REPORT NO. FRA/ORD-78/09

Evaluation of Railroad Lime Slurry Stabilization

June 1978

Final Report

Prepared for

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL RAILROAD ADMINISTRATION
Office of Research and Development
Washington, D.C. 20590

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Technical Report Documentation Page

1. Report No. FRA/ORD-78/09	2. Government Accession No.	3. Recipient's Catalog No. PB282998

4. Title and Subtitle

Evaluation of Railroad Lime Slurry Stabilization

5. Report Date
June 1978

6. Performing Organization Code

8. Performing Organization Report No.

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Graduate Institute of Technology
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10. Work Unit No. (TRAIS)

11. Contract or Grant No.
DOT-OS-40107

12. Sponsoring Agency Name and Address
Federal Railroad Administration
Office of Research and Development
Improving Track Structures Research Division
Washington, D. C. 20590

13. Type of Report and Period Covered
Final Report
July 16, 1974-April 30, 1977

14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the Federal Railroad Administration and the Transportation Systems Center with the aid of the Railroad industry.

16. Abstract

This report describes a multifacted investigation into the application of lime slurry pressure injection (LSPI) to stabilize and improve railroad roadbeds. Areas discussed include (1) the current state of lime-injection technology, (2) soil exploration and testing related to the use of LSPI, (3) costs of roadbed stabilization by the LSPI method, (4) environmental aspects of the use of LSPI, and (5) initial applications of finite element analysis to the track-roadbed structure. In addition, summaries to two types of ancillary reports are included: (1) those resulting from case studies of several specific lime-injection projects and (2) those describing independent research work involving either the lime-soil combination or finite element analysis of the track-roadbed structure.

17. Key Words
Railroads, Roadbeds, Maintenance, Clay soils, Subsoil, Soil analysis, Soil properties, Soil tests, Soil stabilization, Calcium oxides, Slurries, Injection, Civil engineering

18. Distribution Statement
Document is available to the public from the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 190	22. Price PC A09 MF A01

Form DOT F 1700.7 (8-72)

Reproduction of completed page authorized

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS v

LIST OF TABLES vii

GLOSSARY OF TERMS ix

1. INTRODUCTION 1

2. DISCUSSION 7
    - 2.1 Data Collection, Storage and Retrieval 9
    - 2.2 Lime Injection Technology 15
    - 2.3 Soil Exploration and Testing 31
    - 2.4 Economic Study and Statistical Analyses 49
    - 2.5 Environmental Study 53
    - 2.6 Finite Element Analysis 59

3. CASE STUDIES 71
    - 3.1 Forrest City -- Rock Island Lime Injection Case Study, Phase I 73
    - 3.2 Pine Bluff -- Missouri Pacific Lime Injection Case Study 83
    - 3.3 Chillicothe -- Burlington Northern Lime Injection Case Study 87
    - 3.4 Cresson -- Santa Fe Lime Injection Case Study 93
    - 3.5 Belle Fourche -- Burlington Northern Lime Injection Case Study 95
    - 3.6 Forrest City -- Rock Island Lime Injection Case Study, Phase II 107
    - 3.7 Valley Drive Lime Injection Case Study 109

4. RESEARCH REPORT SUMMARIES 117

4.1 A Study of the Effect of a Lime Seam on the Unconfined Compression Characteristics of a South Dakota Clay by Chris H. Lawson 119

4.2 A Study on the Effects of a Saturated Solution from Lime Slurry on the Strength of a South Dakota Clay by Robert E. Blenden 133

4.3 Finite Element Analysis of Conventional Railway Track Support Systems by Chris H. Lawson 145

4.4 An Investigation of Stresses and Deflections in Train-Track Systems Using the Nonlinear Finite Element Method by Billy Jack McAlister 159

4.5 Laboratory Testing of Lime Slurry Pressure Injected Soils by Willliam C. Greeson 163

5. SUMMARY AND CONCLUSIONS 171

SELECTED BIBLIOGRAPHY 175

LIST OF ILLUSTRATIONS

Figure 2.1-1	Sample Contractor's Weekly Work Report	10
Figure 2.2-1	Lime Slurry Tank	19
Figure 2.2-2	On-Track Slurry Haul Truck	20
Figure 2.2-3	Lime Injection Truck on Hy-Rail Wheels	21
Figure 2.2-4	Lime Injection Nozzle	21
Figure 2.2-5	Pneumatic Drill Truck	22
Figure 2.2-6	Lime-Water Ratio Curves	24
Figure 2.2-7	Lime Injection in Progress in Wyoming	25
Figure 2.2-8	Lime Injection in a Typical Deep Cut Problem Area in Oklahoma	26
Figure 2.2-9	Side View of Operator's Position at Rear of Injection Truck	27
Figure 2.2-10	Rear View of Operator's Position on Injection Truck	28
Figure 2.3-1	Drilling Rig Mounted on Hy-Rail Wheels	33
Figure 2.3-2	Drilling in Progress	33
Figure 2.3-3	Soil Profile	36
Figure 2.3-4a	Actual Consolidation Test Results from a Lime-Injected Area	41
Figure 2.3-4b	Typical Inoculated Consolidation Test Results	41
Figure 2.3-5	Decision Flow Chart	44
Figure 2.6-1	Railroad Subgrade -- Transverse Simulation	64
Figure 2.6-2	Railroad Subgrade -- Longitudinal Simulation	65
Figure 3.1-1	Plan of Borings	74
Figure 3.1-2	Boring with Standard Highway Truck	75
Figure 3.1-3	Typical Boring Log	76
Figure 3.1-4	Track Roadbed at Heth	78
Figure 3.2-1	Soil Profile of the Pine Bluff Siding	84

Figure 3.3-1 Soil Profiles 88

Figure 3.5-1 Bentonite Spur Roadbed Shortly After Injection 96

Figure 3.5-2 Belle Fourche Soil Sampling Plan 98

Figure 3.5-3 Soil Testing for Belle Fourche, South Dakota 101

Figure 3.7-1 Boring Location Plan 110

Figure 3.7-2a Longitudinal Soil Profile 111

Figure 3.7-2b Transverse Soil Profile 112

Figure 4.1-1 Typical Lime Seam Formation and Distribution 121

Figure 4.1-2 Lime Seam Specimen 122

Figure 4.1-3 Final Lime Seam Mold Design 123

Figure 4.2-1 Cross-Section of Treatment Apparatus 138

Figure 4.2-2 Apparatus for Internal Application of Solution 139

Figure 4.3-1 Typical Cross-Sectional and Longitudinal Profiles of a Conventional Railway Track 146

Figure 4.3-2 Representation of Element Thickness Spread (after Tayabji) 148

Figure 4.3-3 Simplified Variation in Element Thickness With Depth 149

Figure 4.3-4 Finite Element Simulation of a CRTSS Cross-Section 150

Figure 4.3-4 Finite Element Simulation of Longitudinal Section 152

Figure 4.3-6 Stress Distribution for 80" Top Element 153

Figure 4.3-7 Stress Contours for Refined Grid Analysis 154

Figure 4.3-8 Stress at Ballast-Subgrade Interface from Refined Grid Analysis 155

LIST OF TABLES

Table 2.1-I	Summary of Contractor Cost Data (October 1975 - July 1976)	13
Table 2.1-II	Summary of Contractor Cost Data (October 1975 - July 1976)	14
Table 2.2-I	Example Material Analysis for Hydrated Lime	16
Table 2.3-I	LSPI Soil Tests with Positive Results	38
Table 3.0-I	Fully Developed Case Studies	72
Table 3.1-I	Chicago, Rock Island and Pacific Railroad Soil Classification Data Summary	79
Table 3.3-I	Results of Consolidation and Unconfined Compression Tests	90
Table 3.3-II	Index Property Data of Soil Samples	91
Table 3.5-I	Particle Size Distribution in the Sand-Silt-Clay Portion of Soil Samples Shown, After Clean-Up	103
Table 4.1-I	Soil Parameters	125
Table 4.1-II	Coefficients for Equation 1	127
Table 4.1-III	Influence of the Crack or Lime Seam on the Strength of the Specimens	129
Table 4.1-IV	Linear Model Coefficients for Equation 2	129

GLOSSARY OF TERMS

Accelerated Cure -- See Curing.

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

Carbonation -- Formation of calcium carbonate, $CaCO_3$, by reaction of calcium hydroxide, $Ca(OH)_2$, with carbon dioxide, CO_2, in the atmosphere.

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

Consolidation -- A measure of the reduction in the size of a soil mass under a compressive load, due to water ejection. This is a 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 internal reactions in the soil to take place up to a satisfactory stage.

Normal Cure -- The soil is sealed in a plastic bag and placed to cure at room temperature (22-25° C). The soil is effectively curing in its own atmosphere. It is good practice to place the sealed sample in a controlled-humidity chamber (100% relative humidity) to prevent moisture loss in case of poor sealing.

Accelerated Cure -- The soil is sealed in plastic bag and placed to cure at a temperature of 45-60° C. A good quality plastic must be used to prevent deterioration and subsequent moisture loss. The soil is effectively curing in its own atmosphere.

Deteriorating Track -- Track which is experiencing a progressive reduction in its capacity to carry traffic at predetermined operational characteristics (for example, speed).

Expansive Clay Soil -- A predominately clay soil that undergoes large volumetric changes with variations in moisture content.

Grouting -- Pumping of a cement-sand grout into the railroad subgrade soil through grouting spuds either driven or drilled into the ground. Typical grouting projects in the general construction field--which include slide stablilzation, dam sealing, tunnel construction, and void filling--require the 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 pounds per square inch (psi) in the injection rods. The gage pressure (in psi) at which the lime slurry is injected into the soil. The pressure is usually in the range of 50-200 psi.

Injection Spacing -- Longitudinal distance along the track between each injection hole.

Lime Blending Truck -- Hy-rail truck equipped with a mixing tank and agitation device to mix and haul lime slurry on a job site.

Lime, Hydrated -- A material (calcium hydroxide) obtained by hydrating quicklime with water. It is 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 to treat problem subgrade soils.

Lime Injection Nozzle -- The nozzle portion of the injection rod, usually constructed of machined hard steel several inches long with a suitable 360-degree hole pattern for slurry distribution.

Lime Injection Rod -- Hollow steel pipe used to inject lime into the ground, usually 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 that is significantly modified by lime-soil chemical reactions.

Lime Seams -- Thin sheet-like layers of lime slurry injected into cracks present 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, to accelerate curing or to act as a wetting agent (see Surfactant).

Lime Slurry Tank -- A large tank for storage of dry lime and for mixing, holding, and dispensing lime slurry on a job site.

Lime Transport Truck -- Truck for hauling dry hydrated lime from a lime plant to the job site, generally 18-24 tons in capacity.

Lime-Water Ratio -- The amount of dry lime in pounds added to each gallon of water to form a slurry.

Moisture Content -- The amount of water contained in a soil mass, expressed as a percentage of the oven dry weight of soil as determined by a closely defined test procedure.

Normal Cure -- See Curing.

Plasticity Index (PI) -- 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 PI." Low PI soils are generally more stable and have less volumetric change that do high PI soils.

Post Hole Method -- Lime stabilization using pre-drilled post holes filled with lime slurry. It has seldom been used.

Pozzolanic Reaction -- Mineralo-chemical reaction between lime and the clay minerals of the soil or any other pozzolanic component (such as hydrous silica) to form a tough, water-insoluble gel of calcium silicate that cements the soil particles together. In time, this gel gradually crystallizes into well-defined calcium silicate hydrates, such as tobermorite and hillebrandite.

Pumping Soil -- A soil failure characterized by a water-bed effect that provides an unstable support for the track. Mud pockets under the ties and fouled ballast are often the result of pumping soils.

Railroad Roadbed -- That portion of the trackway below the ties that includes ballast, subballast, and subgrade soils.

Railroad Track System -- System including rails, fastenings, ties ballast, subballast, and subgrade as an integral part.

Refusal -- Most of the slurry that is being injected is escaping to, and flowing freely on, the surface from surface breakouts (see Surface Breakout).

Silty-Clay Soil -- A soil containing substantial amounts of silt and clay. Such soils are usually associated with low strength and are sensitive to low percentages of moisture.

Soil Exploration -- Surface inspection and subsurface soil drilling to obtain information on soil stratification and samples for laboratory-tests and classification.

Soil Tests -- Field and laboratory tests conducted on soil samples obtained during soil exploration.

Spot Treatment -- The use of lime injection or other techniques to improve short trouble spots along a track.

Squeeze -- A roadbed soil failure characterized by the presence of subsurface clay soils extruded to the surface through the ballast (similar to a pumping soil).

Stabilization -- Modifying or changing the properties of a soil mass to improve its serviceability under existing load and environmental conditions.

Subgrade Soil -- Soil below the ballast and subballast in the roadbed.

Supernatant Liquid -- Saturated solution of $Ca(OH)_2$.

Surface Breakout -- The slurry that is being injected begins flowing rapidly back out of the ground at one or more points. The breakout(s) may occur around the injection rods, out of previous injection holes, or through fractures in the soil.

Surfactant -- Chemical added to decrease the viscosity or lower the surface tension and thus to increase the flow characteristics of lime slurry in certain soils.

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 -- A soil with the adverse characteristic of losing strength rapidly when brought in contact with extra moisture.

Water Transport Truck -- Truck for hauling clean water to the job site.

Wet-Dry Cycles -- Natural climatic cycles that cause a soil to alternately gain and lose moisture.

