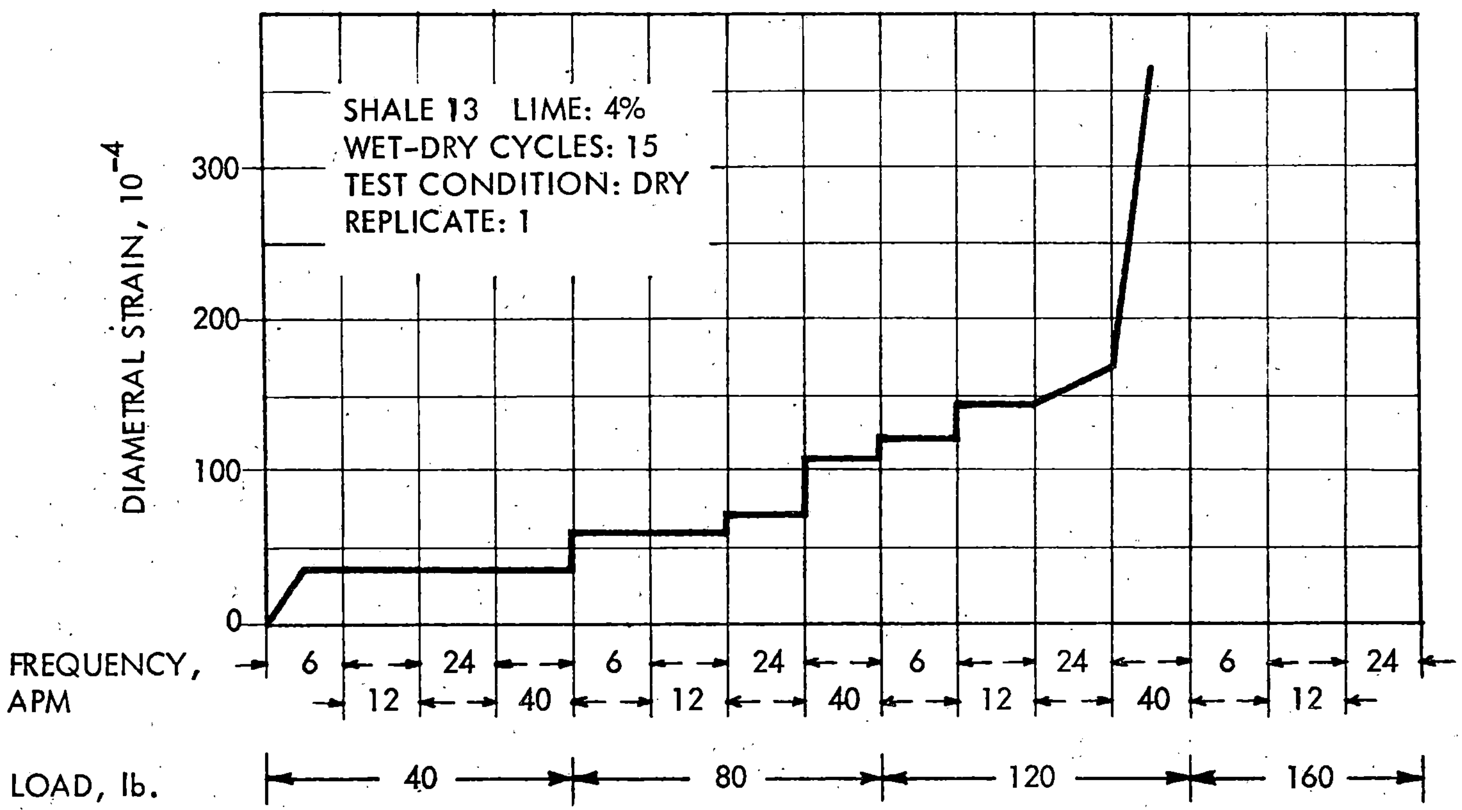

Figure 2. Example of diametral strain, load and frequency relationships for cyclic split tensile strength test.

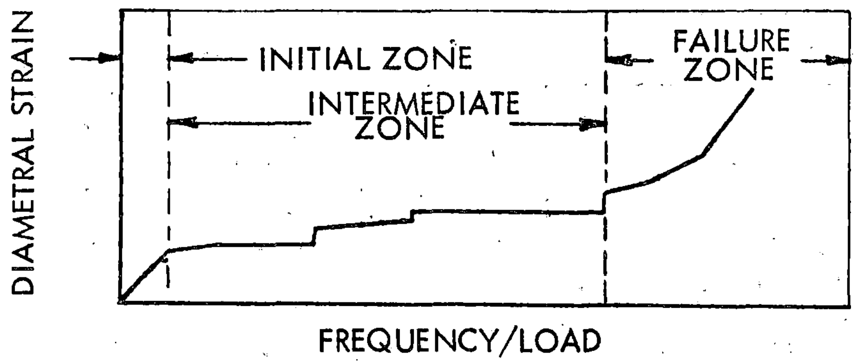

Figure 3. Three zones of a typical strain-frequency-load graph.

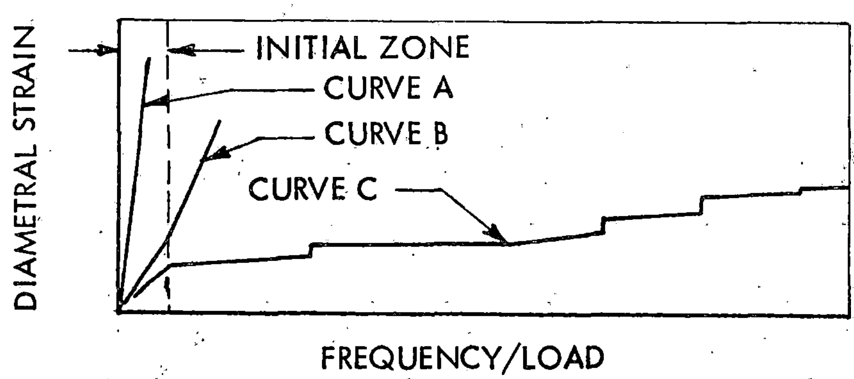

Figure 4. Examples of three common types of strain-frequency-load graphs.

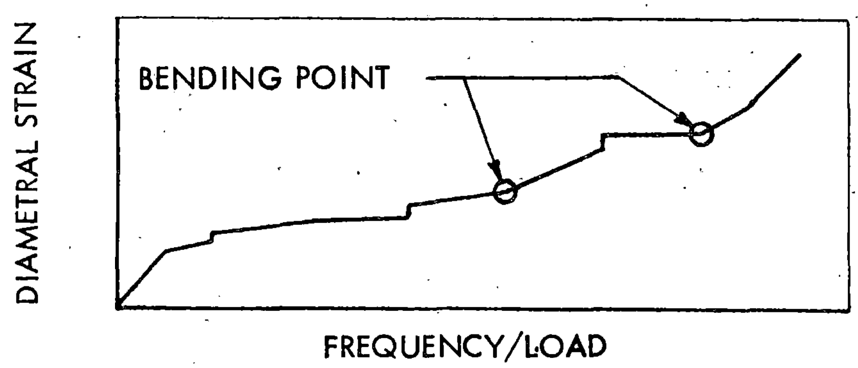

Figure 5. Example of bending points on strain-frequency-load graph.

For the full range of testing, the strain did progressively increase with the increasing number of load applications. There, however, were ranges during which the strain did not apprebiably change. This may be due partly to the difficulty in discriminating very small differences in deformation values on the graph and partly to the fact that indeed no appreciable deformation did occur in that range.

Within the intermediate zone, for a given load, initially the strain increased with a decreasing rate. This observation is similar to that of Moavenzadeh and Carnaghi (1966) on sand-asphalt beams. This trend in strains continues unless interrupted by a trend of the opposite nature in which the strain per cycle starts to increase with the increasing number of load applications (Fig. 5). It appears that the beginning of this trend of the "bending point" marks the initiation of crack formation and/or progressive failure within the specimen. Kawakami and Ogawa (1964) considered the bending point to indicate the beginning os slipping in their rheological model. In their study as well as in the present study a number of samples exhibited more than one bending point. This behavior may be attributed to step-strain phenomenon (Kawakami and Ogawa, 1964). If the trend of increasing strain per cycle does not reverse itself soon it is followed by the failure zone. On approaching the failure zone, either the cracks become visually identifiable or the failure strain exceeds a certain limit.

Initiation of cracking identifiable by visual examination was considered to be the start of failure. For the samples for which the intermediate zone was clearly identifiable, the failure was taken at the last bending point prior to rupture. Also, a sudden increase in the value of diametral strain by 0.0025 over any 50 load applications was considered as an additional criterion for failure. For the samples that did not fail within the range of testing, the end of test value was taken as the final diametral strain.

Shale 13

This A-7(6) shale contained 48 percent clay particles with montmorillonite as the predominant clay mineral (Table 3). In the case of humid samples not subjected to any wet-dry cycles, even small amounts of lime were sufficient to reduce the plasticity characteristics of the shale and introduce brittleness in the sample material. The diametral strain reduced from a high value of 0.0148 for the untreated shale to an average value of 0.0048 for the mix containing 1 percent lime. Further additions of the lime did not materially change either the strain magnitude or the capacity of the shale lime mix to take up greater stresses. For all wet-dry cycles, this effect of lime remained essentially the same. For the untreated shale, however, the effect of wet-dry cycles was significant; even 15 cycles reduced the high average value of 0.0148 for the diametral strain for zero wet-dry cycles to an average value of 0.0074. Not much difference was noticed in the deformation values of the sample after 15 wet-dry cycles.

With the increasing amount of lime and with the increasing number of wet-dry cycles, the humid state samples seemed to become more brittle.

TABLE 3. CLAY MINERAL COMPOSITION OF SHALES

Shale No.	2-micron Clay, %	Percent Clay Mineral[a]					Other[b] Silicate Minerals
		C	I	K	M	ML	
13	48	--	--	11	89	--	Q
15	14	22	38	40	--	--	Q,F
21	25	6	15	--	22	57	Q,F
24	14	--	68	28	4	--	Q,F

[a] C = Chlorite; I = Illite; K = Kaolinite; M = Montmorillonite; ML = Mixed layer Montmorillonite-Illite

[b] F = Feldspar; Q = Quartz

The sample behavior became significantly brittle even at 1 percent lime addition. At early stages of curing, this effect appears to be due to the utilization of lime for the rapid ameliorative effects without formation of any substantial amount of gel (Diamond and Kinter, 1966; Eades and Grim, 1966).

The behavior of dry samples was not much different from the samples of the humid state. However, the effects of lime treatment and application of wet-dry cycles were less pronounced. After one drying the diametral strain had an average value of 0.0112 for the untreated shale and a value of 0.0079 for the mix containing 1 percent lime. Application of wet-dry cycles was effective in reducing the average diametral strain from a value of 0.0112 after one drying to a value of 0.0074 after 5 wet-dry cycles for the untreated shale. No significant differences were noted for the treated and the untreated shales.

Compared to the samples in the humid state, the samples of dry state took greater loads and greater number of load applications. Also, the crack pattern development was slower in the case of dry samples. The behavior of the untreated shale under stress was very much elastic and its samples were able to withstand the total range of load and load applications. Addition of lime to the shale introduced brittleness and the samples failed under much smaller loads, fewer applications of loads and at smaller diametral strain. For zero wet-dry cycles the untreated shale showed an average diametral strain of 0.0112 at the end of test (160 lb - 712 N; 24 APM) while the samples containing 6 percent lime showed an average diametral strain of 0.0067 at the load of 80 lb (356 N) or 120 lb (534 N) and 12 APM frequency.

During the early stages of weathering (0 to 15 wet-dry cycles) the diametral strain showed an increase toward the 6 percent lime treatment level, probably due to the presence of excess unreacted lime in the mix.

Shale 15

In its natural state this A-1-b classification illite-kaolinite shale is associated with high bearing capacity and high permeability. These characteristics are lost on its being ground up and worked upon; its granular structure is broken down and the shale converted to a material of low strength exhibiting low plasticity characteristics. A similar finding is reported by Townsend et al. (1969) for lateritic soils. Addition of lime, in great percentages, perhaps could improve the stress-strain characteristics of this shale.

All shale and shale lime samples in humid state exhibited strengths below the minimum measurable for zero number of wet-dry cycles. Their diametral strain - load - load frequency graphs resemble curve "A" of Fig. 4. For the other wet-dry cycles only the samples of the untreated shale and those containing 6 percent lime showed any measurable strength. There is not much difference in these samples from the strength standpoint, all samples failing within the 40 lb (178 N) load zone. The untreated shale samples did not indicate any significant differences occurring due to the application of wet-dry cycles. At 6 percent lime content, the effect of wet-dry cycles was primarily in stabilizing the shale. Samples became more elastic with the increasing number of wet-dry cycles in that they exhibited smaller diametral strains.

Even though no significant changes were observed relative to the affect of the amount of lime on the diametral strain for the dry samples, the treated samples, as a trend, did show increased elasticity with the increasing percentage of lime; but the treatment became significant only at 6 percent level, and then also, it did not produce a material significantly different from the untreated shale.

Shale 21

Shale 21 had an AASHTO classification of A-6 and it contained 25 percent clay particles predominantly of montmorillonite-illite mixed layer and montmorillonite. The samples not subjected to any wet-dry cycles and containing 0 or 1 percent lime did not show any measurable strength; those containing more lime did. For the amounts of lime 2 percent or greater, the samples containing greater percentage of lime showed greater diametral strain. Since there had hardly been enough time for the shale-lime reaction to produce any pozzolanic materials, the strength gain could be due only to the flocculation of clay. The application of wet-dry cycles and subsequent humidification of samples seemed to have destroyed whatever bond formation took place, and the samples, even after 50 applications of wet-dry cycles, did not show any measurable strength.

The strength development that was destroyed by the humidification of the samples, became apparent on the removal of water from the shale lime mix. The untreated shale samples in the dry state exhibited plastic behavior in that they under went substantial deformation (diametral strains of 0.0151 to 0.0178 for the 120 lb - 534 N and 160 lb - 712 N loads respectively) but under high loads. Increasing amounts of lime made shale increasingly brittle, and the samples took less loads and failed at smaller average diametral strains.

The effect of wet-dry cycles was most significant on the untreated shale. Increasing number of wet-dry cycles made the material more brittle. As the amount of lime added was increased, the effect of wet-dry cycles became less significant. For the 2 percent and the 4 percent lime additions, no significant differences were observed. At 6 percent lime treatment level, the samples exhibited increasingly elastic behavior with the increasing number of wet-dry cycles.

Shale 24

This shale had an AASHTO classification of A-4 and it contained 14 percent clay basically of illite and kaolinite minerals. The addition of lime did not improve the stress strain characteristics of the samples. The flocculation of clay seems to have provided some plasticity to the shale in the initial stages; but the effect was completely lost when 30 wet-dry cycles had been applied and the samples had under gone humidification. Only the untreated samples showed any measurable strength for 30 and 50 wet-dry cycles.

The effect of lime in the initial stages was primarily to make the dry samples more plastic in that they showed greater strain per unit load. With increasing number of wet-dry cycles most samples were able to withstand slightly greater loads or greater number of load applications in the same load zone. Samples containing 1 percent lime, after 15 wet-dry cycles, showed average diametral strain of 0.0096 at 80 lb (356 N) load and 24 APM frequency. The differences for other samples were not so obvious. The diametral strain values for the untreated shale and the shale containing 1 percent lime increased when the number of wet-dry cycles increased to 5 from zero. Beyond this small region, however, there was no definite effect of the amount of lime or the wet-dry cycles on the diametral strain.

SUMMARY AND CONCLUSIONS

The purpose of this study was to investigate the effects of lime stabilization and simulated weathering on the repeated load split tensile strength characteristics of four shales from different locations in Oklahoma.

Shale numbers 15 and 24 had AASHTO classifications of A-1-b and A-4 respectively and contained predominantly illite and kaolinite clay minerals. These shales lost their desirable qualities as a foundation material on being pulverized. The stress strain characteristics of the pulverized shale did not show any significant improvement until the amount of lime added was 6 percent. Even at that treatment level, the material produced was not significantly different from the untreated shale, from the stress-strain standpoint. At 6 percent treatment level, the samples exhibited progressivley more elastic behavior with the increasing number of wet-dry cycles.

Shale number 21 contained predominantly montmorillonite and illite-montmorillonite mixed layer clay minerals and had AASHTO classification of A-6. This shale became amenable to lime treatment when the amount of additive was 2 percent or more. With the increasing amount of lime the sample behavior became progressively more brittle and the effect of weathering became less apparent. At 6 percent lime addition, however, samples of this shale also exhibited increasingly more elastic behavior with the increasing number of wet-dry cycles.

Shale number 13 contained predominantly montmorillonitic clay minerals and had AASHTO classification of A-7(6). With the increasing amount of lime and with the increasing number of wet-dry cycles, the samples became progressively more brittle. Even the samples containing 1 percent lime exhibited significantly brittle behavior. The effects of lime addition and weathering were more pronounced on the "humid" samples than on the "dry" samples.

Compared to the "humid" samples, the "dry" samples, in general and for all the shales, withstood greater stresses, and more number of stress applications; also the development of crack pattern was slower in them.

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NONDESTRUCTIVE TESTING OF ROADBED SOILS AND TRACK SUPPORT SYSTEMS

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ROADBED STABILIZATION : LIME INJECTION CONFERENCE

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NONDESTRUCTIVE TESTING OF ROADBED SOILS AND TRACK SUPPORT SYSTEMS

INTRODUCTION

1. Organizations within the Department of Defense (DOD), including the Waterways Experiment Station (WES), have been engaged for some time in planning facilities for launching spacecraft, for housing delicate electronic equipment, and for various types of heavy weaponry. The necessity to provide stable foundations for these facilities, and others which are subjected to dynamic loadings, has stimulated the development of improved techniques to measure dynamic properties of the in situ soil material. The most significant index of stability under dynamic loading is the ability of the foundation soil to resist deformation over a considerable area. The capacity to resist deformation at relatively low stress levels is expressed in terms of the elastic soil moduli, and in situ vibratory techniques have been successfully used to measure these moduli for design purposes. More recently, vibratory techniques have also been used in structural response and soil structure interaction studies.

BACKGROUND

2. Much of the basic theory and analytical work to develop an in situ vibratory method of measuring elastic soil moduli was first conducted at the Royal Dutch/Shell Laboratorium, Amsterdam, Holland.[1] Both the original vibratory equipment and technique have since been improved through field experience. The WES "vibroseismic" technique is described in the literature[2,3] and has been successfully employed for a wide range of applications, including studies on railway track support systems.[4] The use of vibratory equipment has also been extended to studies of structural impedance, stiffness, damping and other dynamic properties of interest. In response to a wide variety of requirements WES has developed controlled electromagnetic, hydraulic, and electro-hydraulic vibratory systems whose maximum force outputs range from a few pounds to more than 25 tons. In combination with conventional tests, the vibratory techniques described herein can provide a satisfactory understanding of structure/foundation behavior under dynamic loading conditions. This knowledge can be used, in turn, to optimize system performance.

PURPOSE AND SCOPE

3. This paper documents the application of WES in situ vibratory testing to railway track support systems. Included is a brief description of the techniques, equipment and procedures employed in testing at the Department of

Transportation (DOT) Kansas Test Track, as well as some examples of the results obtained in that study. The limited data presented also serve to document the procedures used in analysis and interpretation of results. Current planning for other studies in the railway transit field are briefly discussed.

WES IN SITU DYNAMIC TEST METHODS

General

4. The nondestructive tests described in subsequent paragraphs provide information needed for meaningful analyses of dynamic response characteristics of structural foundations. It is, of course, necessary to augment these data with input from other testing so that a comprehensive analysis can be performed. Depending on requirements, other testing may range from a minimum but essential series of conventional material property tests to an extensive test series involving both laboratory and in situ destructive and nondestructive testing. The latter may include, but are not necessarily limited to, plate bearing tests, field shear and penetration tests, laboratory uniaxial and triaxial tests on undisturbed or remolded specimens, and wave propagation tests such as crosshole and uphole seismic.

Vibroseismic tests

5. The WES vibroseismic method is based on measuring the propagation velocity of surface waves generated by controlled vibratory sources operating at discrete frequencies.[2,3] Determining wave propagation velocities over a range of frequencies provides a reliable means of deriving the soil elastic constants as well as their variation with depth.

6. The vibroseismic method, as the name implies, involves both vibratory and surface refraction seismic investigations. A primary consideration in the vibratory phase of the investigation is selecting vibrators whose frequency ranges and force outputs conform with the desired depth of investigation; generally speaking, low force output/high frequency vibrators are used for near surface (0-15 ft depth) measurements and high force output/lower frequency vibrators are used for deeper (15-200 ft) depths of investigation. Unfortunately, no single vibrator now available has the flexibility to operate at optimum force levels throughout the frequency range (about 5-300 Hz) used in various types of soils investigations.

7. When sustained vibrations are induced in a semi infinite homogeneous medium, waves propagate concentrically outward from the source. The shear wave velocity V_s is defined by:

$$V_s = \lambda f \tag{1}$$

where:

λ = wavelength

f = frequency of the induced vibration

From elastic theory, the shear wave velocity V_s is also defined by:

$$V_s = \sqrt{\frac{G_g}{\delta}} = \sqrt{\frac{G}{\rho}} \tag{2}$$

where:

g = acceleration due to gravity

δ = wet unit weight of the medium

ρ = mass density = $\frac{\delta}{g}$

G = shear modulus

8. In the vibratory test, frequency is controlled and wave length is the measured variable. The length of surface (Raleigh) waves is actually measured, however, for homogeneous soils with Poisson's ratio between 0.2 and 0.5 the difference in shear and Rayleigh wave velocities is less than 9 percent. For practical purposes, then, shear waves and Rayleigh waves may be considered to propagate at the same velocity. And, the shear modulus, G, can easily be computed from (2) above.

9. From elastic theory, Poisson's ratio for the soil can be computed using the ratio, V_r, between compression wave velocity, V_c, and shear wave velocity, V_s, by:

$$V_r = \frac{V_c}{V_s} \tag{3}$$

The value of V_c for the medium is derived from the surface refraction seismic phase of the investigation. These data are typically obtained along the vibratory line of investigation. Poisson's ratio ν is given by the expression:

$$\nu = \frac{V_r^2 - 2}{2\ (V_r^2 - 1)} \tag{4}$$

The compression modulus E (Youngs modulus) can then be calculated using the equation:

$$E = 2\ (1 + \nu)\ G \tag{5}$$

Based on WES experience, it appears that variations in shear modulus G correlate best with conventional exploration methods when it is assumed that the derived G values occur at a depth equal to one half the surface wavelength. Corresponding E values are plotted similarly.

10. In the refraction seismic phase of the investigation, data are taken with a commercially available 12 channel portable seismic unit. These data are plotted in graphic form as travel time of the compression wave from the source to geophones at various distances. The inverse slope of the line connecting the plotted points gives the compression wave velocity, V_c, through the soil media. A change in slope of the line shows that the wave has passed through an interface between two layers of different velocity. The first interface depth, D_1, can be calculated as follows:

$$D_1 = \frac{X_1}{2} \sqrt{\frac{V_{c2} - V_{c1}}{V_{c2} + V_{c1}}} \tag{6}$$

where:

X_1 = distance intercept from slope change

V_{c1} = compression wave velocity in first layer

V_{c2} = compression wave velocity in second layer

Depths to succeeding interfaces can be calculated using equations which are documented in the literature.[5,6]

11. Both forward and reverse refraction profiles are taken so that velocity variations caused by dipping layers can be corrected to the true velocity in each layer. The true compression wave velocity V_t in each layer can be determined from the equation:[5]

$$V_t = \frac{2\, V_u V_d}{V_u + V_d} \tag{7}$$

where:

V_u = apparent layer velocity moving updip

V_d = apparent layer velocity moving downdip

The true velocity and depths to interfaces derived from the seismic data can be used to construct a velocity profile of the subsurface media in the section of interest. These data, together with the E and G moduli derived from vibroseismic studies, serve as input to analytical models which can be used to predict the sections response to known forcing functions.

Impedance tests

12. Mechanical impedance may be defined as a quantitative measure of structural response to a defined vibratory input force. In the WES impedance test a vibrator is used to excite a structure with a sinusoidal force and structural response is measured with velocity transducers which are located at points of interest. While vibration can be described in terms of either acceleration or displacement, velocity measurements are usually preferred since stresses are believed to be more closely related to velocity.[7,8]

13. The sinusoidal force acting on a single degree of freedom system can be represented as a counterclockwise rotating phasor on the complex plane as shown in Figure 1(a). The complex representation of the force F is thus:

$$F = F_o \cos \omega t + j F_o \sin \omega t \tag{8}$$

The force F can also be expressed in terms of magnitude and phase angle[7] as:

$$F = F_o e^{j \omega t} \tag{9}$$

where:

F_o = phasor force magnitude (or length)

e = 2.718 = base of Naperian logarithms from the identity $e^{jz} = \cos z + j \sin z$

$j = \sqrt{-1}$

ω = angular velocity in radians

t = time

Similarly, when the force F is taken as the phase reference, the velocity V is described by the complex notation:

$$V = V_o e^{j\phi} \tag{10}$$

In this notation ϕ is the phase angle between force and particle velocity and equals the impedance angle θ. If the particle velocity V is measured at the point of application of the driving force F, then the ratio of force to velocity is mechanical driving-point impedance Z, defined as follows:[7]

$$Z = \frac{F}{V} \tag{11}$$

where:

F = applied force

V = resultant particle velocity in the direction of the force

The ratio of the driving force to the resulting velocity at other points in the system may also be expressed as a mechanical transfer impedance Z_{12} :

$$Z_{12} = \frac{F_1}{V_2} \tag{12}$$

where:

F_1 = force applied at one point

V_2 = resultant particle velocity at another point in the system

Thus, the most direct approach to determining impedance is to drive a point at a given frequency with a known force, measure the resulting vibration velocity, and then to compute Z from equations 11 or 12. This procedure is repeated at various frequencies until the response of the system has been adequately defined in terms of Z versus frequency. The impedance value obtained in this way at a particular frequency is the absolute magnitude of impedance, given by the equation:

$$Z = |Z|\, e^{j\theta} = R(Z) + j\, I(Z) \tag{13}$$

where:

$|Z|$ = the absolute value of impedance

$R(Z)$ = real part of impedance

$I(Z)$ = imaginary part of impedance

θ = impedance angle

A diagrammatic representation of impedance on the complex plane is shown in Figure 1 (b). The real part of impedance can be thought of as relating the applied force to the component of velocity that is in phase with it, and the imaginary component of impedance as relating the force to the component of velocity that is in quadrature with it. The impedance angle θ , which is equal to the angle ϕ depicted in Figure 1(a), is the phase angle between the force and velocity as shown in Figure 1(b).

14. An impedance analysis of this type can be performed on each part of a physical system; however, the method is adaptable to systems which are so complex that a detailed analysis is neither practical nor desirable. In many

instances, measurements at a few points of interest will suffice, and the physical properties of a linear system can conveniently be described in terms of three idealized elements having lumped constants. Idealized inertial, elastic and damping elements are conventionally represented as masses, springs, and dashpots, respectively. These elements are used in various combinations to represent the physical characteristics of the system and/or subsystems of interest. Mass, spring, and damping elements are usually identified with the symbols m, k, and c, respectively. A discussion of the properties of each element and their impedance characteristics is documented in the literature[7,8] and is beyond the scope of this paper.

15. However, an example pertinent to information presented later in this paper will be described. This system is a parallel arrangement of the idealized elements as shown in Figure 2(a). The impedance at the driving point is:

$$Z = c + j\left(2\pi f\, m - \frac{k}{2\pi f}\right) \tag{14}$$

where:

f = frequency

c = damping resistance

$j = \sqrt{-1}$

m = participating mass

k = spring constant

In this expression, c is the real part of impedance and the term

$j\left(2\pi f\, m - \frac{k}{2\pi f}\right)$ represents the imaginary part of impedance.

16. The absolute magnitude of impedance and the impedance angle are given by:

$$|Z| = \sqrt{c^2 + \left(2\pi f\, m - \frac{k}{2\pi f}\right)} \tag{15}$$

$$\theta = \tan^{-1} \frac{2\pi f\, m - \frac{k}{2\pi f}}{c} \tag{16}$$

These quantities, i.e., $|Z|$ and θ, for a parallel three element configuration, exhibit characteristic trends when plotted as shown in Figure 2(b) and 2(c). From the impedance plot in Figure 2(a) it can be seen that at low frequencies Z approaches the spring value, k, and the system is "stiffness" controlled. At high frequencies, Z approaches the mass value, m, and the system is "mass controlled". At resonance (the resonant frequency f_r where the impedance angle $\theta = 0$) the impedance Z is real and equal to the damping value of c.