1. INTRODUCTION

The lime slurry pressure injection (LSPI) method of roadbed stabilization is one of the few new methods currently being used by the railroads to treat both large and small areas of problem track. The Graduate Institute of Technology (GIT) of the University of Arkansas has been working in the LSPI area since September 1973. In July 1974 the GIT was awarded a Federal Railroad Administration contract to investigate, through engineering experiments, the various factors that influence the success or failure of the LSPI system to improve the subgrade soils of problem roadbeds. The railroad research team has conducted an engineering and chemical analysis and laboratory testing program and has evaluated and documented data generated by the contractors and several participating rail lines covering many aspects of LSPI. Indications are that LSPI is proving to be a valuable method for stabilizing certain problem roadbed soil types and that is is substantially reducing the maintenance cost on some sections of track.

The research program has been expanded beyond its original scope by authorization of seven contract modifications. The additional funds provided have enabled the research team to greatly expand the LSPI case history studies and to conduct several related student and staff research reports. Fifteen GIT special reports have been completed. These cover the results of seven documented case studies, the lime injection handbook, the conference proceedings, and five graduate student research reports. These are each summarized in Chapters 3 and 4 of this report.

The objectives of the program were listed in the original proposal as follows:

1. To examine the ability of the Lime Slurry Pressure Injection stabilization technique to improve the in-place subgrade soils of railroad ways.
2. To thoroughly document, study, and evaluate data generated by the several rail lines that have used the LSPI method over the last five years.
3. To develop information requisite for field utilization of the LSPI stabilization technique as applied to railroad ways. This study includes verifying the concepts and premises on which the method is founded, and delineating the conditions under which application of LSPI is optimally effective.
4. To evaluate the past and present field performance of this method of track maintenance, and to attempt to substantiate its degree of success with preliminary rational design criteria.
5. To provide specific guidelines for future utilization of the LSPI stabilization technique by studying the cost effectiveness and the environmental impact of the method.
6. To submit recommendations for future LSPI research programs to include design, analysis, construction technology, and a full-scale testing program.

These were all essentially met; however, the economic study and statistical analysis areas were extremely difficult and a limited amount of progress was made in these areas. The success in the other areas was quite satisfactory. The LSPI method has grown considerably as an industry since inception of the program, and today the contractors are using better equipment and better lime injection technology. This has resulted in part from this research work.

Much of the work performed under the contract and modifications has been presented in separate papers, reports, and publications, which are included in the Selected Bibliography at the end of this report. This body of literature has been summarized in this report and combined with new materials. In addition to this introduction, the report comprises four major sections, each of which is divided into subsections that treat specific areas related to LSPI soil stabilization. While all of these subsections are interrelated, each subsection is presented as a complete unit, and references to the literature within a subsection are listed at the end of the subsection. There is no master list of references at the end of the report. Instead, the Selected Bibliography has been included to provide the reader with a concise history of materials of significant interest.

One major accomplishment of the GIT railroad research effort was the writing of the <u>Handbook for Railroad Track Stabilization Using Lime Slurry Pressure Injection</u>. This document, which is available for purchase through the National Technical Information Service (see Selected Bibliography), was just recently released, and the full evaluation and impact of the handbook is in the future. It is anticipated that the handbook will be accepted as the standard for railroad lime injection work.

The information contained in the handbook was collected or developed to assist railroads and injection contractors to obtain more effective and economical applications of lime injection. Because this method of soil treatment is constantly undergoing modification and improvement, the handbook is far from definitive and provides only the existing information on the lime injection process, soil testing and evaluation, and project management. It is anticipated that the handbook will be revised as better information becomes available.

The railroad engineer who is considering the use of LSPI stabilization for the first time will find the entire handbook to be helpful, especially the section on Surface and Subsurface Soil Exploration and Testing. This section will be most valuable when developing the initial project plan for a particular problem section of track. Sections on Safety Precautions and Environmental considerations are provided to enable the railroad engineer to gain knowledge quickly about these specialties as they relate to LSPI. The Lime Injection Technology section gives a complete description of the present state of the art. The equipment, procedures, and techniques discussed in this section have been developed by soil engineers, railroad personnel, and the contractors over the past six years of LSPI roadbed stabilization.

In August, 1975, GIT sponsored a lime injection seminar that was attended by 110 railroad and contractor personnel. Proceedings of the Roadbed Stabilization Lime Injection Conference has been published and is being distributed worldwide through NTIS (see Selected Bibliography). The papers covered technical data oriented to railroad track stabilization, maintenance design criteria, and new construction. The data and ideas in the papers are those of the authors and do not necessarily reflect the views of the FRA; the University of Arkansas; other federal, state, or private organizations; or the editor.

Mr. J. B. Farris of the Southern Railroad, the first conference speaker, related his experience with lime injection on the Southern, which was the first railroad to utilize the current method of lime injection stabilization. Mr. Paul Wright of Woodbine Corporation presented a paper explaining the contractor's viewpoint, equipment, and railroad injection techniques.

Dr. James A. Eades of the University of Florida presented an extensive lecture with many color slides on the subject of Lime-Soil Reaction. Dr. Eades is a foremost 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 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 nondestructive testing of roadbed soils, recounting experiences from the Kansas 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 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 Structures 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 the 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.

Dr. Albert Vickers, Mr. 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 Drs. Welch and Kumar discussed laboratory testing methods and results. Mr. Sheaff, who has been working on the case history study program for lime injection maintenance of railroad subgrades, presented a progress report on the accomplishments of that portion of the research.

The collection of lime injection conference papers is the most complete compilation of knowledge on this topic in existence today. 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 in-place 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.

2. DISCUSSION

The research program was divided into six work areas:

Work Area I - Data Collection, Storage, and Retrieval
Work Area II - Subsurface Soil Exploration
Work Area III - Lime Injection Technology
Work Area IV - Economic Impact Study and Statistical Analyses
Work Area V - Environmental Studies
Work Area VI - Track Design and Analysis

The effort in each area is summarized in this chapter. Much of the research progress has been described in separate reports, and the majority of that data has not been reproduced in this document. The Selected Bibliography at the end of this report lists all the University of Arkansas "Special Reports." These are on file in the technical library at the Graduate Institute of Technology, and copies may be purchased upon request.

The six sections of this chapter provide a comprehensive treatise of the lime injection information that was assembled and developed during performance of this contract. All the progress achieved in Work Areas I and IV is recorded in this chapter. These were the only two work areas that were completed according to the original scope of the program. All other areas were expanded by action of the seven contract modifications and independent and university-supported research. The work in Soil Exploration and Testing and Lime Injection Technology was expanded through case history studies, which are discussed in Chapter 3. Similarly, the finite element analysis work was expanded beyond the original scope through independent student projects, which are summarized in Chapter 4.

It should be noted that the area of economic analysis and evaluation of track rehabilitation according to proper economic techniques is equally important to the other work areas. It was given less emphasis during this program only because of a lack of adequate maintenance cost data.

2.1. DATA COLLECTION, STORAGE AND RETRIEVAL

During the initial contract reporting period, a system for collecting data from the injection work crews was devised and implemented. The data were submitted weekly by the field work crews and the railroad companies on forms designed to provide engineering, economic, and environmental information associated with the lime pressure injection stabilization process. A sample of the form on which the data were submitted is shown in Figure 2.1-1. The parameters selected for data storage were chosen to provide the information required to perform statistical analyses in support of the other work areas of the research project.

When all the lime injection crews were working and reporting data, there existed a weekly influx of approximately 1000 discrete items of information. The collection of the lime injection data submitted as weekly reports continued from October, 1974, through July, 1976; and a total of 975 reports were submitted by two contractors. This information describes the injection of 79.33 miles of railroad track owned by 22 railroad companies. A total of 13,165 tons of lime was injected at a cost of approximately $1.4 million. It was, therefore, necessary to develop a convenient and efficient method of storage and retrieval of the data. This was accomplished by means of a Fortran IV computer program. The computer used was an IBM 370-155 located in the Computer Services Division of the University of Arkansas, Fayetteville Campus. The program could be addressed from a remote terminal located at the campus of the University of Arkansas at Little Rock.

The data were first stored on standard IBM keypunch cards and then read from the cards onto magnetic tape. Two cards were required to record the data for each calendar day appearing on the weekly work report. Each pair of cards contained the following data:

LIME STABILIZATION CONTRACTOR'S
WEEKLY WORK REPORT

W.E. 11/10, 19 74

R. R. Name ______ Region ______

R. R. Division Engineer ______ Location ______

R. R. Inspector or Flagman ______ Location ______

Job Location: Fayetteville State North Carolina

DAY	MON	TUES	WED	THURS	FRI	SAT	SUN
DATE	4	5	6	7	8	9	10
Temperature Daily (high and low)	60-80	60-78	50-71	41-59	34-55	34-56	off
Precipitation Daily (inches of rainfall)	none	none	none	none	none	none	
Location of Area Worked (mile post,etc.)	30.2	29.9	29.8	29.6	29.5	29.4	
Track Injected (feet)	429	429	468	624	468	468	
Injected Spacing (cribs)	2/3	2	2	2	2	2	
Injection Depth (feet)	10	10	10	10	10	10	
Injection Pressure (psi)	75	75	75	75	75	75	
Lime Delivered Per Day (tons)	20.1	16.1	15.1	18.2	17.0	16.6	
Lime Water Ratio (lbs. per gallon)	2.5-3	2.5-3	2.5-3	2.5-3	2.5-3	2.5-3	
Customer Delays (hours)	none	none	none	none	none	none	
On Track Work Time (hours)	10	10	10	10	10	10	
Total Hours All Employees on job per day	35	32	33	36	35	33	
Site Description (cut,fill,level,etc.)	fill	fill	fill	fill	fill	fill	
Soil Description (general terms)	clay, clay,	pipe gumbo and	sand	same	same	same	

Lime Supplier and Location ______

Contractor's Injection Unit Number 69-18 Haul Truck Unit Number 68-16

Method of Mixing Lime and Water Slurry tank with mechanical agitator

Type of Surfactant Wet-it Ratio 1 gal. to 6500 gal.

Any Unusual Conditions Monday middle injector stuck in ground. Worked with it and got it out.

Fig. 2.1-1. Sample contractor's weekly work report

First Card

Columns	Data
1-5	Code for contractor
6-10	Code for foreman
11-15	Code for railroad name
16-20	Code for state
21-25	Code for date
26-30	Daily low temperature (degrees F)
31-35	Daily high temperature (degrees F)
36-40	Daily precipitation (inches)
41-45	Length of track injected (feet)
46-50	Number of cross ties between injections
51-60	Injection pressure (psi)
61-65	Lime delivered (tons)
66-70	Lime-water ratio (lbs/gal)
71-75	Customer delays (hours)
76-80	On track work time (hours)

Second Card

Columns	Data
1-5	Time charged to railroad (hours)
6-10	Code for site description
11-15	Code for soil consistency
16-20	Code for soil color
21-25	Code for the soil type
26-30	Code for lime supplier
31-35	Code for method of mixing
36-40	Indication of unusual conditions
41-45	Code for type of surfactant
46-50	Surfactant ratio (parts/10,000)

The data were stored on magnetic tape in the form of card images. In this form whatever processing of the data was required could be performed by writing an appropriate Fortran program. For example, all the data could be printed out in full, or only selected portions of the data could be rearranged and printed. Also, simple statistical analyses (mean, standard deviation, summations, etc.) of the stored data could be performed. If more sophisticated statistical analysis is required, this form of tape storage makes it possible to use a program called the Statistical Package for the Social Sciences (SPSS) developed at the University of Wisconsin.

Once the data storage and retrieval system became operational, it was called upon regularly to provide appropriate input data that were used to develop the Economic Impact Study and Statistical Analyses and the Track Design Analysis of this Final Report. As new data were added to the tape, a summary of the data consisting of totals and averages for each work crew was made available for the Economic and Statistical Analyses. The last summary made is shown in Tables 2.1-I and 2.1-II.

TABLE 2.1-I

Summary of Contractor Cost Data (October 1975-July 1976)*

CONTRACTOR WORK CREW	TOTAL ON TRACK WORK HOURS	TOTAL HOURS CHARGED TO RAILROAD	TOTAL TRACK FOOTAGE	TOTAL TONS OF LIME INJECTED	COST/MILE BASED ON $125/HOUR**
# 1	373	381	17084	613	$21354
# 2	556	666	42148	1589	17390
# 3	58	58	4813	162	14105
# 4	500	548	30395	1120	18698
# 5	131	188	4075	162	37738
# 6	491	545	22399	600	21018
# 7	1369	1771	103689	3545	17588
# 8	336	363	15243	652	23625
# 9	332	383	30410	394	10693
#10	117	168	7805	180	18447
#11	82	150	6891	80	16512
#12	728	790	48791	1283	15541
#13	414	425	32101	648	12476
#14	518	641	21065	951	28425
#15	134	159	9022	392	19676
#16	86	102	4426	212	24083
#17	187	243	12352	446	19627
#18	85	117	6200	136	16455
COMBINED	6497	7698	418909	13165	$17931

*All numbers in this table have been rounded to integers.

**Includes cost of lime at $35 per ton.

TABLE 2.1-II

Summary of Contractor Cost Data (October 1975-July 1976)*

CONTRACTOR WORK CREW	AVG. WORK HOURS PER MILE	AVG. TONS OF LIME PER MILE	AVG. SPACING (TIES BETWEEN INJECTIONS)	AVG. DEPTH (FEET)	AVERAGE PRESSURE (PSI)	AVG. LIME-WATER RATIO (LBS/GAL)
# 1	115	190	2.1	10	100	2.9
# 2	70	199	3.0	7	200	2.5
# 3	63	178	3.0	10	200	3.0
# 4	87	195	3.0	10	131	2.8
# 5	169	209	3.0	14	101	4.3
# 6	116	141	2.0	7	113	2.3
# 7	70	181	3.0	13	106	3.1
# 8	116	226	2.5	12	127	3.1
# 9	58	68	2.7	8	132	4.3
#10	79	122	3.0	7	133	2.5
#11	62	61	2.4	7	100	2.5
#12	79	139	2.3	10	147	2.6
#13	68	107	2.9	12	110	2.9
#14	130	238	2.3	12	123	3.0
#15	78	229	3.0	11	155	3.0
#16	102	254	2.3	13	143	3.6
#17	80	190	3.3	15	122	3.5
#18	72	116	3.0	10	125	3.5
COMBINED	82	166	2.7	11	127	3.0

*All numbers in this table have been rounded to integers or two significant digits.

2.2 LIME INJECTION TECHNOLOGY

The immediate physical goal of lime injection is to achieve economically a uniform dispersal of the lime slurry throughout the treated soil mass. During the past few years of actual railroad LSPI stabilization operations, a step-by-step technology for efficient injection of roadbeds has been developed with this goal in mind. The railroads and lime injection contractors are continuously refining this technology to attain more uniform coverage economically, and future LSPI roadbed projects should utilize better injection technology through improved equipment, procedures, inspection, and quality control.

The current railroad LSPI technology includes criteria for materials, equipment, mixture control, injection techniques, and injection records and inspection. Proper control of each of these items contributes to the success of any particular lime injection project; therefore, the use of a properly prepared plan that includes engineering specifications is recommended for each stabilization project. General specifications developed during this research program are presented in the lime injection handbook. The material in the handbook and the discussion below will help provide a solid foundation for a successful, efficient lime injection project directed toward roadbed stabilization.

Materials

Lime is sold commercially in two forms: quicklime and hydrated lime. Quicklime, CaO, which is produced by burning limestone, $CaCO_3$, in kilns to drive off carbon dioxide, is considered to be hazardous for use in railroad LSPI stabilization projects and, therefore, has seldom been utilized.

Hydrated lime, $Ca(OH)_2$, is manufactured by grinding quicklime, mixing with water, and drying and pulverizing the mixture into a flocculent powder.

Hydrated lime is relatively safe to use and economical to purchase and, therefore, is utilized in the large majority of the LSPI projects. Hydrated lime should be purchased according to a standard materials specification for construction-grade hydrated lime. State highway departments can supply such specifications, as well as a list of qualified material suppliers. Also, the lime can be purchased according to ASTM D C-207, Type N, except that the calcium hydroxide content must be not less than 90 percent and the requirements for popping, pitting, and water retention shall not be applicable. The supplier of the lime shall be prepared to furnish certified evidence of the quality of his product. A physical and chemical analysis for a typical suitable hydrated lime is shown in 2.2-I.

TABLE 2.2-I

Example Material Analysis for Hydrated Lime

Components	Weight (%)
Free Moisture	0.30
Chemically Combined Moisture	23.39
Silicon Dioxide	0.11
Iron Oxide	0.20
Titanium Oxide	0.01
Manganese Dioxide	< 0.001
Aluminum Oxide	0.22
Calcium Oxide	73.98
Magnesium Oxide	0.17
Sulfur Trioxide	0.04
Phosphorus Pentaoxide	< 0.01
Insoluble (Less Silica)	0.16
Carbon Dioxide	1.11
% Passing 200 Mesh	95
% Passing 325 Mesh	87

Carbonation of hydrated lime is caused by absorption of carbon dioxide, CO_2, from the air. Excess water used in forming the lime paste evaporates and is gradually replaced by CO_2, causing any free lime hydrate to revert to the original $CaCO_3$ [i.e., $CA(OH)_2 + CaCO_3 + CO_2 \rightleftarrows CaCO_3 + H_2O$]. Hydrated lime will carbonate rapidly when exposed to air. Carbonation of the hydrated lime is not desirable and should be prevented prior to injection because the carbonated lime will not react with the soil minerals to form the necessary soil-cementing agents.

The subject of waste, or reclaimed, lime currently is of interest to several of the railroads because of substantial reductions in purchase price over new certified hydrated lime. The use of waste lime is considered to be outside of the scope of this report, and report statements are not to be considered as applicable to stabilization using lime other than that purchased under acceptable specifications. Some of the injection work performed in the infancy of the LSPI method utilized waste lime. Virtually all of those jobs were considered to be failures, probably due not only to the use of waste lime but also to the inadequate hand injection methods that were available prior to the development of hydraulic equipment.

In addition to certified hydrated lime, materials for lime injection include water and, possibly, a surfactant (wetting agent). Water used in mixing lime slurry shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to the desired lime-soil reaction. If nonpotable water is proposed for use and if there is any doubt concerning compliance with the above statement, then laboratory tests should be conducted to compare the lime-soil reaction of specimens incorporating the nonpotable water with the reaction of similar specimens incorporating potable water.

A surfactant may be used as indicated by the particular soil conditions of the injection site. The surfactant, which should be used according to the manufacturer's recommendations, helps reduce surface tension between fine-grained soil particles and the lime slurry, thus allowing further penetration into the soil mass.

Equipment

The equipment used for modern railroad lime injection stabilization was designed and engineered for precisely this one function. It was the development of this specialized equipment for railroad applications that made LSPI stabilization economically feasible and routinely practical.

A typical injection fleet for railroad stabilization applications comprises a storage tank, a slurry mixing unit, slurry transports, and hy-rail injection truck. Such a fleet normally is operated by three or four crewmen. The lead crewmen, who is experienced in lime injection, is trained to supervise the lime injection sequence and to look for and troubleshoot problems. In addition, he is responsible for customer coordination, ordering materials, accepting deliveries, and keeping field records. One or two men handle the slurry mixing and hauling, and one crew member operates the injection truck.

Bulk Storage

Lime transport trucks are used to transfer the dry hydrated lime from a lime plant to the job site. Water transport trucks are used if water of the required quality is not available at the job site. The lime may be stored at the site in the transports or in large wet or dry holding tanks. The wet holding tanks, called lime slurry tanks (Figure 2.2-1), are utilized both as

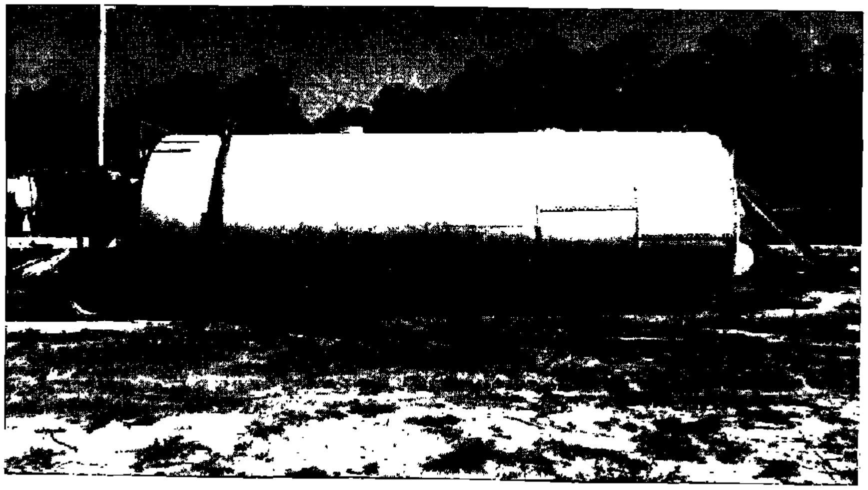


Fig. 2.2-1. Lime slurry tank.

storage tanks and as mixing units. The dry tanks are equipped with a pneumatic blower system to transfer the lime to the equipment that mixes the slurry.

Mixing Equipment

Currently, there are two slurry-mixing systems. In one system, the large lime slurry tank is used to mix lime slurry in bulk. In the other system, lime is transferred from the dry holding tanks to small blending trucks. Each system is used to mix dry lime and water and to agitate the solution to form a slurry. The main difference between the two systems is size.

The lime slurry tank is capable of producing up to 17,000 gallons of slurry in one batch. The tank, which is equipped for road travel when empty, has a centerline paddle-wheel agitator to insure uniform suspension of the lime.

The blending truck is used to mix 1500 to 2000 gallons of slurry at one time. Blending trucks are equipped with pump or paddle-wheel agitation systems, and some have hy-rail wheels.

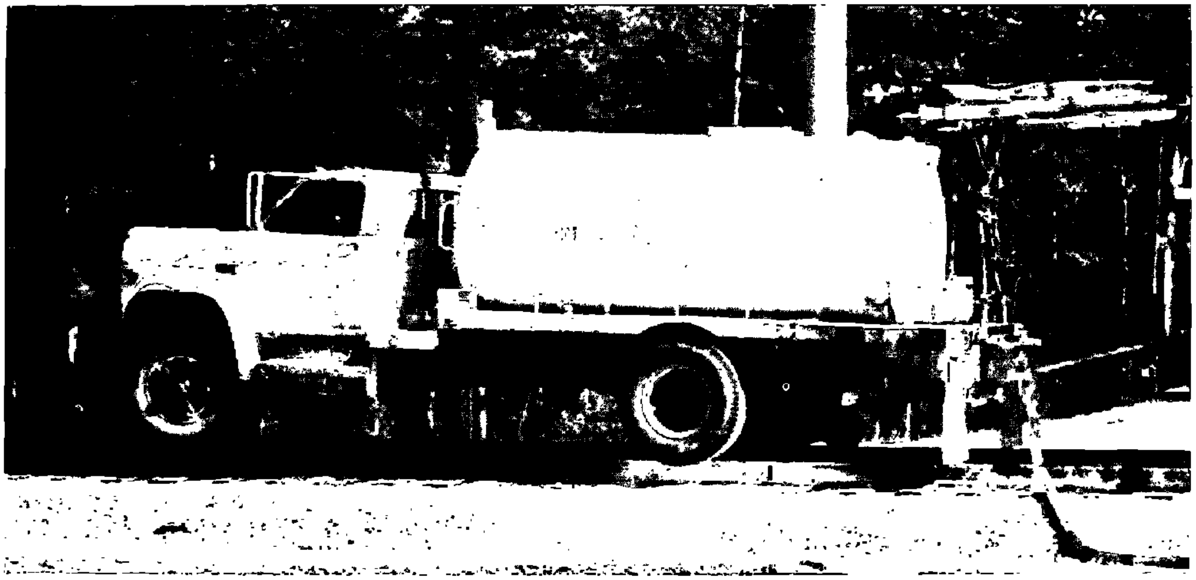


Fig. 2.2-2. On-track slurry haul truck.

On-Track Haul Truck

The link between the mixing system and the injection rig is the on-track haul truck (Figure 2.2-2). Equipped with hy-rail wheels, these trucks are capable of accompanying the injection rig as it moves along the track from one injection site to the next. Each haul truck has a slurry tank capable of holding 1500 to 2000 gallons, an agitation system, and a transfer pump.

When the lime slurry tank is used, the slurry may be pumped directly to the on-track haul truck if it is possible to locate the tank near the track. Otherwise, the slurry is transferred from the tank to the haul truck via a slurry transport truck. When the blending truck is used, the slurry may always be pumped directly to the on-track haul truck; however, in some cases, the blending truck may double as the haul truck.

Lime Injection Truck

The basic item of equipment for the LSPI process is the lime injection truck, which is equipped with hy-rail wheels for on-track operation

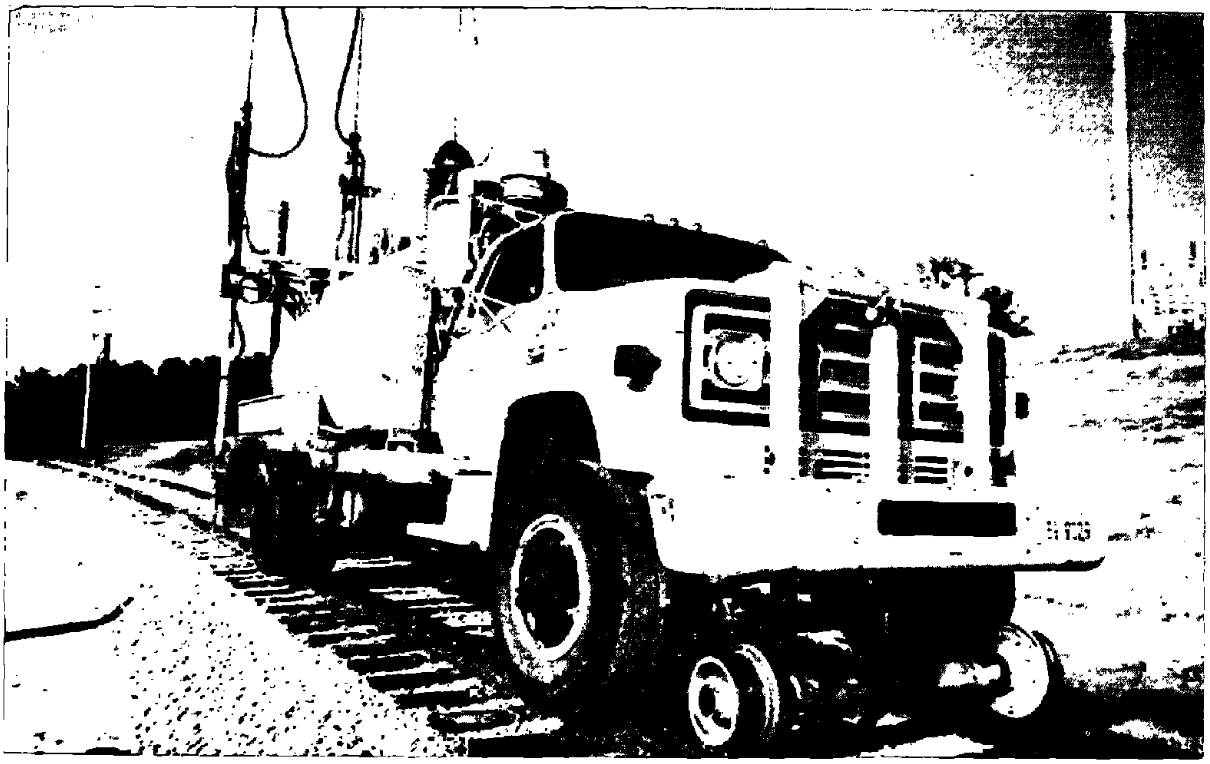


Fig. 2.2-3. Lime injection truck on hy-rail wheels.