17. Given that a set of impedance data have been acquired in the manner described it is apparent that the desired dynamic properties, i.e., stiffness, damping, participating mass, and resonant frequency, can be interpreted from the impedance plots. The measurement technique can conveniently be applied to as many points as are required to fully define the system dynamic response characteristics. Transfer functions, briefly mentioned in paragraph 12, can be used to determine the vibration transfer characteristics within the system.

18. The practical value of an impedance study is that the data serves not only to define the system response but may also be used to evaluate the effects of system variations, whether these be controlled or uncontrolled. For example, a controlled variation might consist of adding mass to the system; an uncontrolled variation might result from system degradation due to wear, weathering, or other causes. The change in system response resulting from such variations can be determined or predicted from impedance analyses.

TESTING AT THE DOT KANSAS TEST TRACK

General

19. The Santa Fe Railway, under the sponsorship of the Department of Transportation, has constructed approximately two miles of main line track near El Dorado, Kansas, for the purpose of evaluating various track support systems. The test track is comprised of nine different track support systems which are founded on an engineered embankment.[9,10,11,12] The embankment was designed and constructed under rigid controls in order to achieve the maximum possible degree of subgrade uniformity, so that meaningful performance comparisons of track support systems could be made. The approximate 8-ft thick embankment was built from mixed residual clays and was founded on limestone bedrock. In the final stage of construction, the embankment received a nominally 6-in.-thick lime stabilized surface layer which was topped with a sprayed asphalt membrane. This treatment, together with a spreading of Santa Fe slag ballast, was undertaken to protect the embankment from weathering in the winter season. The slag ballast was used for track support throughout the length of the test track.

Pretraffic studies

20. The pretraffic studies were conducted in two phases; the first phase consisted of a vibroseismic investigation[4] of the embankment prior to installation of the track support systems. The vibroseismic study was conducted in a two week period beginning 28 September 1971; all nine sections of the two mile long embankment were tested. In the second phase of the pretraffic studies, a series of mechanical impedance tests were also conducted on the nine installed

track support systems. Data acquisition was completed during a two week period in April, 1973, just prior to opening the track for traffic.

21. The nine test sections are grouped in two general categories, i.e., cross-tie systems and non-conventional continuous reinforced concrete systems. Within the tie grouping, various tie spacings, ballast thicknesses and/or treatments were utilized for performance comparisons. The non-conventional reinforced concrete systems were built in two different configurations and two types of construction were used for one of these configurations. The nine sections and their construction details are summarized in Table 1.

22. During the vibroseismic investigation, the vibratory tests were run on top of the embankment; the refraction seismic tests were made along the flank of the embankment to prevent the relatively high velocity lime stabilized layer from influencing test results. The presence of the lime stabilized layer had no adverse effect on the vibratory data.

23. Results of the vibroseismic testing conducted on each section of the embankment are summarized in Table 2 together with the results of other pre-traffic testing covered later in this paper. A summary plot of average shear wave velocity versus depth in the embankment is presented in Figure 4; average elastic moduli plotted versus depth in the embankment are shown in Figure 5. The appreciable variation in elastic moduli at a depth of 1 ft, as indicated in Figure 3, is undoubtedly due to the presence of the lime stabilized layer. The fact that sections 5, 7, 8, and 9 exhibit relatively high moduli at 1 ft depth, while the remaining sections do not, suggests some nonuniformity in thickness and/or strength of the lime stabilized layer over the length of the embankment.[4]

24. In the second phase of pretraffic testing, impedance tests of the track systems were carried out using the hydraulically operated, counter rotating, eccentric weight vibrator shown in Figure 6. The vibrator hydraulic system was driven by a gasoline powered variable displacement pump; the variable displacement feature served to provide frequency control. This vibrator had been modified from its previous usage by adapting it to a 2-in.-thick steel baseplate which spanned both rails of the test track. A clamping mechanism, consisting of the butt-block and wedge arrangement shown in Figure 7, was provided at each corner of the baseplate so that the vibratory apparatus could be securely fastened to the railheads at four points. The total weight of the vibrator with baseplate attached is 2855 pounds, and the vibrator can produce dynamic forces of up to 4000 pounds in the frequency range from 1 to about 50 Hz. The vibrator may be operated in either the vertical or rocking mode simply by changing positions of the eccentric weights, as shown in Figure 8.

25. Forces applied to the track system were measured with load cells which were located at each corner of the baseplate. Prior to testing, the cells were clamped (preloaded) between the baseplate and railheads by means of the butt-block and wedge arrangement provided. Output from the four load cells was summed to give the total force applied during vertical excitation. The output from the pair of cells on each rail was summed separately to give the force applied in the rocking mode.

26. Vertical particle velocities at various points on the track support system and on the embankment were measured with the instrumentation array shown in Figure 9. The transducer locations shown in Figure 9 are all referenced to the centerline of the vibrator, as shown in Figure 10. The transducer locations were selected to provide both vertical and longitudinal measurements of track and embankment response. With the vibrator and instrumentation array in place, vertical mode testing was begun by sweeping the frequency range from 1 to 50 Hz with 14 oz eccentric weights on the vibrator. Next, the frequency range from 1 to 15 Hz was swept using 10 lb eccentric weights. The same pattern was followed for testing in the rocking mode. All of the data were recorded on broad band magnetic tape; data reduction was accomplished with the instrumentation system diagrammed in Figure 11.

27. The impedance test procedure outlined was dictated primarily by the frequency and force output limitations of the vibrator employed, i.e., at low frequencies it was necessary to substitute larger weights to achieve the force output necessary to excite the system. And, the vibrator could not be driven to frequencies higher than about 50 Hz. These and other limitations of the vibratory system used were recognized prior to embarking on the test program, however, time and funding constraints precluded further refinements and/or additions to the test apparatus. Ideally, the test would have been conducted by sweeping the frequency range from 1 to, say, 150 Hz at a constant force level, and the vibrator would have been fastened to each rail at one point rather than two. However, the basic intent of the testing was to demonstrate the feasibility of the impedance approach and to provide data for comparative analyses of the various track support systems. Some examples of the test results obtained are presented in subsequent paragraphs. Only vertical mode results will be discussed because results from rocking mode tests proved to be similar.

28. In Figure 12, driving point mechanical impedance Z test results are presented for test track sections 1, 2, and 3. Driving point impedance in this test configuration was measured at the rail flange, location No. 1 in Figure 9. Figure 13 presents similar test results for test sections 4, 5, and 7. The driving point mechanical impedance Z is plotted versus frequency on log-log scales in both figures. In addition, stiffness, k , and mass, m , scales are included on the plot. These scales are provided to facilitate interpretation of the spring (stiffness) controlled and mass controlled portions of the impedance plot. As shown in Table 1, the nominal difference between sections 1, 2, and 3 is the spacing used between the precast concrete ties; since section 3 has the closest tie spacing (24 inches), intuition suggests that it should also have the greatest initial stiffness. The curves in Figure 11 are in good agreement with the previous assumption for the frequency range from 4 to 10 Hz, and indicate a general trend of stiffness increasing as tie spacing decreases. An apparently anomalous response may be noted for sections 1, 2, and 3 at frequencies above about 15 Hz; the reasons for the apparent inversion in stiffness relative to results obtained at lower frequencies has not been established. This phenomenon may be associated with the variation in force levels which was caused by changing the eccentric weights; unfortunately, a continuous frequency sweep at a constant force level, which might have clarified the phenomenon, was not possible for reasons cited previously. Nevertheless, the initial, or

"spring" controlled portion of the impedance plot in Figure 12 can be used to determine the pretraffic vertical stiffness of track sections 1, 2, and 3. The apparent stiffness values for sections 1, 2, and 3, in the frequency range from 4-10 Hz, are 7×10^4 lb-in., 4×10^5 lb-in., and 1×10^6 lb-in., respectively. Stiffness values derived similarly for all of the track sections are tabulated in Table 2, together with average subgrade dynamic properties determined from Phase 1 testing.[4] Referring to Figure 13, it will be seen that driving point impedance results from Sections 4, 5, and 7 are in good agreement and exhibit an initial dynamic stiffness of about $2.5 - 3 \times 10^6$ lb-in. The track support systems in sections 4 and 5 are continuous case in place beams and slabs, respectively, while section 7 is comprised of precast concrete beams which were joined in the field, as per the descriptions in Table 1. These systems were designed to have identical structural stiffness.[10]

29. Comparing Figures 12 and 13, it can be seen that impedance in the lower frequency region, from 5 to about 20 Hz, is stiffness controlled. Above 20 Hz the character of the impedance curves changes, indicating that the test sections typically resonate at frequencies between, say, 25 and 40 Hz. The curves for driving point impedance Z at the rail flange, as presented, are generally similar in character to the classic curve derived from the parallel three element system described earlier in this report. Such a system is stiffness controlled at low frequencies, passes through resonance at an intermediate frequency, and is mass controlled at higher frequencies. The phase angle, θ , between force, F , and velocity, V , for the stiffness controlled portion of the impedance plot should thus be -90 degrees at low frequencies, and should increase to +90 degrees at the higher frequencies which encompass the mass controlled portion of the plot. Figure 14 shows a typical phase angle, θ , versus frequency plot which was derived from the test data taken in test section 7. It is apparent that these results closely follow the data trend predicted for a three element system, and that resonance occurs at approximately 40 Hz. Results from other test sections exhibit similar trends. Had it been possible to excite the sections with frequencies beyond 50 Hz, which was not desirable because the vibrator would have produced upward accelerations greater than 1 on the rail, resonance(s) and anti-resonance(s) could have been better defined, as well as the mass controlled portion of the impedance curve which is assumed to exist at higher frequencies. It is possible to estimate a value for the participating mass (consisting of the vibrator, rail, tie, ballast and embankment masses being excited) from the plot in Figure 13. Assuming that the impedance curve is symmetrical about the resonant frequency line, the participating mass in the case of track section 7 should be on the order of 2 to 3×10^4 pounds, and the damping impedance, c , at resonance is clearly between 5 and 6×10^4 lb-in. A proposed second generation vibratory apparatus for impedance testing is discussed later in this report; this equipment will have the capability to operate at higher frequencies and at a constant force level in order to fully define the impedance curve for the frequency range of interest.

30. The data obtained on the test track sections were also used to derive velocity transfer ratios R_{1-2} which are defined by the formula

$$R_{1\rightarrow 2} = \frac{V_1}{V_2} \qquad (17)$$

where:

V_1 = particle velocity measured at some point in the system

V_2 = particle velocity measured at another point further from the source of excitation.

This ratio is an index of the impedance (or attenuation) occuring between any two measurement points in the system. Figures 15 and 16 present typical transfer ratio versus frequency curves derived from data obtained in test sections 8 and 9, respectively. The top curve in Figure 15 shows a transfer ratio R_{1-2} of nearly 1 in the frequency range from 12 to about 50 Hz; this shows that the rail to tie fastener was tightly clamped and was presumably in good condition because the rail and tie moved in concert. The R_{1-5} and R_{2-5} curves in Figure 15 are nearly identical, as they should be since the rail and tie moved essentially as one unit; the attenuation between curves R_{1-2} and curves R_{1-5} and R_{2-5} is thus contributed by the tie-ballast-embankment interaction and properties which lie between measurement location 2 (on the tie) and location 5 (on the embankment at the bottom of the ballast). The least attenuation, i.e., highest transfer ratio, for curves R_{1-5} and R_{2-5} occurs at a frequency of about 26 Hz. This is the approximate resonant frequency of track section 8. Similar results for points lower on the embankment are shown in Figure 16; these data were obtained in track section 9. The least attenuation in curves R_{5-6} and R_{5-7} occurs at about 33 Hz; this frequency coincides with the resonant frequency which was determined for section 9 (from impedance and phase angle plots which are not presented).[13]

31. A more detailed analysis of the data will be presented at a later date in the report documenting impedance studies at the Kansas Test Track.[13] The few examples of impedance results shown herein merely serve to illustrate the viability of the technique and to provide some insight into methods of interpretation.

Planned post traffic studies

32. Planning for post traffic testing at the Kansas Test Track is now underway at WES. Primary emphasis is currently being placed on a series of tests to assess post-traffic track system response and ballast and embankment properties, since some problems have resulted from the rail traffic to date. This investigation will include both conventional laboratory testing and in situ vibratory testing.

33. Also being included is a series of impedance tests to evaluate the traffic related performance of the various track support systems. Experience

with the pretraffic impedance testing is being used to develop a second generation impedance testing system; the package envisioned will be more convenient and efficient to operate, and will also enjoy the advantages of increased flexibility and performance. Under consideration is an existing WES mobile test package as well as other commercially available equipment which is compatible with the purpose. The choice of a final design will naturally be influenced by economic considerations, however, portability and performance will also be stressed in anticipation of expanded use of the impedance testing technique.

WES SOIL STABILIZATION RESEARCH

General

34. For more than a dozen years WES has been actively engaged in soil stabilization research for military and civil projects. During this period a number of stabilization techniques, including stabilization by the addition of a cementing agent, have been applied to troublesome foundation materials. More recently, other investigators have developed and successfully used chemical injection techniques to achieve stabilization. In terms of its application to railway transit systems, the injection process offers a cost attractive and expedient means of stabilizing track support systems so that derailments and the associated costs for preventive maintenance can be minimized.

35. Although the injection stabilization technique has generally proven successful, there have been instances where the desired results have not been obtained. Hence the need for further research to determine which chemicals and techniques are most effective in stabilizing various soil materials, and to develop a nondestructive test method with which stabilization effectiveness can be evaluated.

Previous stabilization studies

36. During April 1964, a special test section was constructed with the specific purpose of evaluating, by dynamic test techniques, the effectiveness of soil stabilization in order to develop design criteria for stabilized soil.[14] The purpose for conducting the nondestructive surface vibration tests was twofold: first, to compare the vibratory method of determining certain soil characteristics with conventional static methods, and secondly, an opportunity was afforded to evaluate the vibration method on a controlled test section. The section consisted of two traffic lanes, each 10 ft wide and 120 ft long. Subgrade for both lanes was a 31 in. layer of heavy clay having an 8 to 10 CBR. The tests were designed in such a way that the two lanes were divided into three equal panels 40 ft in length; each panel was treated with a different percentage of Portland Cement.

37. Vibroseismic testing in the above program was conducted as outlined earlier in this paper, and both shear (S) and compression(P) wave velocities were derived, as a function of depth, before and after stabilization. Shear

wave velocities averaged about 600 fps in the untreated material. After stabilization with a 6 percent Portland Cement treatment the shear wave velocity increased to about 900 fps, which corresponds to a 225 percent increase in shear modulus, G , from the untreated condition. Typical test results are shown in Figure 16 for the treated and untreated material tested. The foregoing serves to illustrate that the vibroseismic technique is sensitive to relatively small changes in in situ conditions, and that the technique is well suited to defining such changes even in relatively thin, near surface layers.