(Figure 2.2-3). The injection truck also is equipped with a suitable agitation system, slurry tank, high-pressure pump, and three hydraulic injection rods.

The three injection rods are spaced 5 feet apart across the rear of the injection truck with the center rod at the track centerline. Each injection rod is made of steel pipe that is threaded on the lower end so that an injection nozzle may be attached. The machined-steel nozzle is perforated so that the slurry is properly distributed in a 360-degree arc into the soil (Figure 2.2-4).

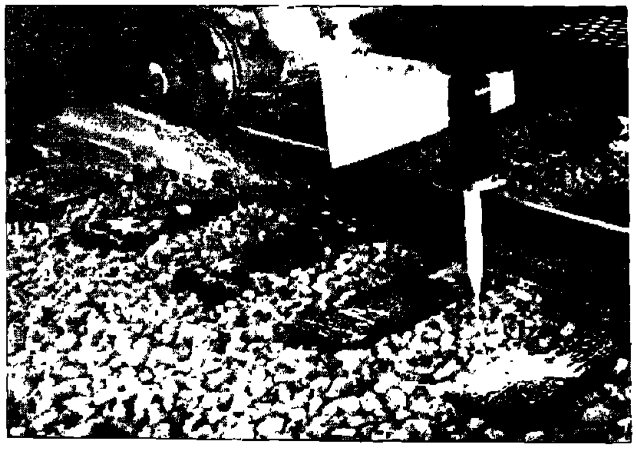


Fig. 2.2-4. Lime injection nozzle.

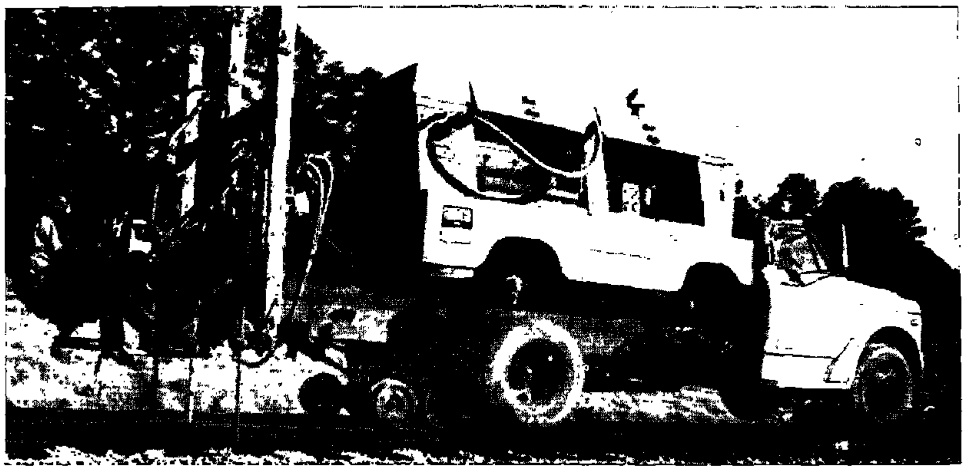


Fig. 2.2-5. Pneumatic drill truck.

Pneumatic Drill Truck

A relatively new piece of equipment for lime injection is the pneumatic drill truck (Figure 2.2-5), which is equipped with rock drills, compressors, and hy-rail wheels. The rock drills are aligned to produce a hole pattern that matches the hole pattern of the standard injection truck. The drill truck is used to perforate cement-stabilized soil or other previously placed hard-surface grouts prior to injection.

Slurry Mixing

The on-site mixing of lime slurry is one of the more difficult steps in the injection process. According to information obtained from the contractors' weekly report forms, the average amount of lime used per railroad mile in 1975 was 158 tons. When mixed with water, this would yield approximately 125,000 gallons of slurry per mile. The logistics of obtaining water and lime in such large quantities on a rigid time schedule and in remote areas sometimes are very taxing. The operation requires durable equipment and considerable prior planning.

In addition to the physical difficulty of on-site mixing, there is the requirement that the lime slurry be proportioned and maintained at the proper consistency. Field experience with applying LSPI to roadbeds has shown that the optimum range for the lime-water ratio is usually 2½ to 3 pounds of lime per gallon of water. Site conditions will require that the contractor adjust the ratio within this range. In some instances, it may be necessary to increase or reduce this range; however, the lime should never exceed 4 pounds per gallon of water.

Achieving the proper slurry consistency is relatively simple when the lime slurry tank is used. After 20 to 24 tons of lime (the capacity load of a bulk transport) have been transferred to the tank, the tank is filled with water to a prescribed level, producing slurry of the desired ratio of lime per gallon of water.

More care must be taken when using the smaller blending trucks. The tank of the truck is first filled with water, and then dry lime is pumped from the bulk storage truck until the proper consistency is obtained. Because it is not possible to weigh the lime as it is transferred into the blending truck, another method of proportioning the lime to the water must be used.

Two methods have been recommended for checking the consistency of the lime slurry: the hydrometer method and the Baroid Scale method. While both methods have been used in the past, it is felt currently that the Baroid Scale method is the more accurate. The Baroid Scale is not sensitive to temperature changes, requires less skill to operate, and has the same accuracy for thick and thin mixtures. The gravest difficulty with the hydrometer method is that, with varying techniques, the tester can obtain a wide range of specific-gravity readings, especially for a thick mixture. Figure 2.2-6 compares the total slurry weight (Baroid Scale method) and the specific gravity (hydrometer method) with the lime-water ratio.

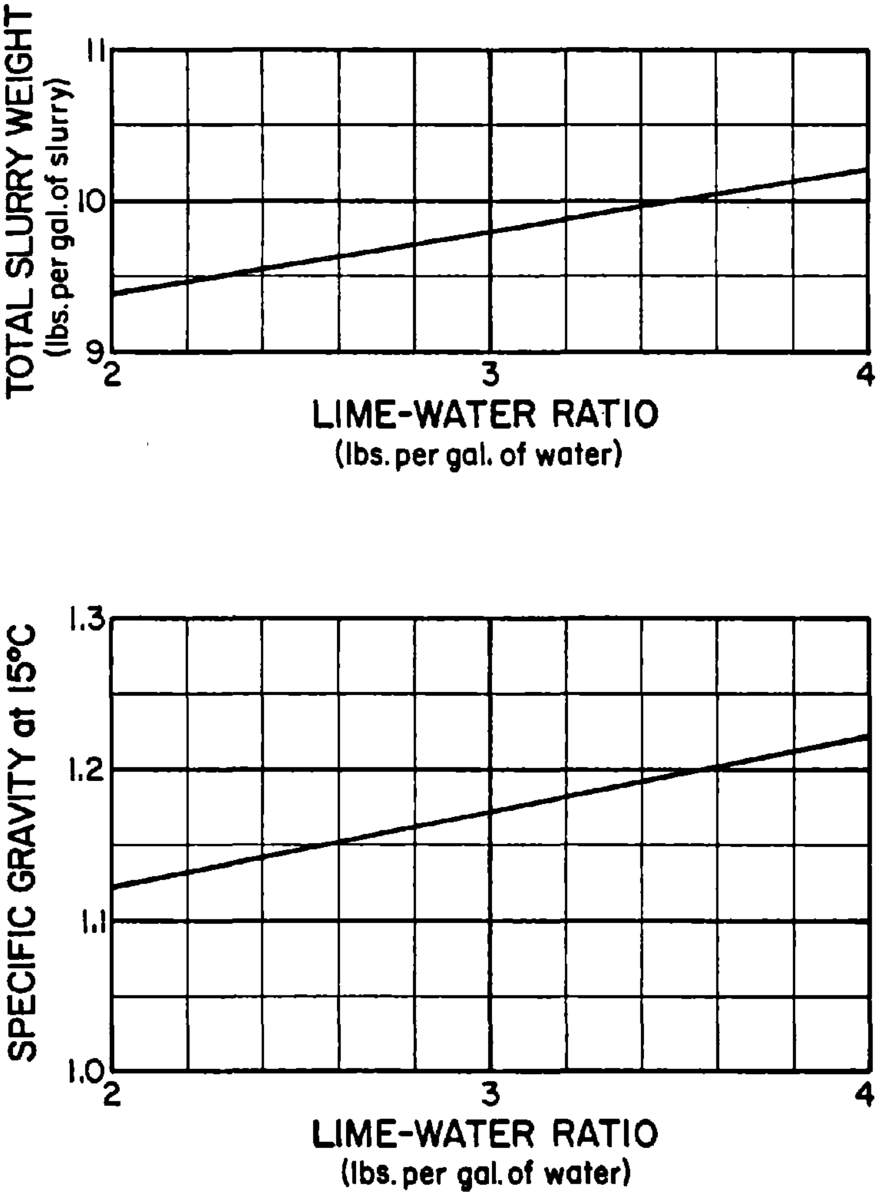


Fig. 2.2-6. Lime-Water Ratio Curves.

Injection

The injection procedures for any particular track section will vary with the roadbed condition and engineering considerations. For example, when injecting a high embankment in arid Wyoming soils (Figure 2.2-7) it may be necessary to use a thin slurry mixture of approximately 2 pounds of lime per gallon of water. However, when injecting a deep cut with standing water in side ditches (Figure 2.2-8) it may be necessary to inject a thicker mixture of perhaps 3 pounds of lime per gallon of water. It is necessary to have sufficient water in the slurry to carry the lime particles into the ground and then be available to support the chemical reactions. In addition, in dry swelling clay soils, it is best to provide enough water to swell the clays and, therefore, stabilize them at a higher moisture content.

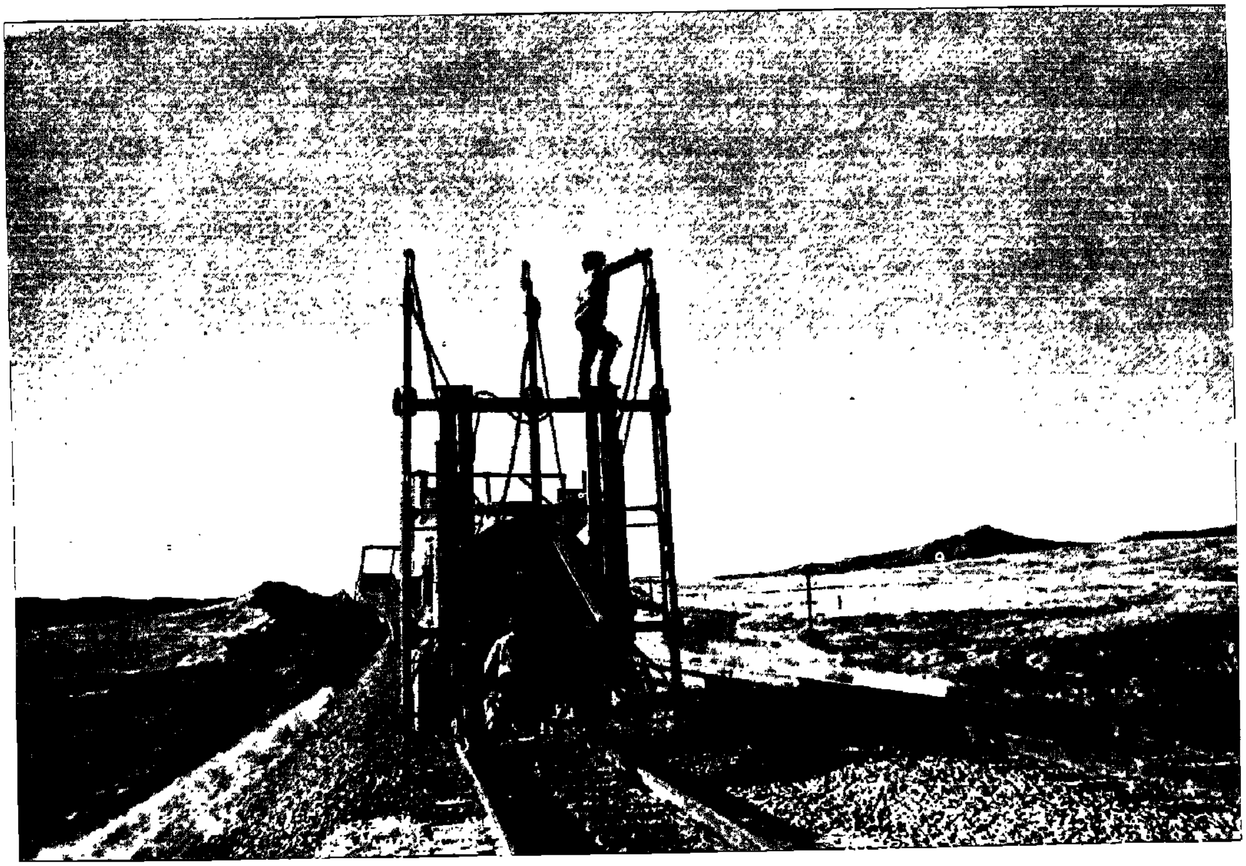


Fig. 2.2-7. Lime injection in progress in Wyoming.

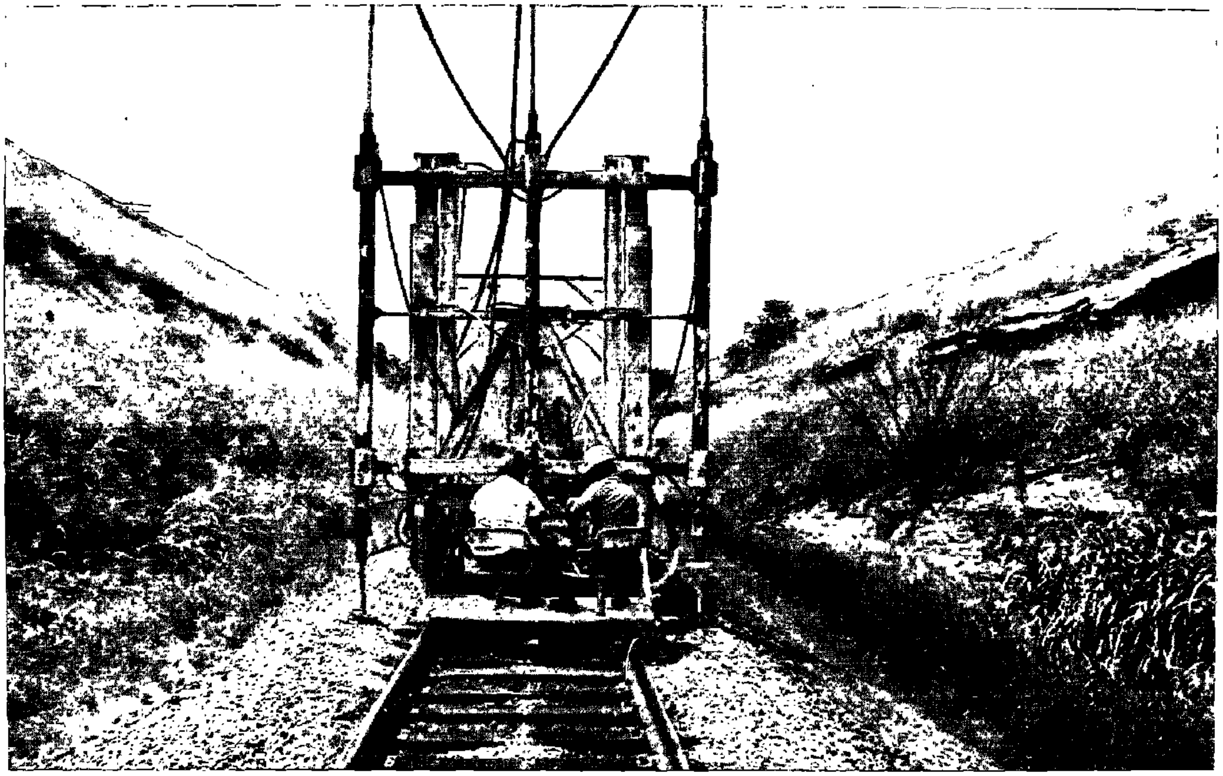


Fig. 2.2-8. Lime injection in a typical deep cut problem area in Oklahoma.

The injection operator sits or stands at a control console on the rear of the injection truck with a clear view of the equipment, which is necessary for accurate control and quick reaction (Figures 2.2-9 and 2.2-10). The operator carefully positions the truck at each injection set-up point. He then operates a hydraulic valve to lower the injection rod to the proper depth and operates the flow valve to allow the slurry to be pumped into the soil from the holes in the injection nozzle. Each rod is lowered farther and the slurry flow continued until the injection at that set-up point has been completed. The flow is then stopped and each injection rod raised so that the truck may be advanced to the next set-up point. The operation at each set-up point is conducted in a somewhat continuous manner, with first one injection rod being lowered a bit and then the next and so on until the total depth is reached on each rod. Studies have shown that each injection setup requires from 3 to 5 minutes, depending on

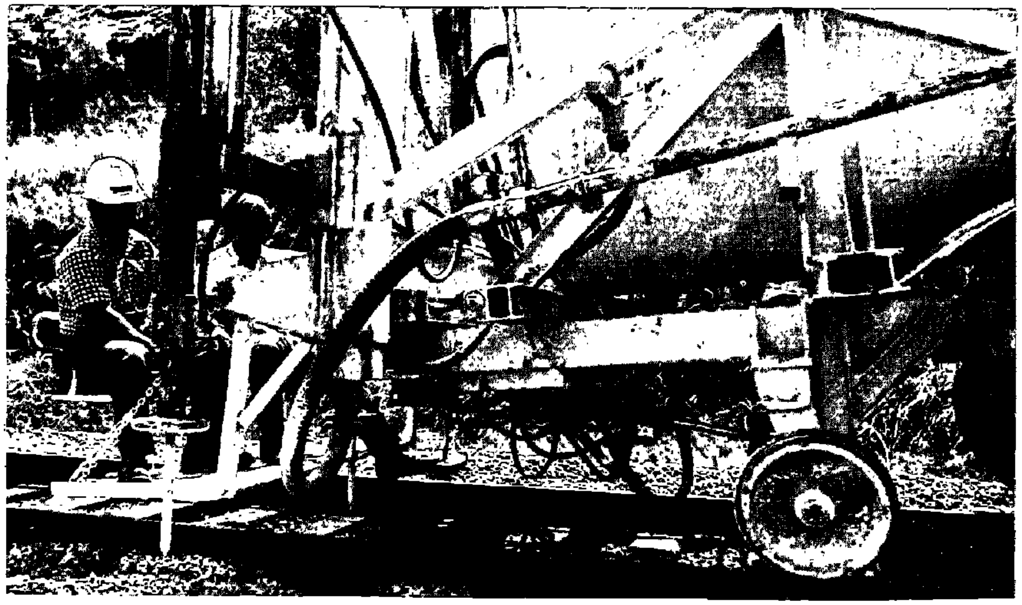


Fig. 2.2-9. Side view of operator's position at rear of injection truck.

the operator and soil conditions. Of this time, 10 to 15 seconds are required to move the truck the distance forward to the next set-up point.

To gain the most benefit from lime injection, it is essential that the injection operator be given technical directions specifying the depths to inject and the quantity of slurry to pump. The nature of injection equipment makes it easier to inject more slurry at deeper levels because there is less chance of a surface breakout. This may be exactly what should be prescribed if the injection area involves a weak or unstable deep problem and a strong, stable upper roadbed. In many cases, however, the problem soils are near the surface and the deep soils require little or no treatment. In these cases, the operator must use more difficult techniques to place the majority of the slurry in the shallow problem soil.

Both surface and subsurface soil exploration and soil testing are usually necessary to determine where the problem soil is located and to define the soil

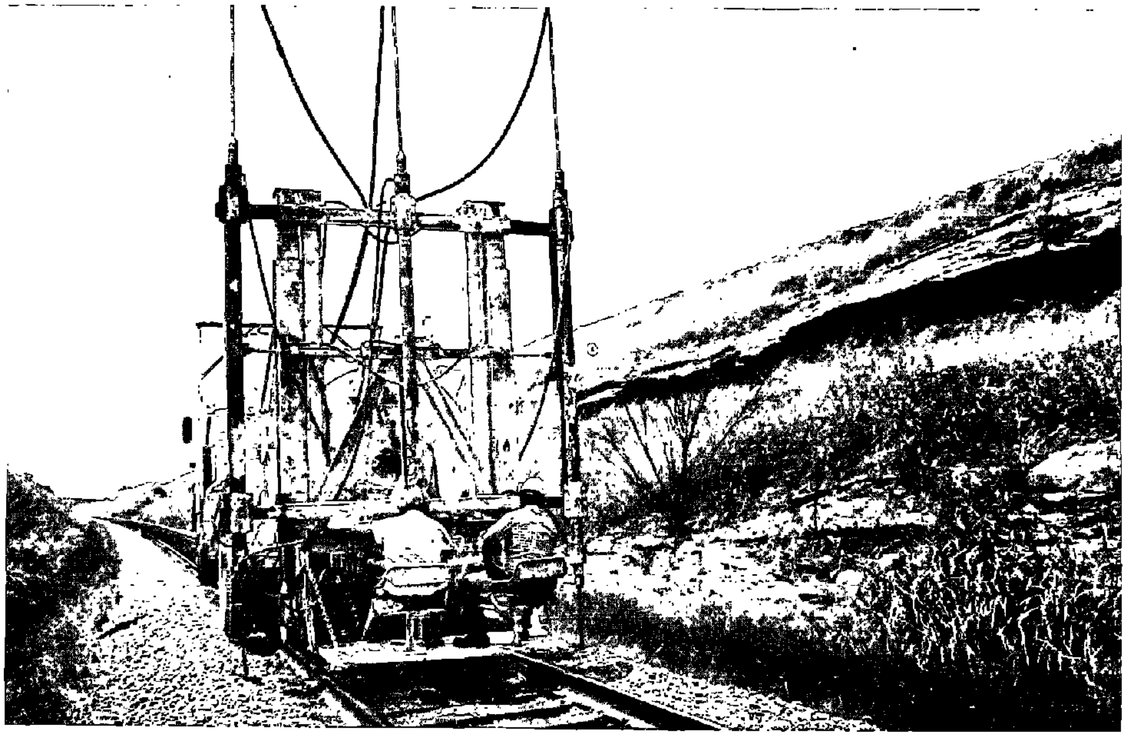


Fig. 2.2-10. Rear view of operator's position on injection truck.

layers to be injected. With information from a soil exploration program, the soils engineer, the railroad engineer, and the contractor working as a team should prepare the injection plan. Each member of the team should study the problem and all available related data prior to developing the plan, which will include the injection specification. The specification will include data for the control of the depth of injection and the quantity of lime to be injected. The plan should not only indicate the total depth; it should specifically indicate which soil layers are to be injected and with how much slurry of what consistency. This degree of accuracy will be difficult to achieve in most cases, but it should be the goal of those writing the specification and instructions to be as specific as practical.

The other injection parameters--such as spacing, interval, pressure, and flow rates--will need to be adjusted to achieve the above prescribed depths of injection and quantity of injected lime.

The injection spacing, which is usually set at every second or third tie, should be varied to achieve the proper quantity of lime slurry at the proper depth. In some cases it may be necessary to "double inject" to place the desired amount of lime at that depth. The procedures for double injection have not been thoroughly documented; however, various methods have been tried with some success. Perhaps the method most used is that of staged injections, i.e., after the initial injection to refusal, the contractor waits a minimum of 48 hours and then re-injects between the original injection holes. The other methods are:

1. Inject every other tie to full depth and to refusal for a distance of 200 or more feet and then back up and repeat the injections for the in-between tie spaces.
2. Inject every other tie to a shallow depth only and return a few days later for full-depth injections.
3. Inject every second or third tie as a normal operation and return months later to re-inject. (This obviously would be much more costly.)
4. For the shallow problem only, inject a limited amount of slurry--not to refusal--and then, hours or days later, repeat until the proper amount of slurry has been injected into the soil.

The vertical injection interval is a much maligned term. In the early literature on lime injection, it was generally stated as varying from 12 to 18 inches. The optimum distance for the injection interval depends to a great extent on the soil structure and how quickly the soil will reseal itself around the injection rod after the rod is advanced. However, it may not be necessary to control this parameter as long as there is strict control of the prescribed quantity of lime slurry injected at each proper depth within the unstable soil layers. If the problem soil is uniformly distributed to the total depth, then a small, uniform interval such as 18 inches would need to be prescribed and adhered to. It then would be necessary to inject approximately the same quantity of lime at each interval and to adjust the injection procedure to achieve the specified total amount of slurry to be injected per track foot.

No significant influence on the injection procedure has been consistently observed for various changes in pumping pressure. Currently, most specifications recommend the use of 150 pounds per square inch of pressure at the pump. It is possible that this may be shown to be an important parameter in future studies; however, additional data will be required in this area before more definitive criteria may be developed. It is suggested that pressure be within a range of 50 to 250 pounds per square inch.

One other critical item concerns the technique of injecting slurry to refusal. Does the operator stop the flow at the first trickle of lime or wait for more signs of lime breakouts and for the lime to flow freely on the surface? The manner in which this is handled will greatly affect the quantity of lime placed unless the inspector requires the operator to adhere to a predetermined specific quantity of lime to be injected. In any case, it will be found that different roadbed soils react differently and trial-and-error injections will be necessary to determine the best procedure.

2.3 SOIL EXPLORATION AND TESTING

Application of the LSPI method of stabilization to a section of problem track should be based upon a thorough soil investigation, including both surface and subsurface exploration. A detailed surface exploration often will provide preliminary identification of the problem. Subsurface exploration, soil sampling, and laboratory testing will help verify the identity of the problem and indicate whether LSPI has the potential to improve the roadbed soils. If the use of LSPI is indicated, the data obtained from exploration and testing will serve as a basis for preparing the injection specification.

The site exploration and testing technology portions of this research program were developed principally through the activities associated with the case study projects and in a smaller part through the research conducted individually by graduate students and engineering consultants. In the beginning all the soil testing procedures used to conduct the case studies were those that had been developed prior to the start of the research program. The development of soil testing standards was not addressed as a separate study project; therefore, this portion of the final report will not include discussion on the development of soil testing and site exploration. The reader is referred to Chapters 3 and 4, which include data from the numerous testing programs conducted in support of the case studies and the graduate student research.

A complete, concise discussion of surface and subsurface soil exploration and testing is included in the lime injection handbook, and a portion of that data is included in the remainder of this chapter. The handbook also includes the standards, specifications, and procedures for the recommended LSPI evaluation soils laboratory tests.

Surface Exploration

Most squeezes, differential soil movements, and embankment failures can be broadly classified as resulting from two different, but often related, problems: low strength and volumetric instability of the embankment soils. The information obtained during a surface exploration together with historical data from railroad maintenance records will help indicate if there is a strength problem or a volumetric stability problem or both. Subsurface exploration will aid further in identifying the nature of the problem.