Lime injection program

38. Discussions relative to a lime injection test program(s) have been conducted between representatives of the DOT, WES, and the University of Arkansas. As a result of these discussions, WES has presented a proposal for testing on lime injected material. The objectives of the proposed test program will be:

a. To determine the most sensitive and accurate suite of geophysical and vibratory test methods to detect changes in material properties resulting from lime injection.

b. To determine the effectiveness of lime injection stabilization by evaluating the soil characteristics in a prepared test section before and after treatment.

39. A comprehensive series of conventional laboratory material property tests and various geophysical and vibratory tests are planned for the prepared test section before and after lime injection. These data will be used to assess the effectiveness of the injection process and, through cross correlation techniques, will be used to select the most definitive suite of test techniques. It is believed that this program will provide the information necessary to predict stabilization effects from a range of lime injection treatments.

40. If the proposed test program is adopted, and if the anticipated results are achieved, WES will participate with the University of Arkansas in testing the effects of lime injection on operational track support systems.

SUMMARY AND CONCLUSIONS

41. Derailments, and the cost of track maintenance to prevent such occurences, constitute a major problem for railway transit systems. The higher train speeds now envisioned will certainly require the design of new track support systems with improved performance characteristics. The DOT has recognized the requirement for improved track support systems and already has a study underway at the Kansas Test Track.

42. In order to determine the most efficient and economical design(s) for operating rail systems, and to provide a continuing index for maintenance requirements, performance evaluations will have to be conducted at regular intervals. The condition of existing trackage will have to be evaluated at some future time before scheduling repairs and/or replacement on a priority basis. Research programs directed towards expedient means of improving existing track

systems offer attractive possibilities, providing the effect of such treatments can be accurately assessed.

43. The test results presented herein, although limited in scope, clearly indicate that the vibratory techniques described are a viable means of evaluating track support system performance. The test techniques are expedient and non-destructive, and their results can be used to make meaningful comparisons to track support system condition and/or performance characteristics. Dynamic properties of interest, including system stiffness, damping, participating mass, resonant frequencies, elastic moduli of the soil materials, etc., can be assessed from the test results, and these data are required for a comprehensive rational analysis of dynamic response.

44. Based on the foregoing, it is reasonable to conclude that the vibratory techniques described can contribute valuable information in the evaluation and research phases of railway transit system studies on track support systems. And, further developments leading to the increased mobility and flexibility of test equipment will enhance the present capability to determine significant dynamic properties.

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TABLE I

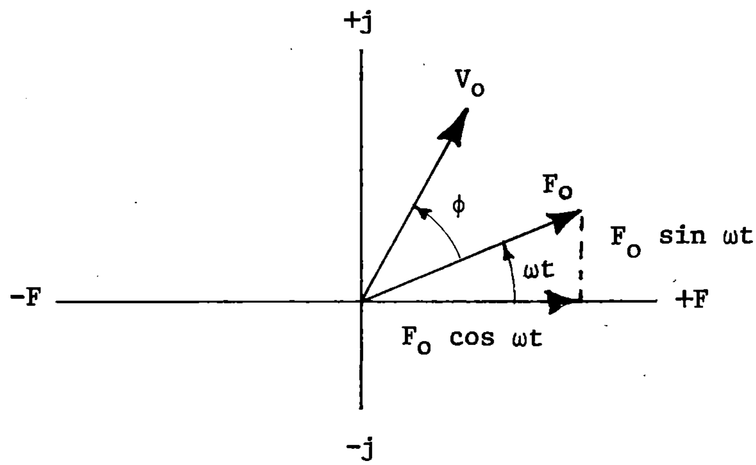
TEST SECTION CONFIGURATION

Section Number	System Type	Ballast Depth	Remarks
1	Concrete ties, 30 in. C-C	10 in.	
2	Concrete ties, 27 in. C-C	10 in.	
3	Concrete ties, 24 in. C-C	10 in.	
4	Continuous concrete beams	6 in.	Cast in place structure
5	Continuous concrete slab	6 in.	Cast in place structure
6	Wood ties, 19.5 in. C-C	10 in.	6 in. stabilized ballast layer on subgrade
7	Continuous concrete beams	6 in.	Precast beams, installed and field joined
8	Concrete ties, 27 in. C-C	15 in.	
9	Wood ties, 19.5 in. C-C	10 in.	Control section (standard Santa Fe)

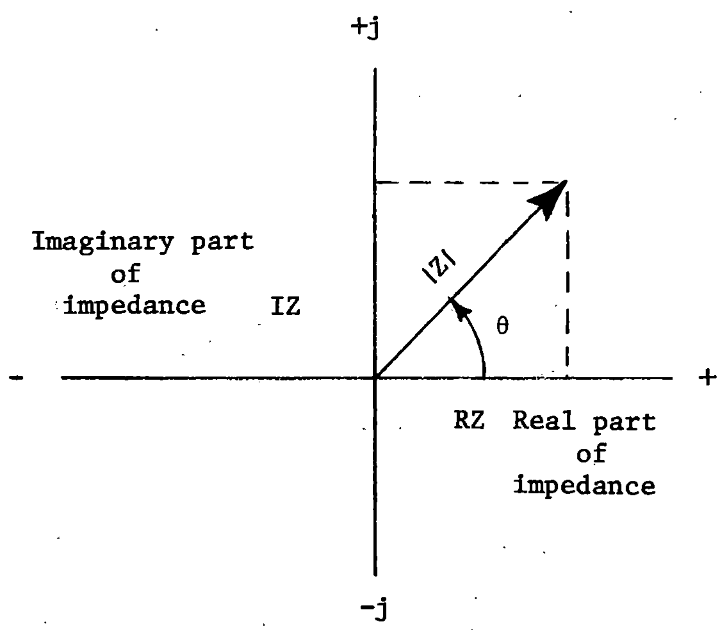
TABLE II

DYNAMIC PROPERTIES OF TEST SECTIONS

Section Number	Measurement Parameter (Vertical Velocity)	Initial Dynamic Stiffness, lb-in	Average Subgrade Properties			
			Total Depth		Top 4 ft	
			E, ksi	G, ksi	E, ksi	G, ksi
1	Rail flange-point impedance	7×10^4	24.3	8.7	20.0	7.1
2	Rail flange-point impedance	4×10^5	25.8	9.3	23.0	8.2
3	Rail flange-point impedance	1×10^6	22.8	8.1	19.0	6.8
4	Rail flange-point impedance	2.5×10^6	21.7	7.8	19.2	6.9
5	Rail flange-point impedance	2.5×10^6	22.4	8.0	20.5	7.2
6	Rail flange-point impedance	1.6×10^5	23.9	8.8	19.0	7.0
7	Rail flange-point impedance	3×10^6	31.7	11.4	25.0	8.5
8	Rail flange-point impedance	1×10^6	24.9	8.9	21.0	7.5
9	Rail flange-point impedance	5×10^5	26.1	9.3	21.0	7.5

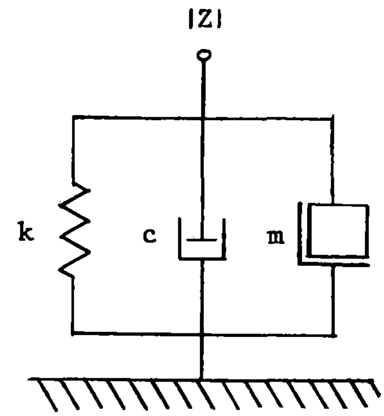


(a) Force phasor

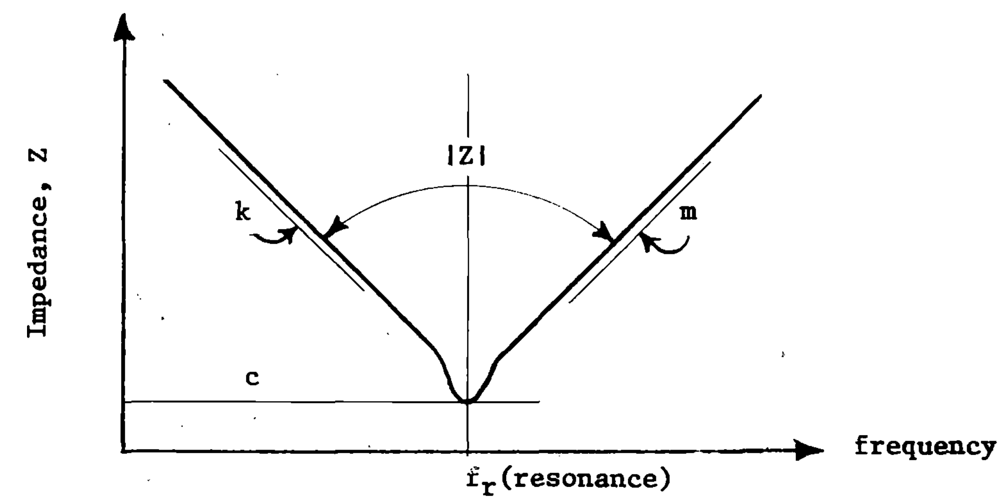


(b) Impedance phasor

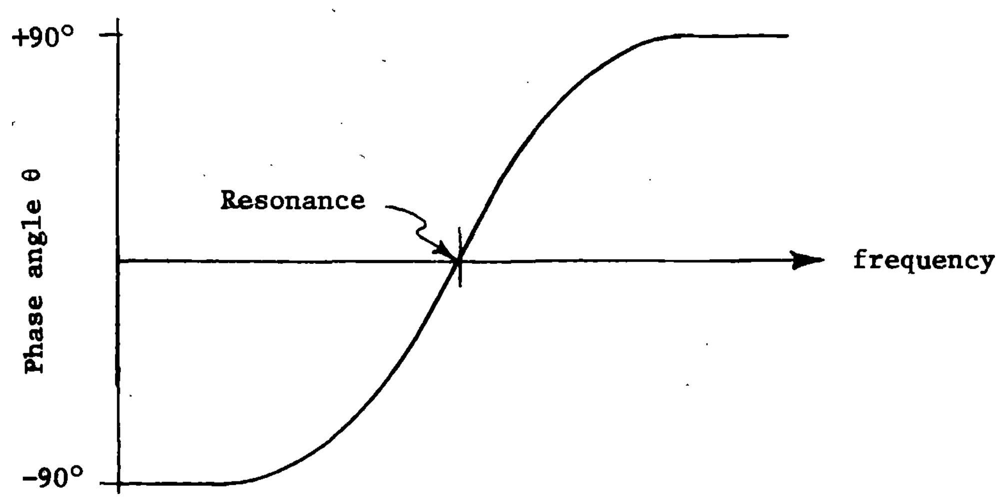
Fig. 1. Phasor representations of force and velocity in the complex plane



(a) Parallel three element linear system

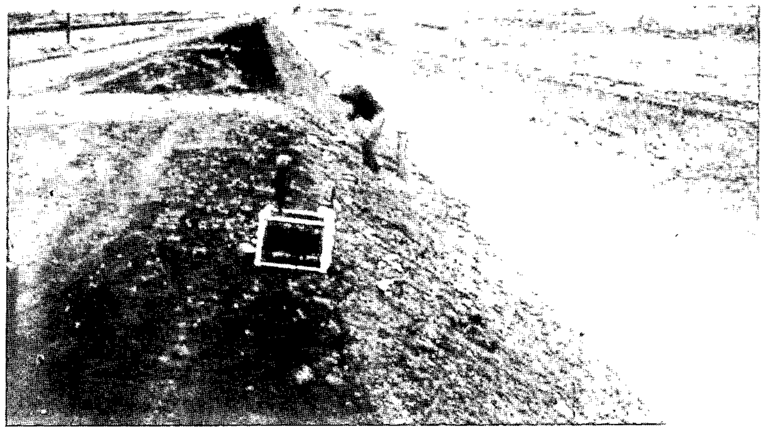


(b) Impedance versus frequency (log-log plot)

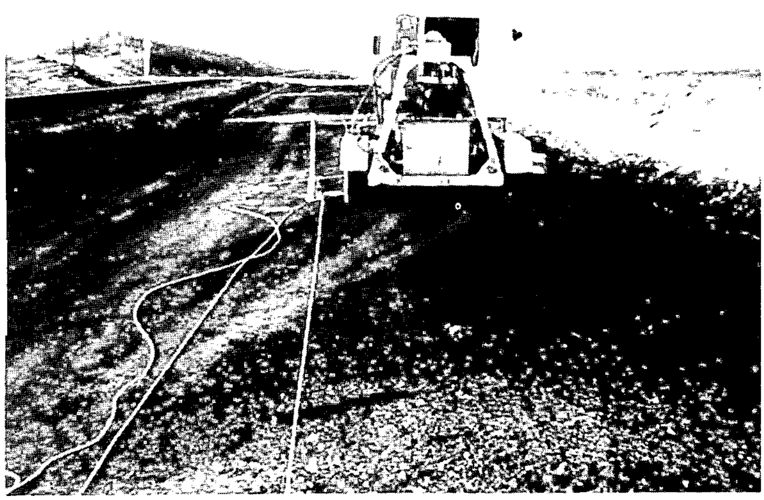


(c) Phase angle θ versus frequency

Fig. 2. Idealized representation of three element parallel system



Refraction seismic test setup along
side of, embankment



Equipment in position for vibratory test

Fig. 3. Typical Phase I Pretraffic vibroseismic tests in progress at the DOT Kansas Test Track