Surface exploration should include a detailed visual inspection of the problem track area and the surrounding terrain features (e.g., embankment, drainage ditches, adjacent fields). Photographic records and detailed sketches of the problem track area should be prepared. A series of cross-sectional elevation measurements at intervals close enough to describe the important changes in topography provide additional important information.

Subsurface Exploration

Laboratory testing of soil samples obtained by drilling will indicate the nature and engineering properties of the roadbed soils. Soil drilling usually can be best accomplished with a standard highway-type drill truck equipped with hy-rail wheels (Figures 2.3-1 and 2.3-2). In some instances, drilling can be accomplished with a rubber-tired truck; however, for general mobility, the hy-rail vehicle has proven best.

It often is a good rule to locate the first boring in the middle of the problem section. This exploratory boring should extend below the water table. The engineering can closely monitor the boring and determine a reasonable depth at which to terminate the subsequent borings. For example, if the water table is found to be very deep, the subsequent borings might not need to penetrate it.

Fig. 2.3-1. Drilling rig mounted on hy-rail wheels.

For the actual drilling operation, it is considered good practice to:

(1) Obtain undisturbed samples according to ASTM D 1587-74.

(2) Obtain continuous Shelby tube samples for a distance of 5 feet just under the ballast and at regular or selected intervals to completion of the boring.

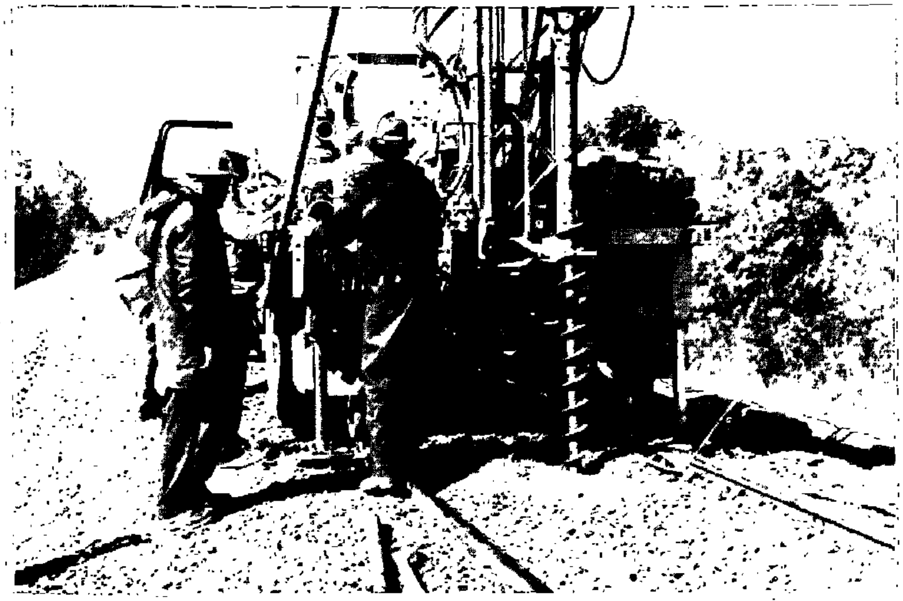


Fig. 2.3-2. Drilling in progress.

(3) Obtain bag samples wherever it is not possible to obtain undisturbed samples. This includes that portion of the roadbed containing ballast, small gravel, and silt. It is important to log this zone.

(4) Determine the elevation of the water table.

(5) Determine Standard Penetrometer values in loose material. (These values can be used as a guide in achieving a subjective determination of the nature of the problem at the site.)

Close study of the extrusion of the samples from the Shelby tubes will yield important information. The extrusion process should be supervised by a soils engineer or technician experienced in identifying sand or silt lenses, seams, cracks and fissures, root lines, voids, slickensides, and other means by which the slurry could be expected to travel extensively through the soil mass. This information is essential in making the final decision regarding injection.

The preparation and interpretation of soil and moisture-content profiles is an important aspect of the subsurface exploration. The soil profile should be plotted to a reasonable scale, showing important surface features and each soil layer. The plotting of a moisture-content profile, either on the soil profile or as an overlay to the soil profile, is good practice. Such a profile is a ready reference for determining zones of elevated moisture content in relation to the soil profile and will help to determine the injection depths when writing the injection specifications. Figure 2.3-3 is an example of a soil profile showing the moisture contents and other soil test results.

Soil Testing

Soil testing for LSPI stabilization of roadbeds can best be described as a developing technology. The purpose of the testing program is to determine whether LSPI will improve the roadbed soils and to guide in preparing injection specifications. Although the suggested tests will give some data that will, in effect, indicate the soil improvement; it is not possible at the present time to obtain a one-to-one correlation between laboratory results and the precise degree of success in the field.

The development of yes-no tests for the use of LSPI is still in the preliminary stage. However, researchers have made a significant contribution to LSPI testing by developing and refining "lime inoculated" testing. This procedure, which attempts to simulate the LSPI field conditions, involves inoculating soil samples with lime slurry. The results of tests on the inoculated samples and on the control samples are then compared.

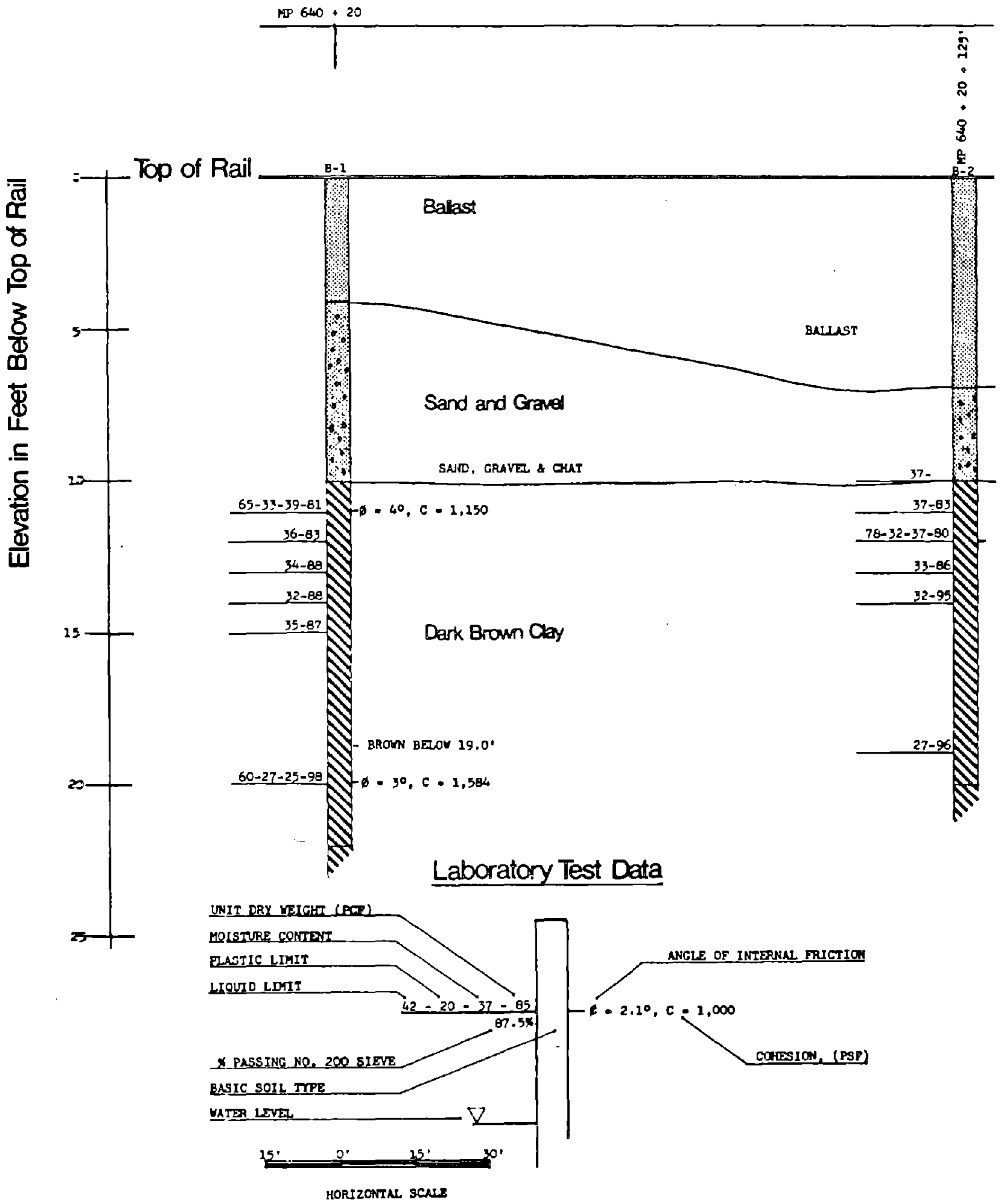
The amount of lime used in inoculated testing is 1 percent or less of the soil dry weight. This has been determined to be the amount of lime generally injected during railroad LSPI operations, based on injections on 5-foot centers. Just as it may be necessary in the field to double inject or to reduce the space between injections to compensate for certain soil conditions, it may be necessary to modify the tests to account for the same conditions. All of the tests are readily adaptable to these situations.

Inoculated samples may be used in swell, consolidation, triaxial, and unconfined compression testing. The tests that have been used in railroad LSPI applications are described below and presented in tabular form in Table 2.3-I.

Preliminary Soil Tests

The two preliminary tests should be performed according to standard specifications, except that the treated samples containing 1 percent by weight of intimately mixed dry lime are compared with control samples containing no lime.

Atterberg Limits. A positive result from this test, which is a combination of the Liquid Limit and Plastic Limit tests, is a reduction of the Plasticity Index (PI). Generally, the liquid limit can be lowered by no more than


MP 640 + 20
MP 640 + 20 + 125'
Elevation in Feet Below Top of Rail
Top of Rail
B-1
B-2
0
5
10
15
20
25
Ballast
BALLAST
Sand and Gravel
SAND, GRAVEL & CHAT
Dark Brown Clay
65-33-39-81
36-83
34-88
32-88
35-87
60-27-25-98
ø = 4°, C = 1,150
BROWN BELOW 19.0'
ø = 3°, C = 1,584
37-
37-83
78-32-37-80
33-86
32-95
27-96
Laboratory Test Data
UNIT DRY WEIGHT (PCF)
MOISTURE CONTENT
PLASTIC LIMIT
LIQUID LIMIT
42 - 20 - 37 - 85
87.5%
% PASSING NO. 200 SIEVE
BASIC SOIL TYPE
WATER LEVEL
ANGLE OF INTERNAL FRICTION
ø = 2.1°, C = 1,000
COHESION, (PSF)
15'
0'
15'
30'
HORIZONTAL SCALE

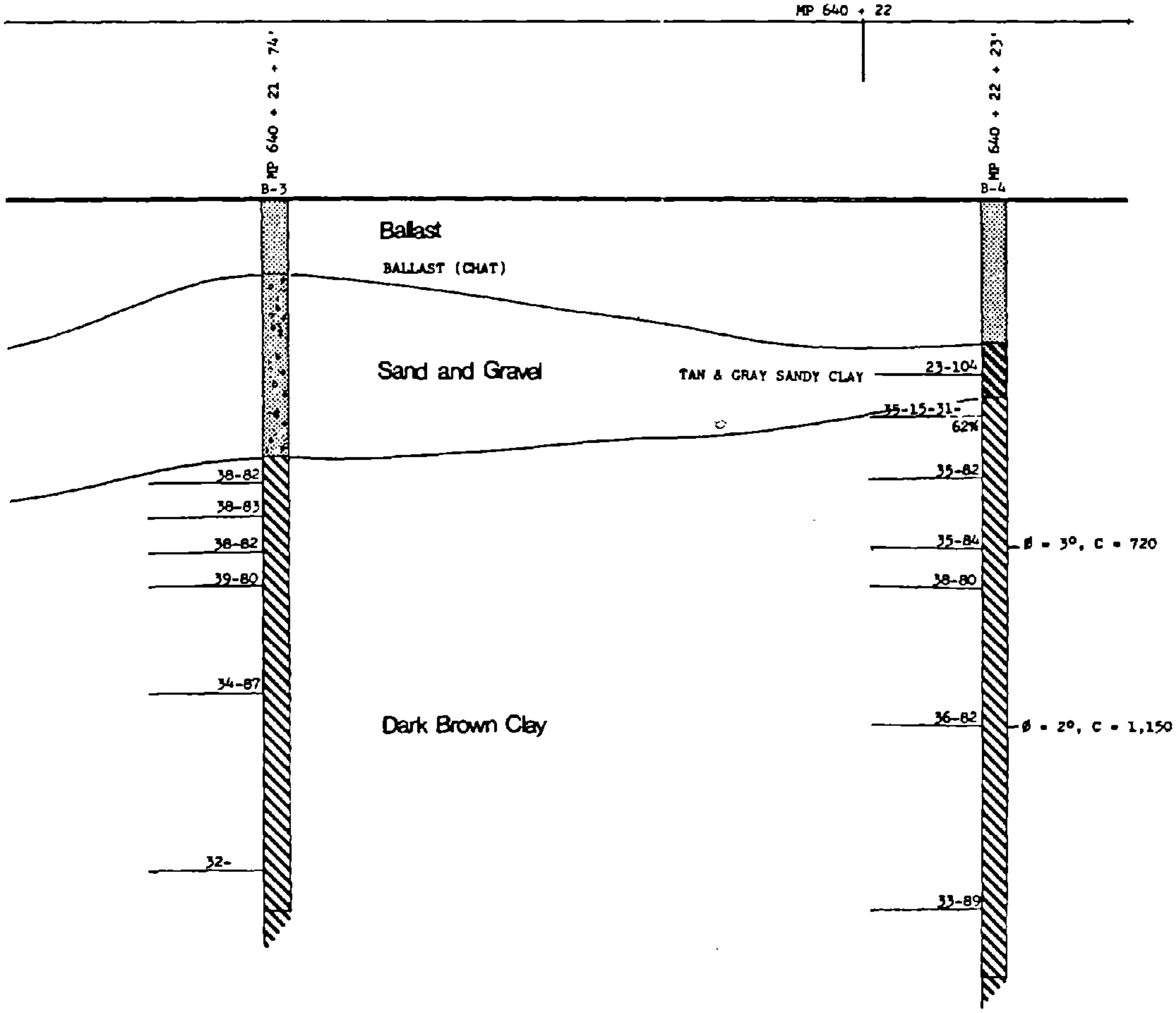


Fig. 2.3-3. Soil profile.