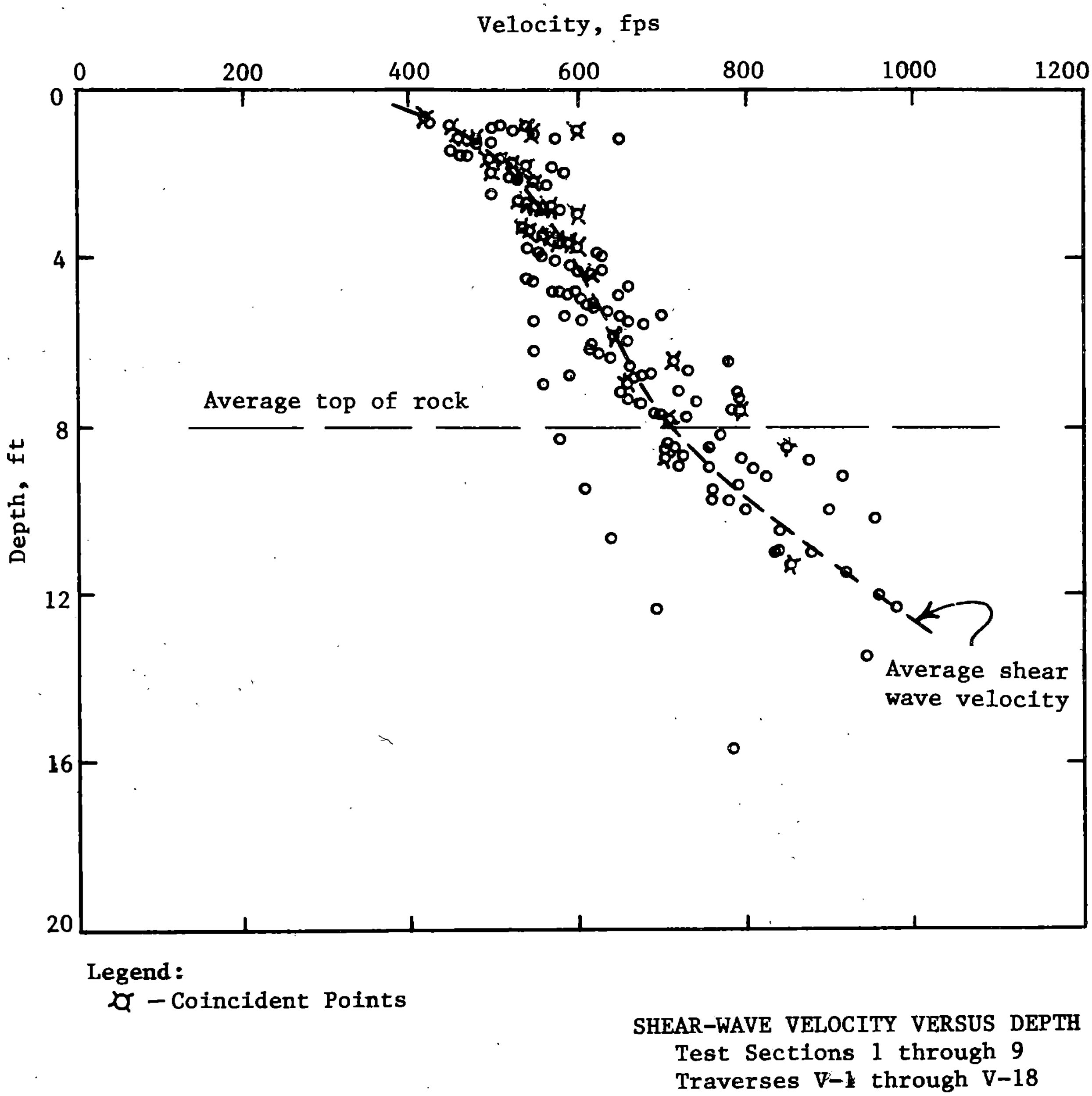
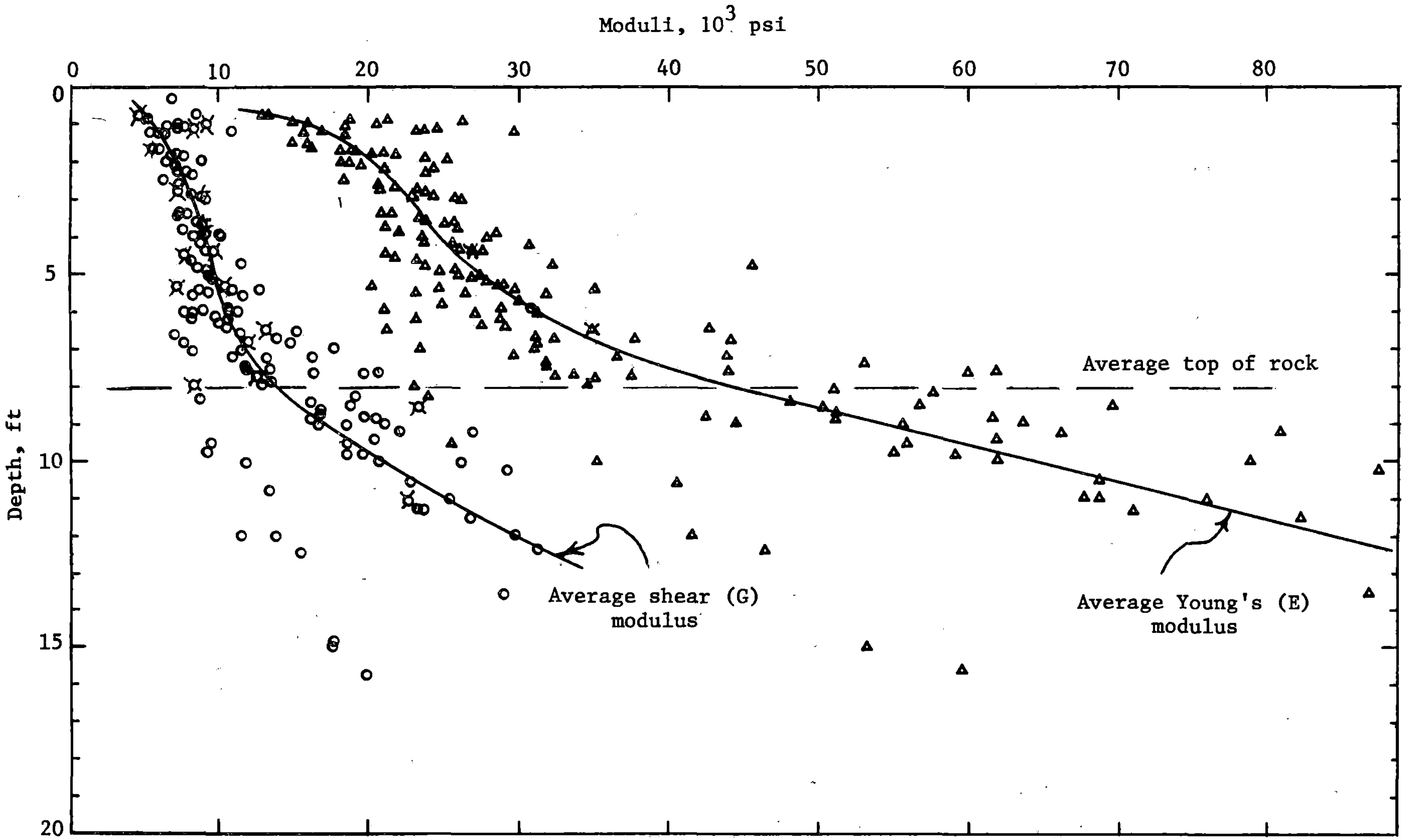


Fig. 4. Plot of average shear wave velocity versus depth in the embankment from pretraffic vibroseismic tests



<u>Legend</u>:

○ Shear Modulus, G

△ Compression Modulus, E

⌑ Coincident Points

MODULI VERSUS DEPTH
Test Section 1 through 9
Traverses V-1 through V-18

Fig. 5. Plot of average E and G moduli versus depth in the embankment from pretraffic vibroseismic tests

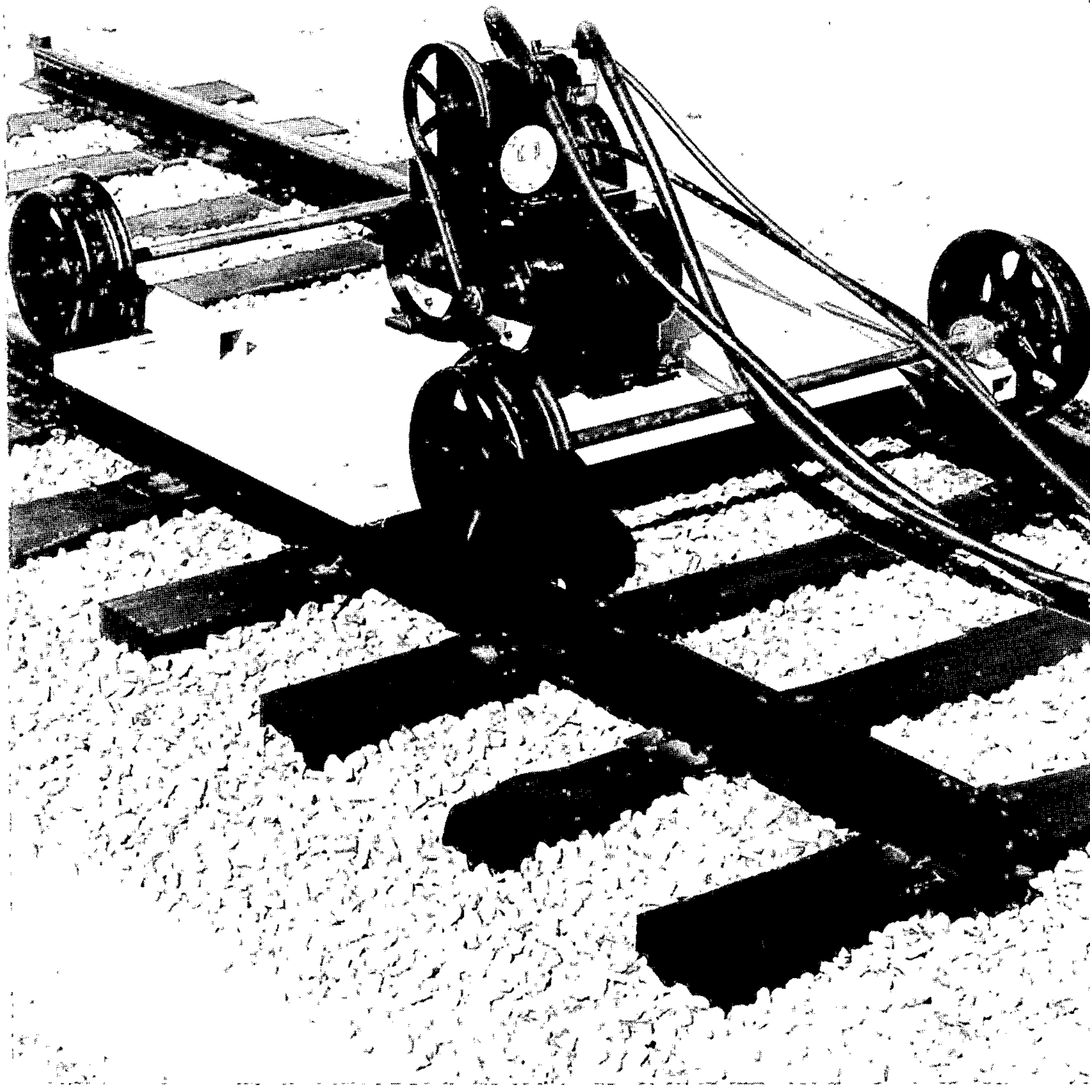


Fig. 6. Vibratory apparatus used for impedance testing

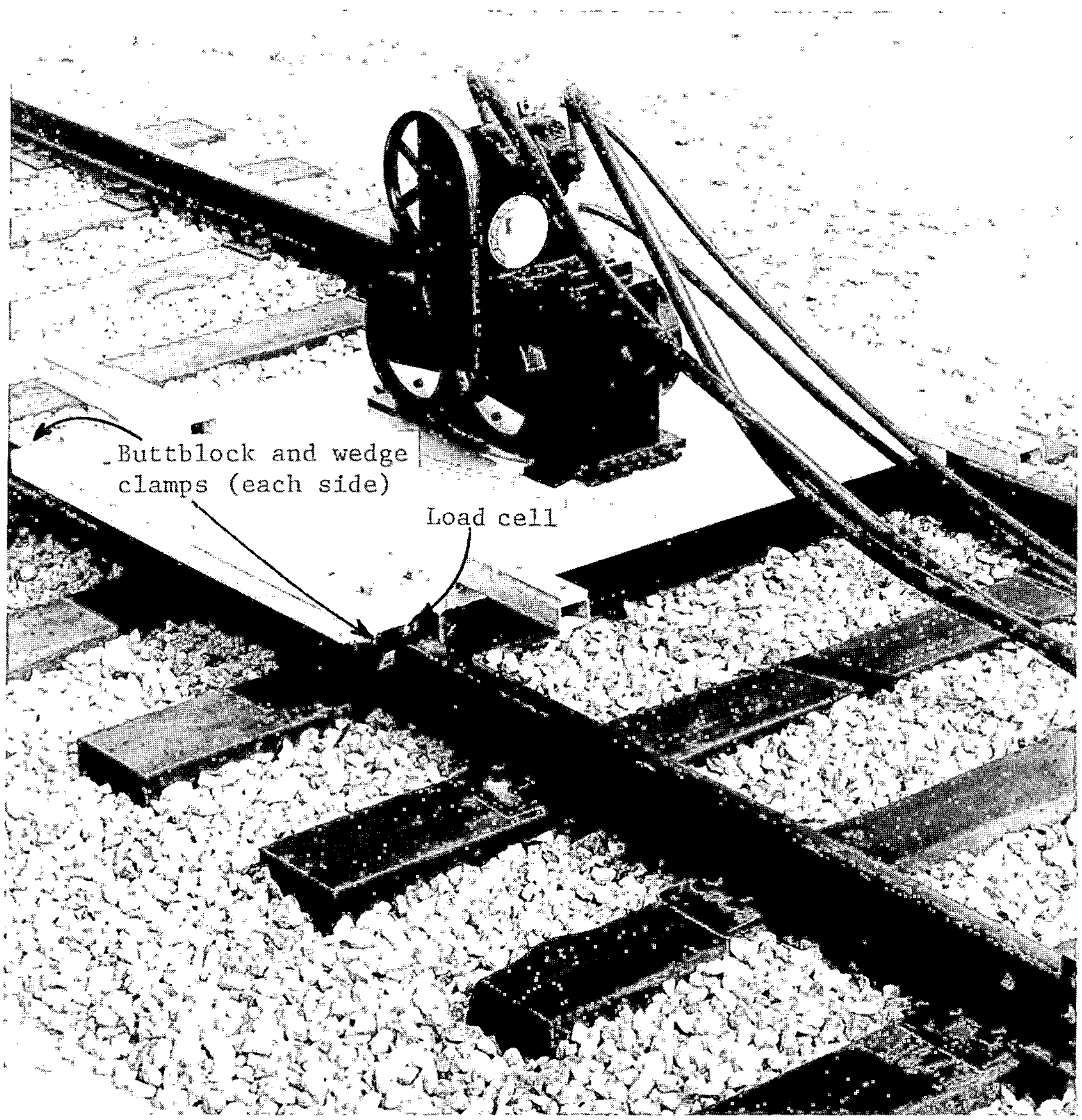


Fig. 7. Vibrator fastened to railheads and configured for testing

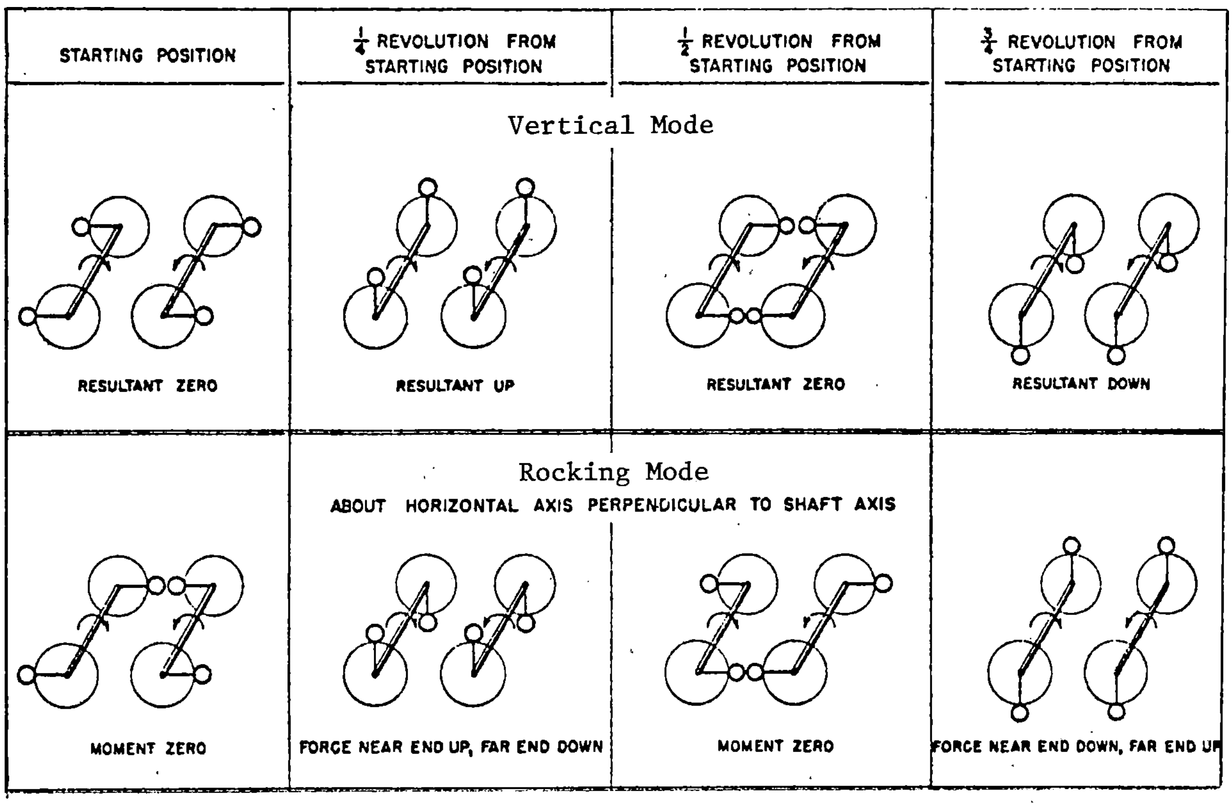


Fig. 8. Eccentric weight configurations for vertical and rocking modes of excitation

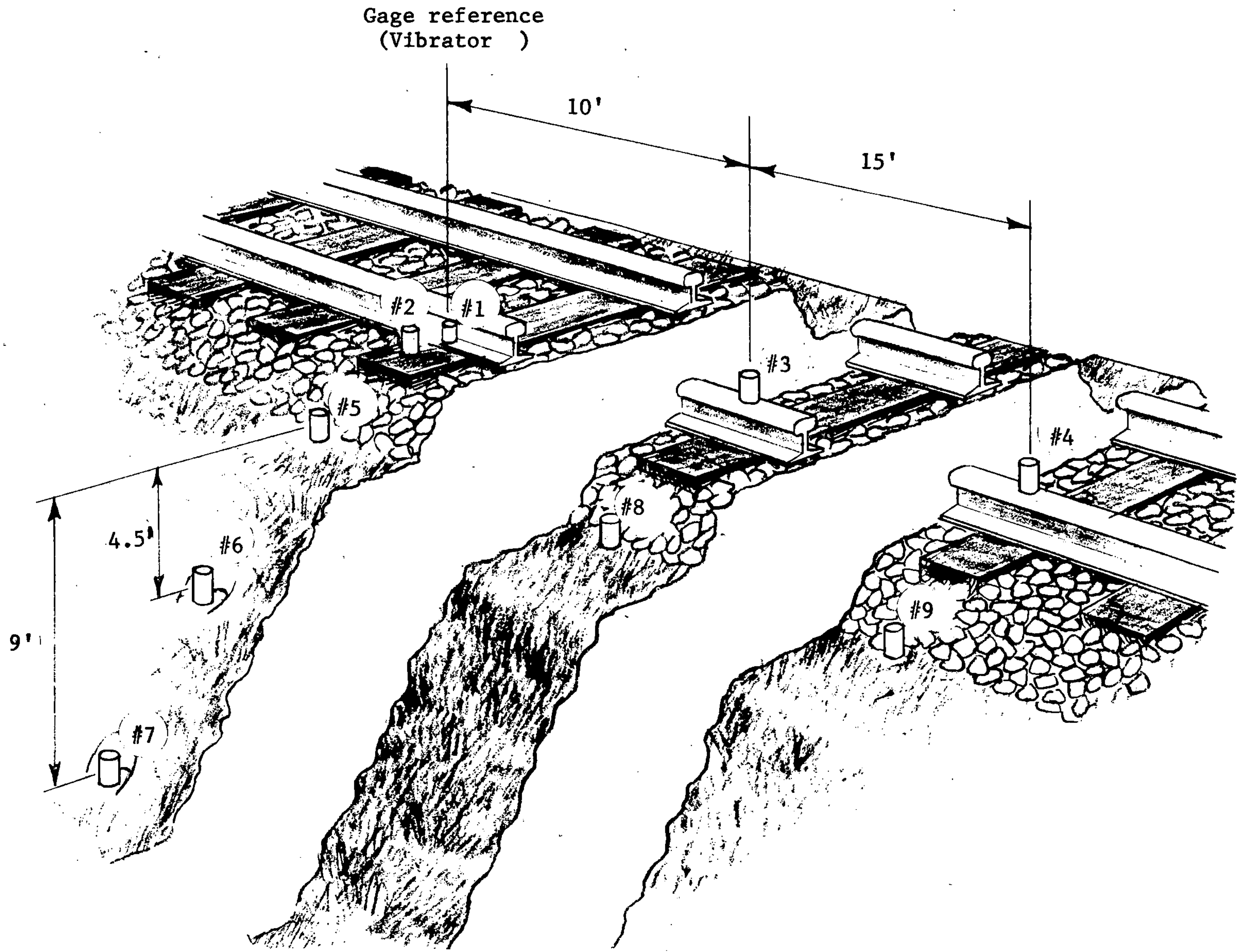


Fig. 9. Typical instrumentation array

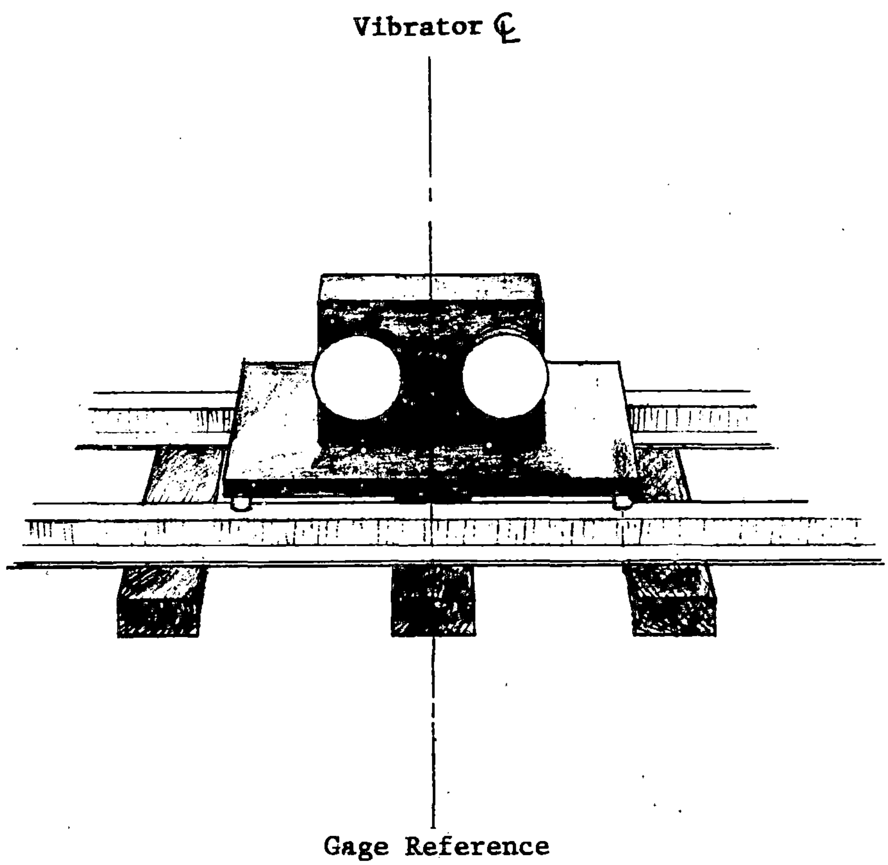


Fig. 10. Reference line for instrumentation array

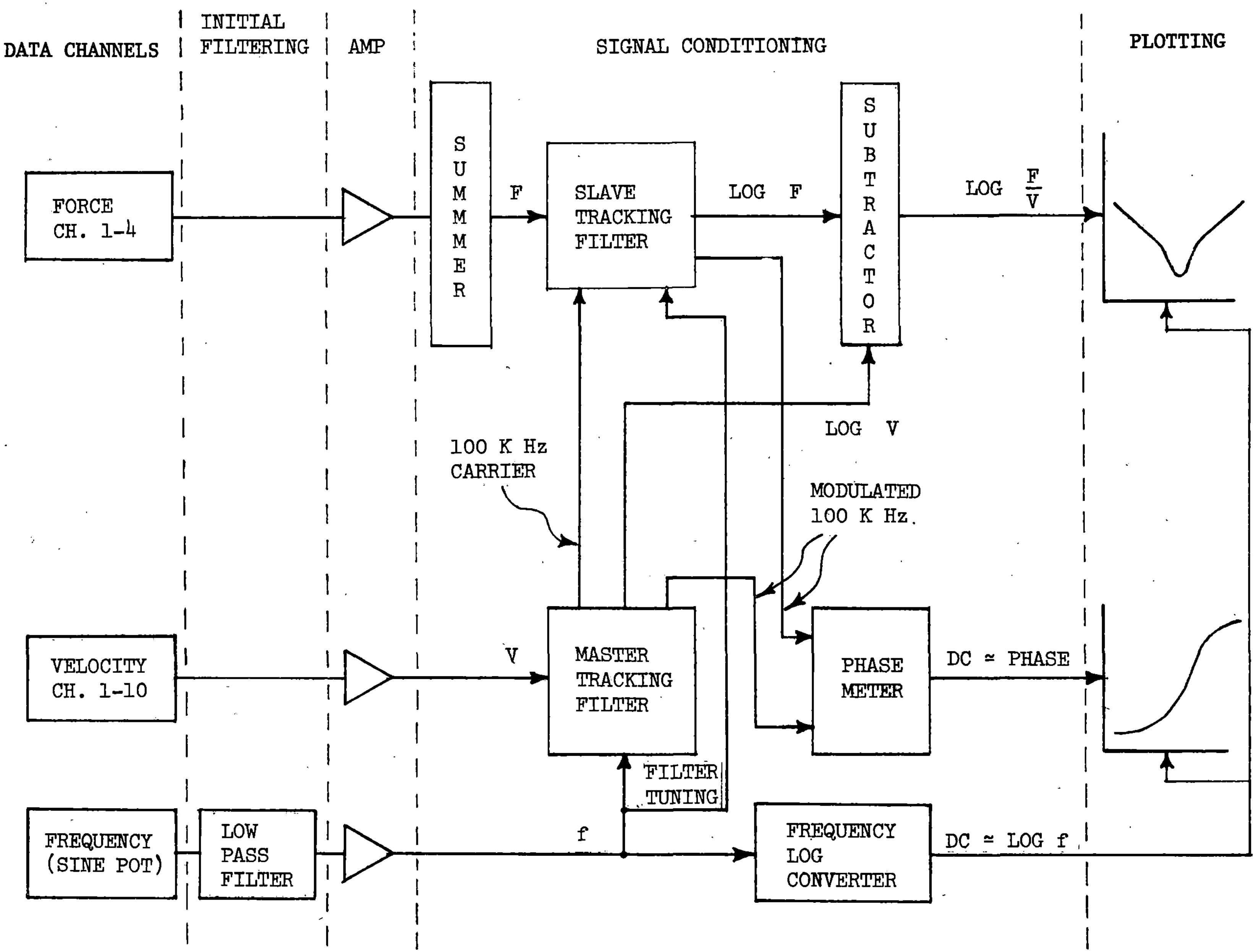


Fig. 11. Block diagram of instrumentation system used for reduction of impedance data