TABLE 2.3-I

LSPI Soil Tests with Positive Results

Group	Test	Positive Result for LSPI
Preliminary	Atterberg Limits	Decrease in Plasticity Inde
	Linear Shrinkage	Decrease in linear shrinkag
Strength	Natural Triaxial	No positive results; data used for comparison only
	Inoculated Triaxial	Increase in average peak strength and in slope of stress-strain curve
	Remolded Triaxial or Unconfined Compression	Increase in average peak strength and in slope of stress strain curve
	Inoculated Consolidation	Apparent increase in preconsolidation load
Volumetric Stability	Volumetric Shrinkage	Decrease in the amount of shrinkage
	Inoculated Free Swell	Decrease in the percent swell
	Inoculated Consolidation	Increase in the modulus of compressibility

approximately 2 percent, so the major change must occur in the plastic limit. There are no criteria for ascertaining how great a reduction in PI is necessary before it may be termed a significant improvement. Whether the improvement is significant will depend upon the type of soil, the other test results, and the judgment of the engineer. Reductions in PI ranging from 5 to 15 have been obtained in soils judged reasonably responsive to LSPI treatment.

Linear Shrinkage. Any reduction of shrinkage detected in this test is a positive result. Generally, reductions of 5 to 10 percent indicate that LSPI has a good chance of reducing shrinkage in the field.

Soil Strength Tests

Natural Triaxial. Triaxial compression tests on natural, undisturbed samples (unconsolidated, undrained) are recommended to ascertain the *in situ* strength of the soil mass. The soil strength must be compared with the stresses caused by train loads and overburden pressures. If there is no accurate way to determine soil stresses, either through calculations or field tests, the results can be interpreted only subjectively as to whether the soil has a low, medium, or high strength. However, this is necessary and useful information for determining whether the soil has the strength to support the loads or whether the track system must be modified (e.g., by increasing the ballast depth) to reduce soil pressures.

Inoculated Triaxial. The purpose of this test is to determine whether LSPI will produce a strength gain in the soil mass. Positive results of this test are those indicating that the treated sample (inoculated with lime slurry) is stronger than the control sample (inoculated with water). A strength increase of greater than 5 percent is generally required.

Remolded Triaxial or Unconfined Compression. These tests, comparing remolded samples using either (1) supernatant liquid from lime slurry or

(2) lime slurry with remolded samples using only water, have the advantage of requiring less soil than do some of the other tests. However, because these tests require remolded samples, natural triaxial testing is necessary to provide supporting data. Comparison studies of the resulting stress-strain curves give a good indication of whether the remolding has radically changed the soil characteristics. A dramatic shape change would indicate that the remolding is not a successful method of testing for the particular soil. A strength increase of 5 to 10 percent or greater is a positive result.

Inoculated Consolidation. This test compares the consolidation characteristic (i.e., the void ratio versus the log of the applied stress) of soil samples inoculated with lime slurry with that of soil samples inoculated with water. The inoculated consolidation test is considered to give the most definitive, most consistent information of all the tests discussed in this section. The best method of interpreting the data from the test is outlined below.

Typical consolidation characteristics for an LSPI-treated foundation soil are shown in Figure 2.3-4a. Researchers have developed a diagnostic laboratory test (inoculated consolidation) that produces results (Figure 2.3-4b) that closely match those determined for the LSPI-treated soil. In interpreting the data of Figures 2.3-4a and 2.3-4b, the following results of treatment can be observed:

(1) The slope of Part I of the curve is less for the inoculated soil than for the natural, or control, soil.

(2) The slope of Part II of the curve is greater for the inoculated soil than for the control soil, and the inoculated curve approaches the control curve at higher loads.

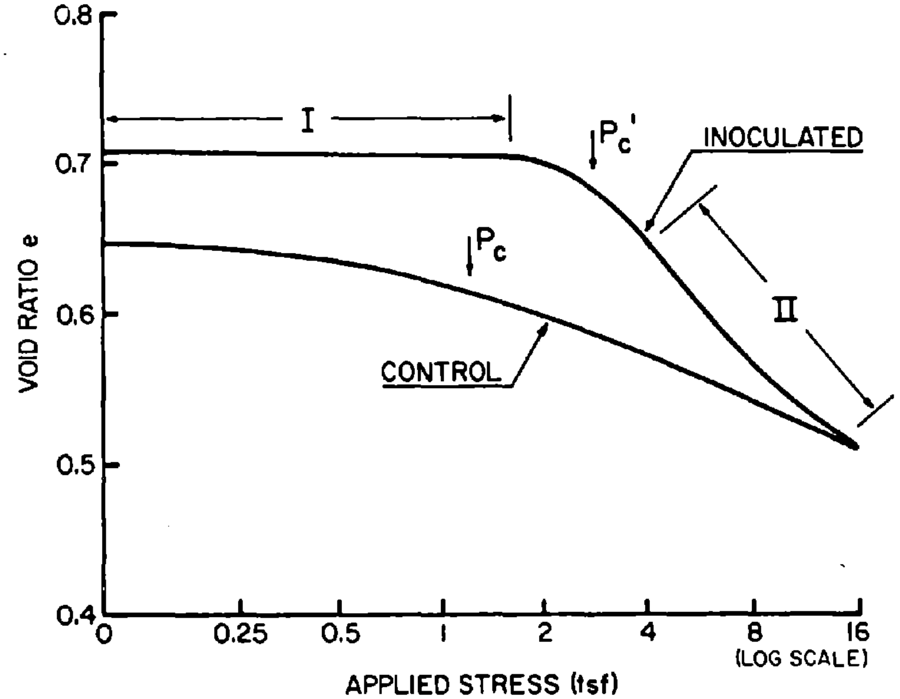


Fig. 2.3-4a. Actual consolidation test results from a lime-injected area.

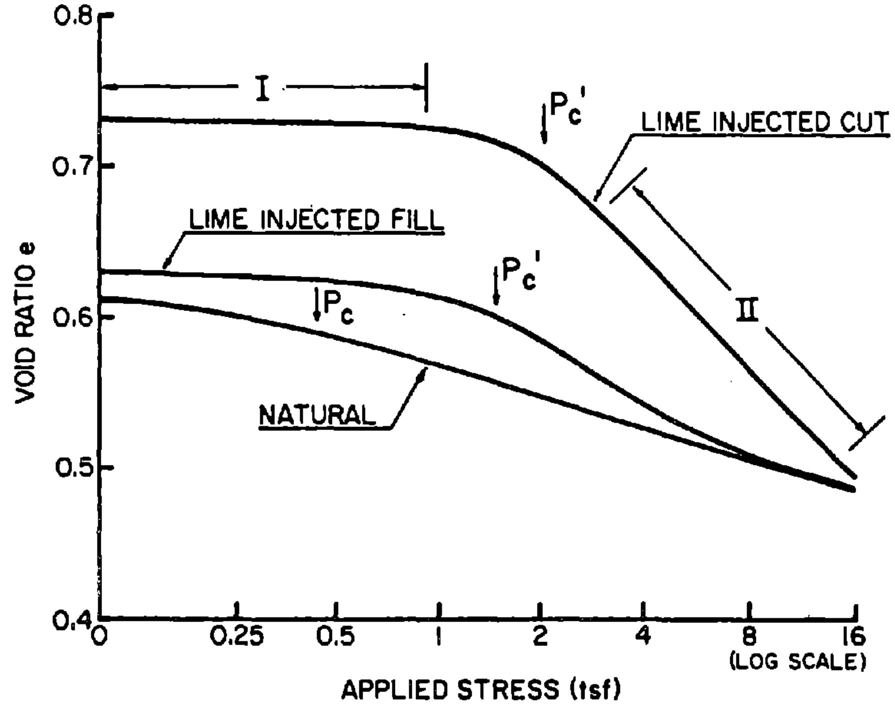


Fig. 2.3-4b. Typical inoculated consolidation test results.

(3) The preconsolidation load for the inoculated soil (P_c') is greater than that for the control soil (P_c). This is sometimes referred to as an apparent increase in preconsolidation load.

The consolidation characteristic for the inoculated soil exhibits the benefit of the cementing of particles that have reacted chemically with the lime, i.e., a reduced rate of consolidation [see Result (1) above] or an increase in the modulus of compressibility of the soil. At greater loads, this curve shows an increase in the rate of consolidation [see Result (2) above], indicating that the cementing of the soil particles is breaking down and that the soil is reverting to the characteristic of the control soil.

It is not currently possible to set a range of changes in the consolidation parameters that give positive indications of the success of LSPI. However, data from inoculated consolidation testing that exhibit the cementing results shown in Figure 2.3-4b are a positive indication for success of LSPI. Results (1) and (2) are significant in both volumetric stability considerations (increase in the modulus of compressibility) and strength considerations. Result (3), the apparent increase in preconsolidation load, is an indication of the increase in soil strength.

Volumetric Stability Tests

Volumetric Shrinkage. For this test, samples intimately mixed with 1 percent dry lime are compared with untreated samples to obtain results similar to those produced by the linear shrinkage test. However, this test provides further information regarding volumetric shrinkage, rather than linear shrinkage. The results can be interpreted in the same way as in the linear shrinkage test.

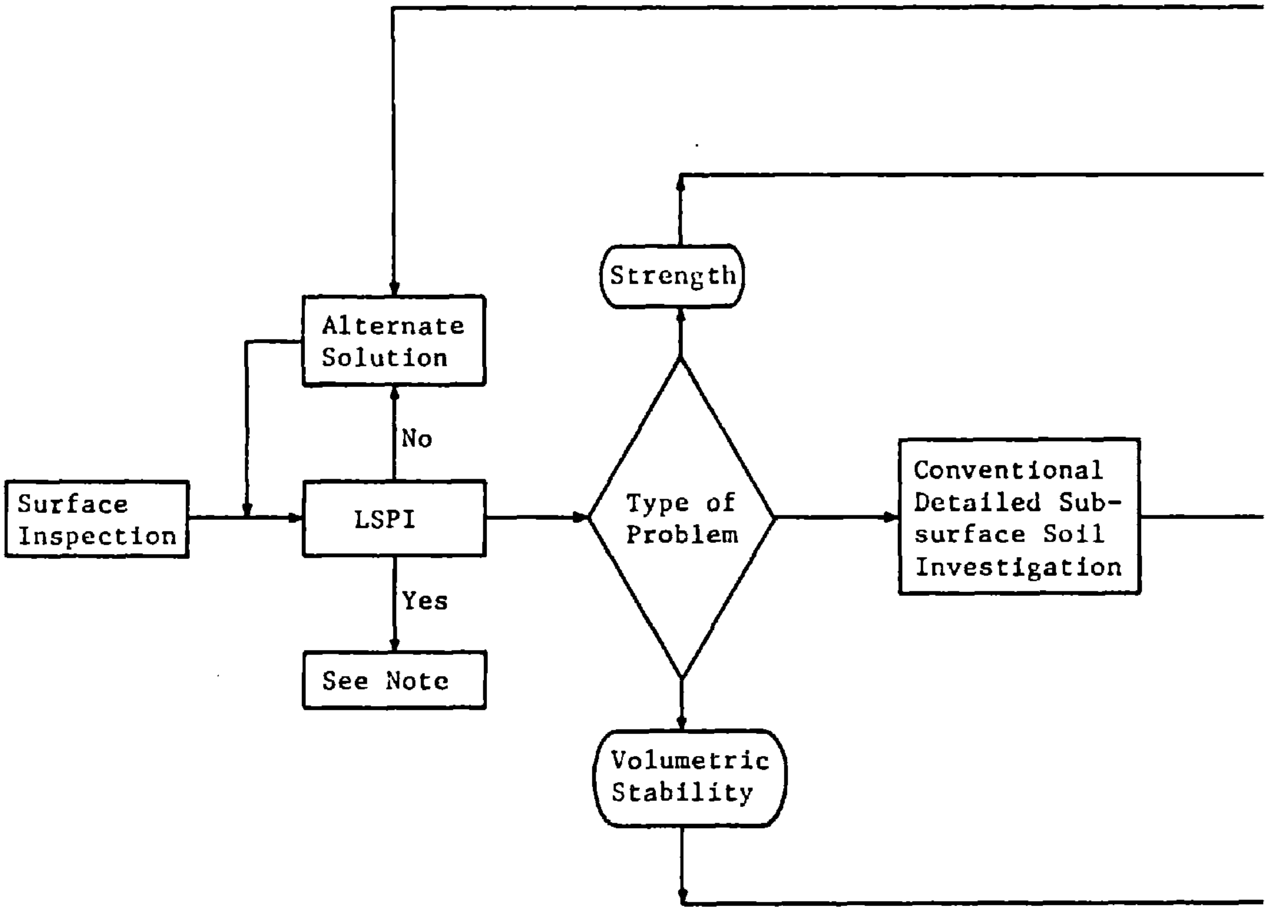
Inoculated Free Swell. Treated samples are inoculated with lime slurry, and control samples are inoculated with water. A net reduction in swell of 5 percent or greater due to the treatment is a positive result.