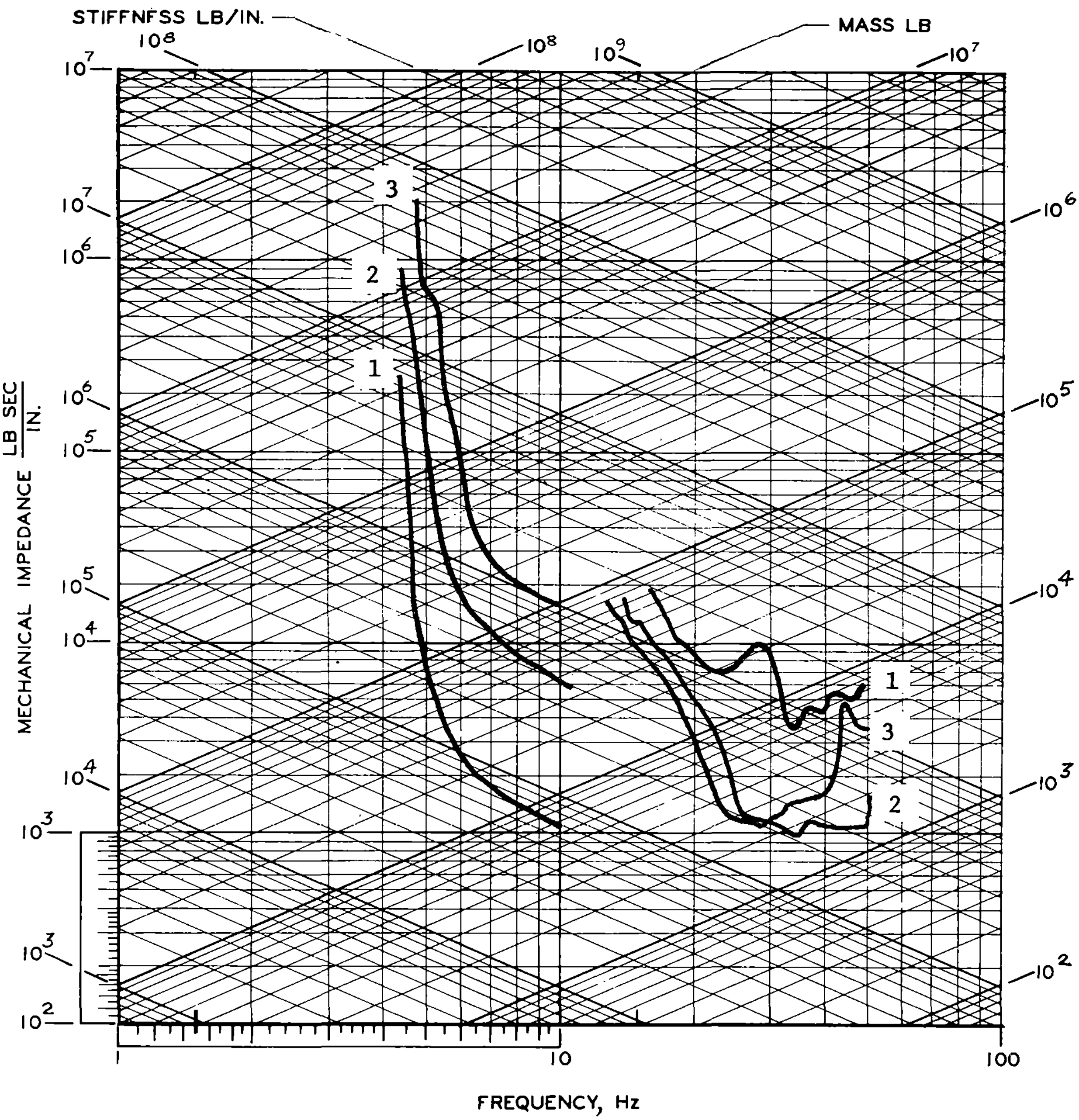


Fig. 12. Drive point mechanical impedance test results on track sections 1, 2, and 3

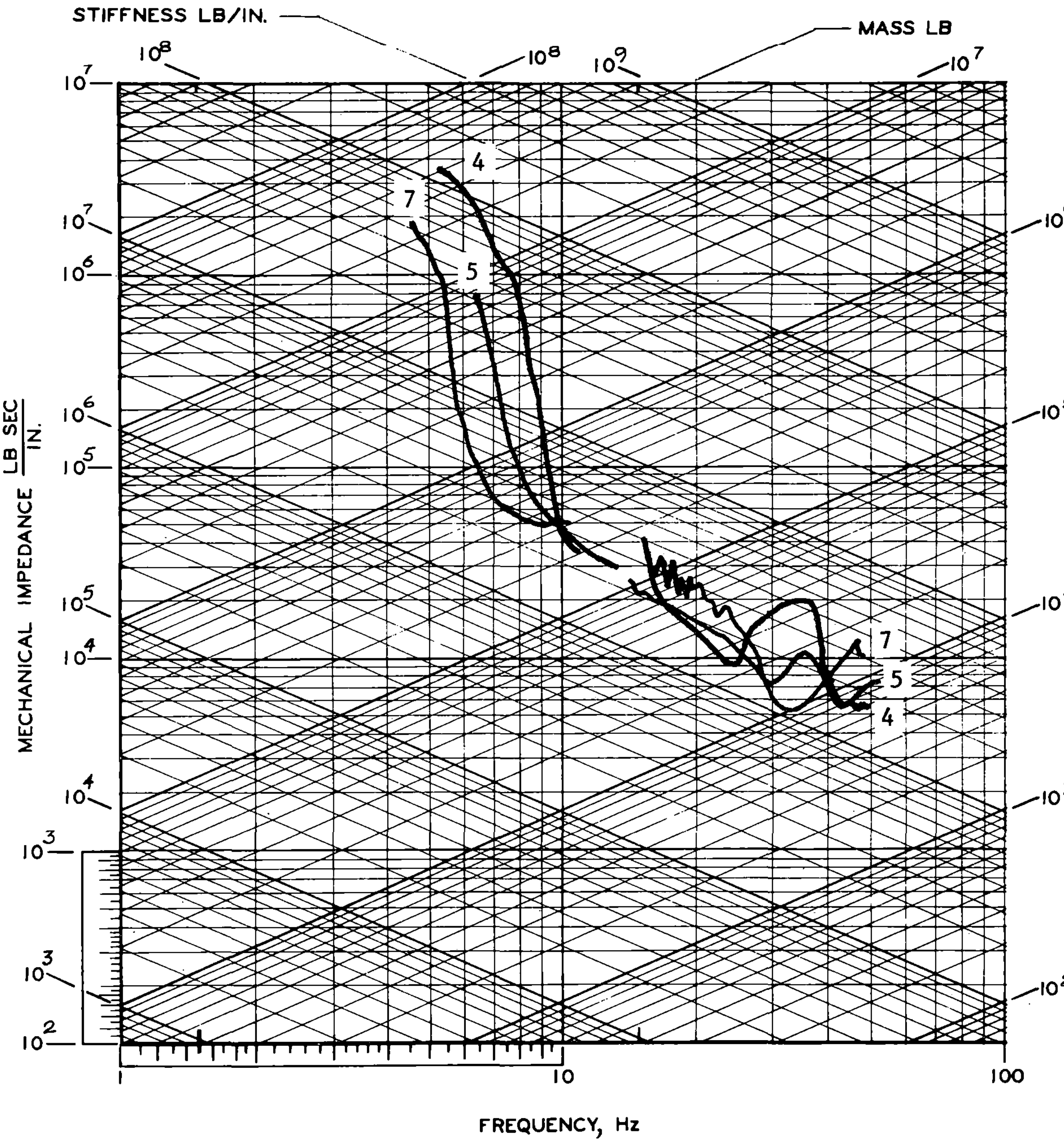


Fig. 13. Driving point mechanical impedance test results from track sections 4, 5, and 7

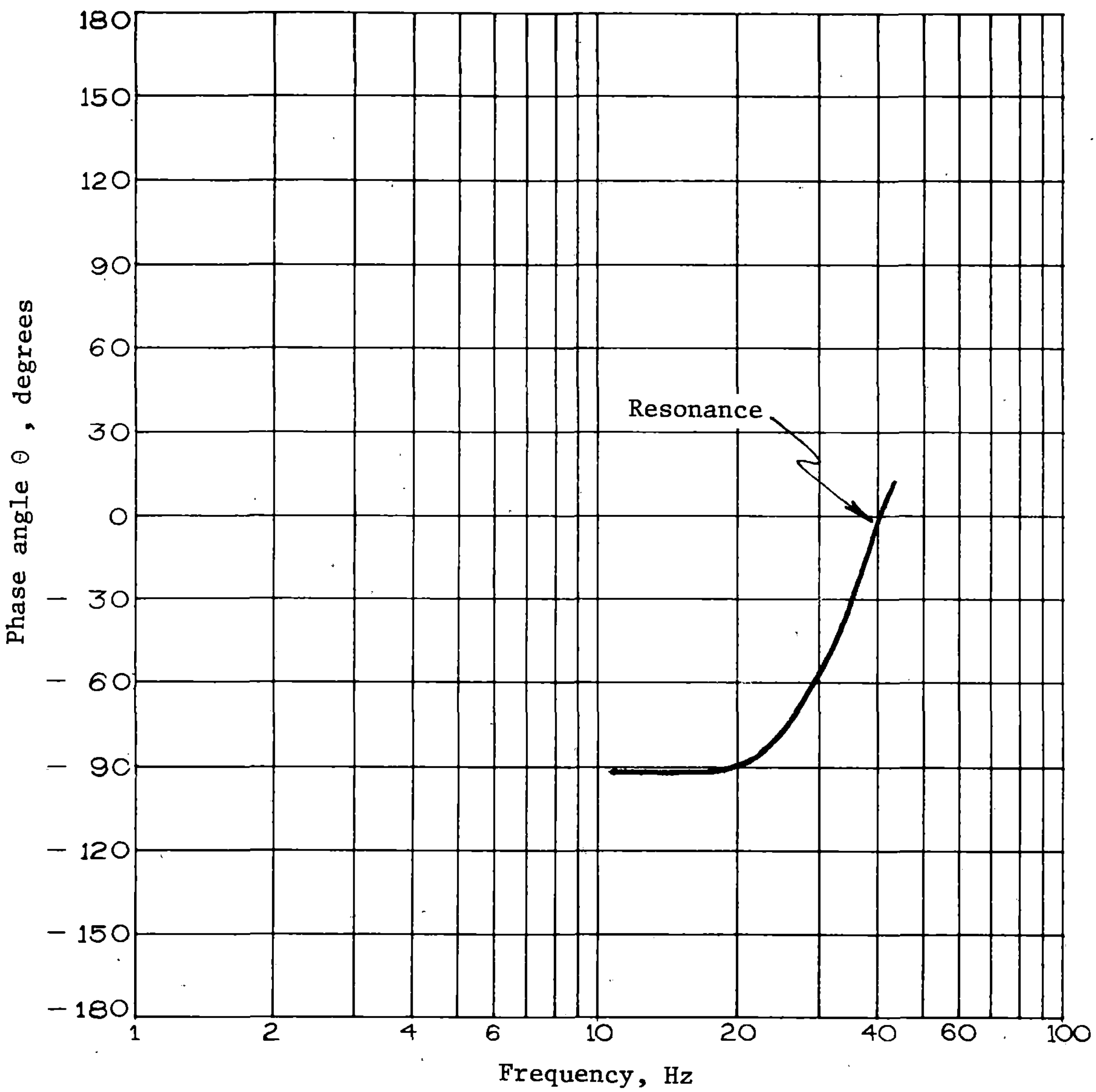


Fig. 14. Typical phase angle verses frequency plot from driving point impedance data obtained in track section 7

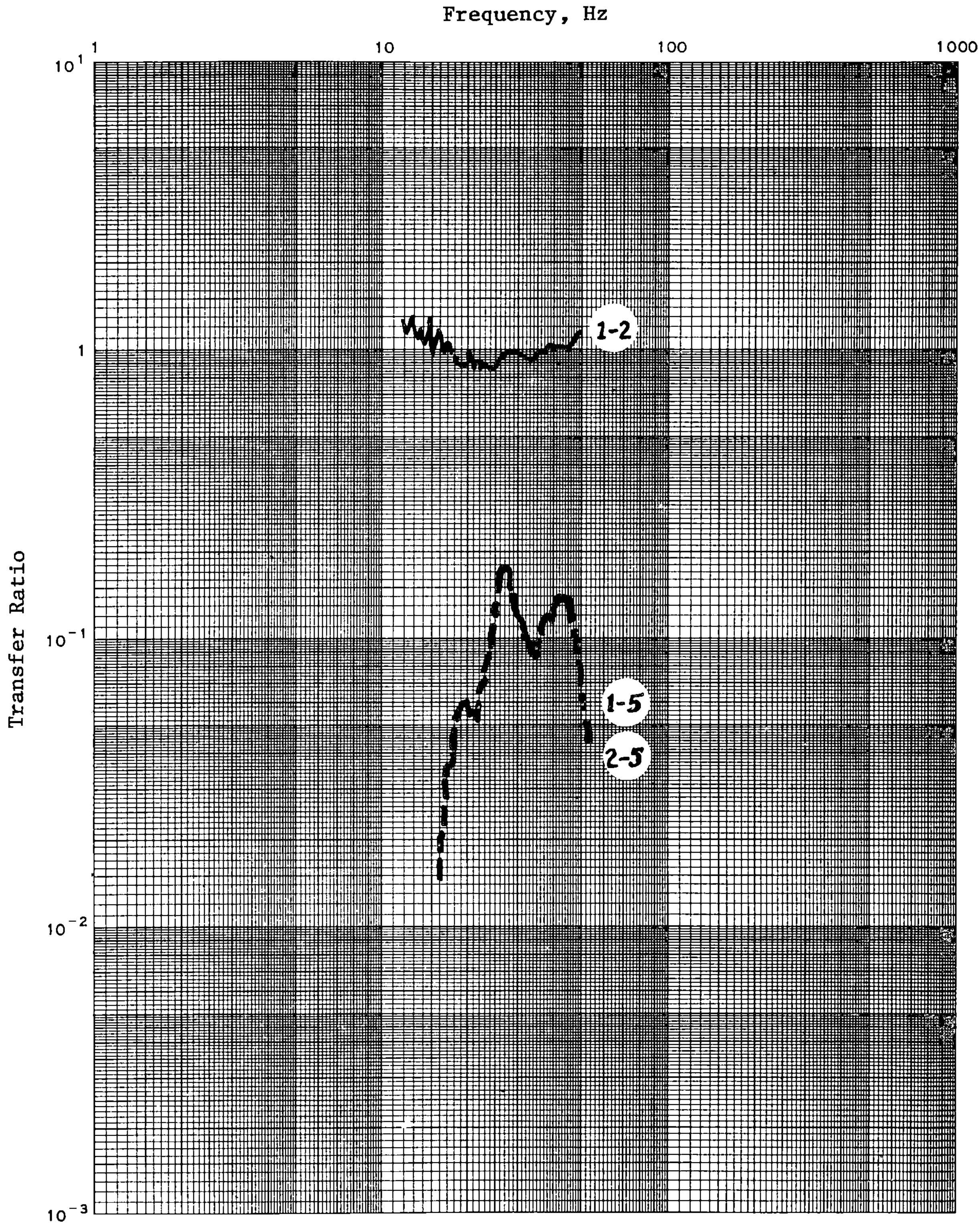


Fig. 15. Point to point velocity transfer ratios plotted versus excitation frequency

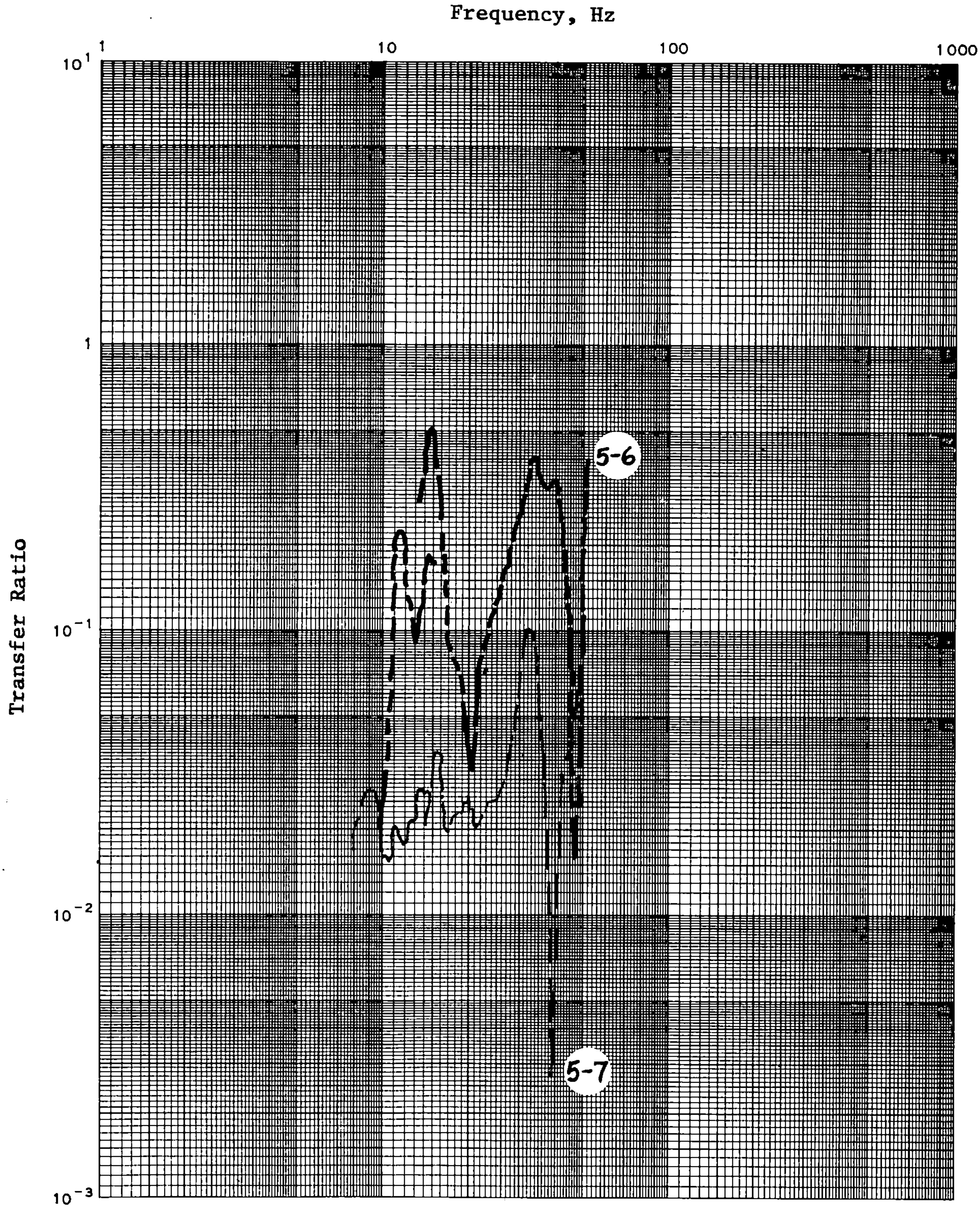


Fig. 16. Point to point velocity transfer ratios plotted versus excitation frequency

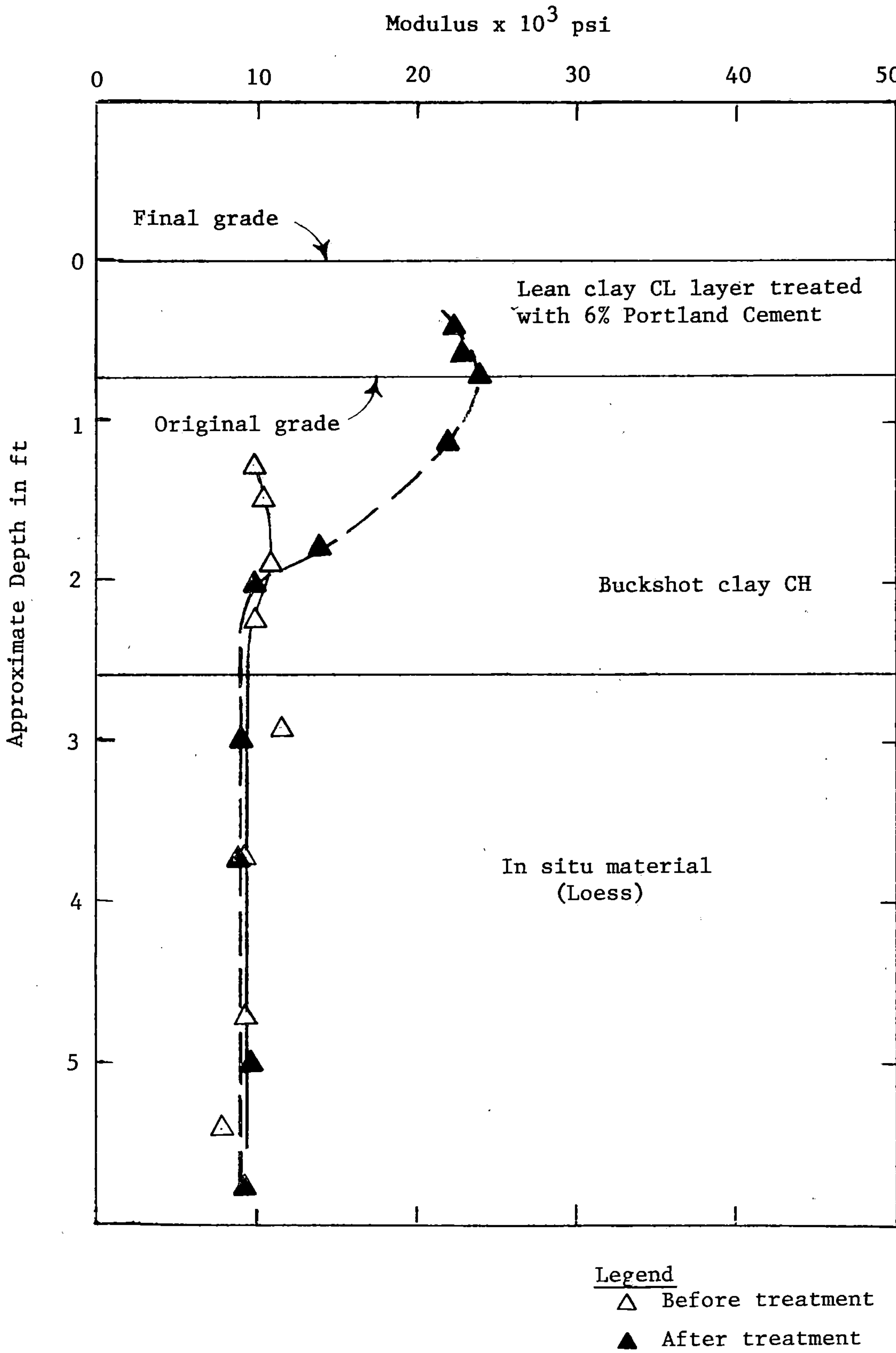


Fig. 17. Results of vibroseismic tests on cement stabilized soil

SUMMARY

The collection of lime injection conference papers presented in this report represents the most complete assembled body of knowledge on this topic in existence today. Each of the papers was individually prepared by the speaker/authors for presentation at the Roadbed Stabilization Lime Injection Conference. The railroad industry is indeed fortunate that these men were willing and able to attend and participate in this meeting. The lime injection method of roadbed stabilization has hence forth been placed in a position of technical legitimacy. This will surely aid in the future study, understanding and application of this valuable method of soil stabilization.

Many of the papers contain new ideas and pose new problems that are yet to be solved. The presentation of these papers is a major step in the development of this technology for inplace soil stabilization. It should be recognized, however, that this work is only part of the initial phase of the research and development engineering required to fully understand and apply the lime injection method of soil stabilization to the rehabilitation of railroad tracks and highways. It is essential that additional research be funded for continued study of lime injection and more importantly for the study of all promising chemicals which can be utilized in the modern injection pressure method of inplace soil stabilization for maintenance and rehabilitation of not only railroads but also highways, dams and other earth constructed structures.

APPENDIX A

ROADBED STABILIZATION : LIME INJECTION CONFERENCE
University of Arkansas - Federal Railroad Administration
Camelot Inn - Little Rock, Arkansas
August 21 and 22, 1975

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GLOSSARY OF TERMS

Adsorption -- Attraction of lime particles to surfaces of clay particles.

Amelioration -- Initial stage in clay-lime reaction in which plastic properties are improved but no permanent strength is achieved

Bearing Capacity -- The measure of the ability of the soil mass to support a static load.

Carbonation -- Formation of $CaCO_3$ by reaction of $Ca(OH)_2$ with CO_2 atmosphere.

Cementation -- Hardening action in which calcium silicates and aluminates are the main products in the chemical reactions of lime slurry with the principal soil components, namely, silica, alumina and aluminosilicates.

Consolidation -- A measure of the reduction in the size of a soil mass under a compressive load, due to water ejection. This is the time dependent process in which excess pore pressure dissipation results in void ratio reduction.

Curing -- Process of maintaining a soil mass or sample for a specific period of time under specific conditions of temperature and relative humidity so as to allow interval reactions in the soil to take place up to a satisfactory stage.

Deteriorating Track -- Track which is experiencing repeated progressive operational failures due to trackway problems.

Expansive Clay Soil -- A soil of predominantly clay particles which undergoes large volumetric changes with variations in moisture content.

Grouting -- The grouting of railroad subgrade usually refers to the pumping of cement-sand grout into the subgrade soil through grouting spuds either driven or drilled into the ground. Typical grouting projects in the general construction field include stabilization of slides, dam sealing, tunnel construction and void filling operations requiring the in-situ injection of large solid masses of hardenable structural materials. There is some overlap between the terms injection and grouting and sometimes the terms are used interchangeably.

Injection Pressure -- The lime slurry pumping pressure in psi in the injection rods. The absolute pressure (in psi) at which the lime slurry is injected into the soil. The pressure is usually in the range 50-200 psi.

Injection Spacing -- Horizontal distance between each injection hole.

Lime Blender Truck -- Hy-rail truck equipped with a mixing trank and agitation device to mix and haul lime slurry on a job site.

Lime Compatibility Tests -- Simple laboratory tests performed to determine in a nonquantitative manner whether or not lime will improve a given soil.

Lime, Hydrated -- (Calcium hydroxide). A material obtained by slaking "quick lime" with water, purchased according to standard materials specifications.

Lime Injection -- The process whereby lime slurry is pumped under pressure into the ground in large quantities at regular spacing intervals to specified depths required to treat problem subgrade soils.

Lime Injection Points -- The nozzle portion of the injection rod, usually constructed of machined hard steel several inches long with a suitable 360° hole pattern for slurry distribution.

Lime Injection Rod -- Hollow steel pipe (probe) used to inject lime into the ground, 10-20 feet long.

Lime Injection Truck -- Hy-rail truck equipped with a slurry holding and agitation tank, a high volume - high pressure pump, hydraulic injection mechanisms for pushing injection rods and necessary hoses and controls.

Lime Reactive Soil -- Soil which is significantly modified due to lime-soil chemical reactions.

Lime Seams -- Sheet like thin layers of lime slurry injected into cracks occurring within the soil mass.

Lime Slurry -- A liquid mixture of hydrated lime and water with or without additives.

Lime Slurry Additives -- Any chemical added to the lime slurry mixture, usually to act as a pozzolan or to accelerate curing.

Lime Slurry Tank -- Blending and storage tank for mixing, holding and dispensing lime slurry on a job site.

Lime Transport Truck -- Truck for hauling dry hydrated lime from lime plant to job site, 18-24 tons.

Lime-Water Ratio -- Refers to the amount of dry lime in lbs added to each gallon of water used to form slurry.

Moisture Content -- Amount of water contained in a soil mass, expressed as percentage of the oven dry weight of soil as determined by a closely defined test procedure.

Plasticity Index (P.I.) -- An indicator number which is numerically equal to the difference between the Liquid Limit and the Plastic Limit of a soil specimen. An expansive clay would have a "high P.I." Low P.I. soils are generally more stable and have less volumetric change than high P.I. soils.

Post Hole Method -- The post hole method refers to lime stabilization using pre-drilled post holes filled with lime slurry; it has been seldom used.

Pumping Soil -- A soil failure characterized by a water-bed type effect which provides unstable support for ballast and track. Mud pockets under the ties are often produced by pumping soils.

Railroad Roadbed -- That portion of the trackway below the ties which includes ballast, subballast and subgrade soils.

Railroad Trackway -- System including rails, ties, ballast and subgrade as an integral part, over the length of which trains travel.

Silty-Clay Soil -- General description of a soil containing substantial amounts of silt and clay. Such soils are usually associated with low strength and are sensitive.

Soil Exploration -- Subsurface soil drilling to obtain information on soil stratification and samples for laboratory tests and classification.

Soil Tests -- Laboratory tests conducted on soil samples obtained during soil exploration.

Spot Treatment -- Refers to the use of the lime injection method to stabilize short trouble spots along a trackway.

Squeeze -- A roadbed soil failure characterized by the presence of subsurface clay soils being extruded to the surface through the ballast.

Stabilization -- The act of modifying or changing the properties of a soil mass to improve its serviceability under existing load and environmental conditions.

Strength -- Soil shear strength or compressive strength.

Subgrade Soil -- Soil contained below the ballast and subballast in a trackway.

Supernatant Liquid -- Saturated solution of $Ca(OH)_2$.

Surfactant -- Chemical added to decrease the viscosity or lower the surface tension thus increasing flow characteristics of lime slurry in certain soils.

Tobermorite Gel -- Calcium silicate hydrate formed by slower reaction of lime with silica.

Treated Soil -- Soil which has been lime injected or otherwise chemically modified.

Untreated Soil -- Soil which has not been lime injected or chemically modified.

Volumetric Change -- The swell or shrinkage of a soil mass brought about by changes in moisture content.

Water Sensitive Soil -- Adverse characteristic of some soils to lose strength rapidly when brought in contact with extra moisture.

Water Transport -- Truck for hauling clean water to job site.

Wet/Dry Cycles -- Natural climatic cycles that cause a soil to alternately gain and lose moisture.

GPO 899-933

Proceedings of Roadbed Stabilization Lime
Injection Conference, 1975, 1976
US DOT, FRA