Inoculated Consolidation. This test, which is discussed above under Soil Strength Tests, also has volumetric stability considerations. These are described in the previous section.

The Decision Process

The ultimate question faced by the soils engineer who is contemplating the use of LSPI is: Will the injection of lime slurry make a positive improvement in the soil mass? In compiling the data on which to base his answer to this question, the engineer must make numerous decisions, beginning with the surface exploration of the site and culminating in the evaluation of all the data, especially the information obtained from the appropriate tests. The flow chart in Figure 2.3-5 has been devised to guide the engineer through this decision process.

After the tests have been performed, the engineer will be faced with making a yes-no decision on the use of LSPI based on the test results and all other available data. In assessing the test results, the engineer should credit as a "yes" any positive improvements. If no improvement is detected by a test, a "no" should be registered. While a "no" result does not indicate that LSPI will be bad for the site, it does mean that the laboratory test gives no encouragement for the prospects of positive soil improvement. In most cases, several "no" answers will lead the engineer to conclude that LSPI should not be recommended; and if all treatment-type tests give no indication of improvement, LSPI definitely should not be recommended. Because of the large number of possible variables in this type of testing, statistical analysis of the data is often of considerable benefit.



NOTE: In some instances (e.g., spot treatment), it may be more economically viable to base the decision to use LSPI purely on the basis of the surface inspection. This is recommended only when the cost of the laboratory analysis is comparable with, or exceeds, the cost of injection.

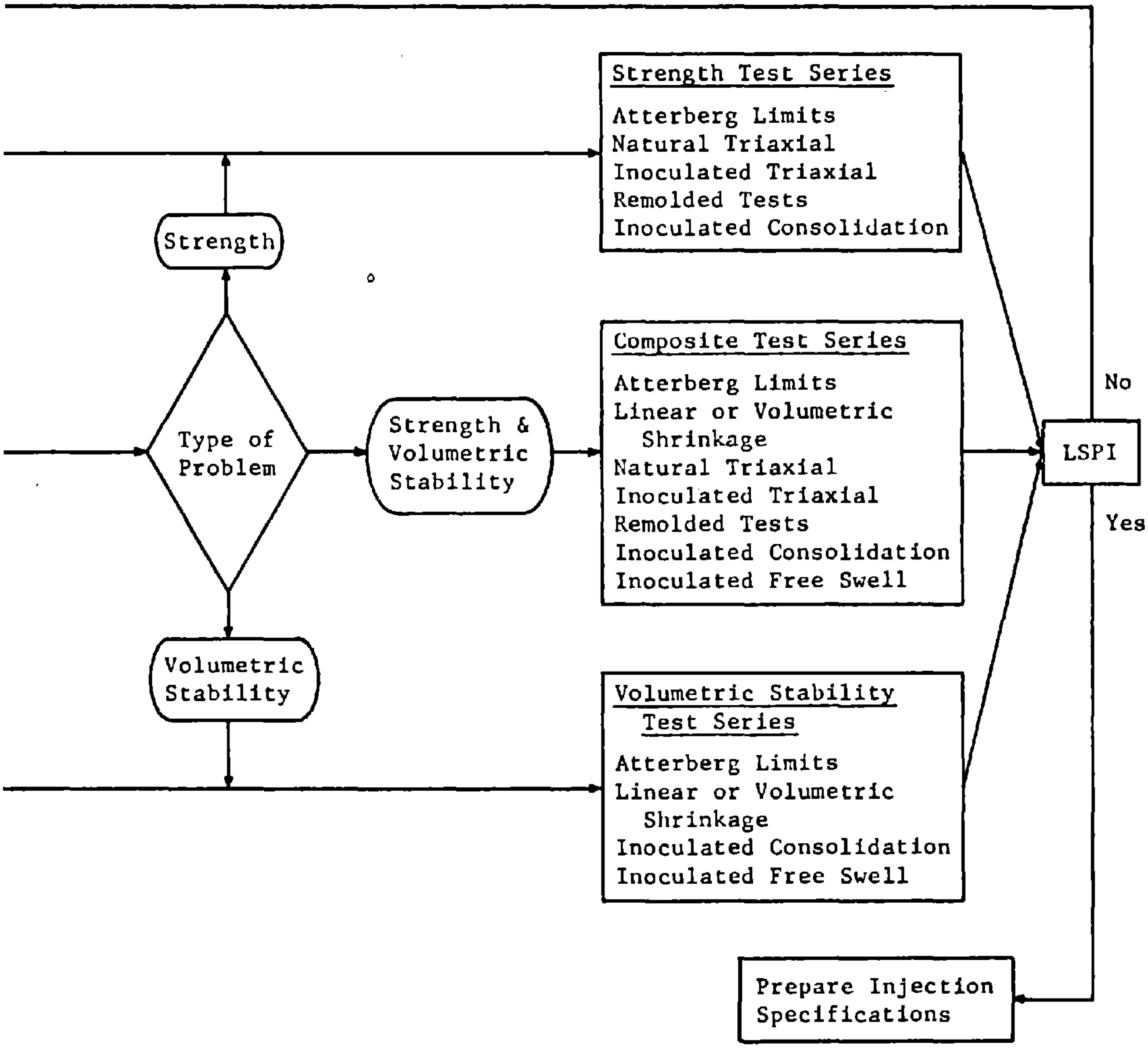


Fig. 2.3-5. Decision flow chart.

Interpretation of Results

Interpretation of the data obtained from the appropriate tests is not a simple task because the mechanisms by which LSPI stabilizes the soil are not totally understood. Also, some of the tests more closely simulate field conditions than do others. For example, inoculated testing better simulates the LSPI treatment of the *in situ* soil than does remolding. Thus, strength increases indicated by the addition of lime slurry in remolded testing must be interpreted in conjunction with other data. The particle size of the soil (i.e., clay, silt) and the existence of fissures and cracks must be considered because it is unlikely that lime particles will be transported very far into the soil mass if the soil is a heavy or fat clay and if no flow paths exist.

Furthermore, any improvement shown in the tests is only an improvement in the quantities measurable in a laboratory on a laboratory-sized soil sample. The soil sample is not an exact model of the soil mass. For example, the effects of any cracks in the samples will be magnified because the samples are small. Also, inoculated samples that show certain improvements will not reveal other possible improvements--such as those caused by lime seams and moisture stabilization. Therefore, the results of inoculated tests will generally be conservative.

Data interpretation is further complicated by the fact that some tests have more weight than others in indicating whether LSPI will stabilize the soil. Inoculated consolidation testing has both strength and volumetric stability interpretations; therefore, its results have considerable weight. For strength considerations, inoculated triaxial and remolded triaxial tests give supporting data for inoculated consolidation test results. For volumetric stability considerations, the inoculated free swell test supports the inoculated consolidation test. No decision should be made solely on the basis of the data from the two preliminary tests--Atterberg limits and linear shrinkage--or from the volumetric shrinkage test.

It is for these reasons that a large variety of tests is suggested. Their use and interpretation will depend upon the individual engineer's understanding of the LSPI process and the improvements ascribed to it.

In cases where considerable doubt exists as to the practicality of LSPI treatment, it may be feasible to consider injecting only a small test section of track, perhaps one mile. This method would be cost effective if (1) other sections of track were being injected and (2) the railroad could wait for an extended period of six months to a year to determine whether LSPI improved the soil mass. If this method is selected, an evaluation plan that fully considers the actual source of track improvements must be prepared. For example, a tie-and-surfacing operation often precedes or follows an LSPI treatment. The tie-and-surfacing operation alone provides a better track surface for a period of time, and it may sometimes prove difficult to separate the beneficial effects of that operation from those attributable to LSPI.

Today there is no simple method of obtaining a yes-no answer for all possible LSPI sites. Further research and the development of new tests may provide more answers. However, no one single test now exists that can give a definite answer. The surface and subsurface soil explorations and the tests outlined in the lime injection handbook will aid in obtaining more effective and economical utilization of the LSPI method.

2.4. ECONOMIC STUDY AND STATISTICAL ANALYSES

An economic study or cost-benefit analysis can be used as a management tool to help select the type of subgrade improvement program to be incorporated into the rehabilitation of any problem section of track. Four main categories of savings should be calculated for rehabilitation projects:

1. Savings due to increased train speed,
2. Reduction in future non-discretionary maintenance costs,
3. Savings due to avoidance of derailments, and
4. Savings in local train operating costs.

Of these, the largest proportion of track-maintenance-project savings are attributed to speed-related savings, mainly in upgrading from 10 mph to 30 or 40 mph. It is on those tracks with such low permissible operating speeds as 5-10 mph that lime injection stabilization can most likely be cost-effective.

An attempt has been made during the lime injection research program to evaluate the cost savings that could be achieved by injecting lime into a problem section of track. A considerable amount of data was obtained and evaluated concerning the actual cost of lime injection and the effects of the various injection parameters upon the total cost of lime injection, but in no instance was it possible to determine the cost savings achieved by the user railroad solely as a result of lime injection.

The LSPI method is only one of several possible methods of subgrade improvement. The various rates of application possible with the LSPI method can cause the price to fluctuate and, in most instances, can yield different degrees of success. This then is the area of economic analysis which was addressed during the research program: Given the existence of a subgrade problem that definitely requires repair, determine the cost of lime injecting the subgrade for comparison with other methods of reconstruction.

The price of lime injection varies as a function of several variables, but it is principally controlled by the number of tons of lime to be injected in a given distance. The information in the data collection, storage, and retrieval system indicates that the time required for the contractor to place one ton of lime was between 21 and 61 minutes (average was 29.59 minutes). A linear model was developed to show the effect of various lime injection parameters on the time required to inject one ton of lime:

$$T = -59.2085 + 1.30182N + 16.9193D + 12701.6/P - 2513.84D/P - 3.73935DC + 522.924DC/P,$$

where

T = Time in minutes required to inject one ton of lime (does not include time lost due to delays).

N = Average number of injections required to inject one ton of lime.

D = Average injection depth in feet.

P = Average injection pressure in pounds per square inch.

C = Average slurry concentration in pounds of lime per gallon of slurry.

The linear model has a standard error of 3.48 minutes and a coefficient of determination of .944 (meaning that 94.4 percent of the variation in time was due to the variation of the parameters included in the model).

The total cost to inject one ton of lime can be estimated by

$$\text{Cost} = C_H T_I + C_H T_D + C_L,$$

where

C_H = The contractor's fee per hour.

T_I = T/60, where T is the minutes required to inject one ton of lime based on the linear model.

T_D = The estimated time in hours that work will be delayed per ton of lime injected (averaged .09 hours for collected data).

C_L = The cost of one ton of lime.

The average amount of time required to inject one ton of lime in 1975 is estimated to have been

$$\begin{aligned} T = & -59.2085 + 1.30182(7.0) + 16.9193(10.82) + 12701.6/(127.41) \\ & - 2513.84(10.82)/(127.41) - 3.73935(10.82)(3.01) \\ & + 522.924(10.82)(3.01)/(127.41) \\ = & \ 31.06 \text{ minutes.} \end{aligned}$$

The total cost to inject one ton of lime in 1975 is estimated to have been

$$\begin{aligned} \text{Cost} &= \$140(31.06)/60 + \$140(.09) + \$34 \\ &= \$119.07. \end{aligned}$$

The average amount of time required to inject one ton of lime in 1976 is estimated to have been

$$\begin{aligned} T = & -59.2085 + 1.30182(6.5) + 16.9193(12.17) + 12701.6/(123.7) \\ & - 2513.84(12.17)/(123.7) - 3.73935(12.17)(3.15) \\ & + 522.924(12.17)(3.15)/(123.7) \\ = & \ 29.23 \text{ minutes.} \end{aligned}$$

The total cost to inject one ton of lime in 1976 is estimated to have been

$$
\begin{aligned}
\text{Cost} &= \$150(29.23)/60 + \$150(.09) + \$38 \\
&= \$124.58.
\end{aligned}
$$

Lime usage averaged 158 tons per mile in 1974; 182 tons per mile in 1976; and on several jobs, 225 to 250 tons per mile. The cost of lime is currently approximately $40-$50 per ton, and the contractors are currently charging $125 to $175 per hour for a contractor crew. With this data plus a 10 percent customer delay rate, the cost of lime injection would amount to approximately $12,000 per hundred tons of lime per mile. Thus, for a railroad that needs to stabilize 10 miles of track, the price could vary from $120,000 to $360,000, depending upon the variation in lime usage from 100 to 300 tons per mile.

2.5. ENVIRONMENTAL STUDY

Lime slurry pressure injection presents the potential for adverse environmental effects if reasonable care and precautions are not used in the implementation of this technique. The purpose of this study was to delineate the apparent adverse effects that can result from lime slurry injection and the means of avoiding these problems.

The adverse effects can be categorized into three divisions: physiological, aquatic, and botanical. The physiological effects include potential effects on man and the aquatic biota. Injecting fluids into the ground may contaminate 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 include the fish kills as well as the initiation of algae blooms, caused by the addition of phosphate with a concurrent adjustment of pH in waterways.[1] Botanical effects consist of denuding 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.

The constituents of lime slurry that were investigated for possible adverse effects were trace materials in the technical grade lime, phosphorous present in the lime, the alkalinity of the lime, and the surfactants which are added to the lime. 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 percent phosphate, 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 a significant amount 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 percent phosphate builders. Spillage of lime slurries into surface waters can potentiate eutrophication of these waterways. Lime contains sulfates, which 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.[2,3] 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 with soft waters suggests that this will have a beneficial effect.[4] 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). Depending upon the starting acidity of the water, 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 upon the species of fish present.[5,6] Fish kills have occurred in streams adjacent to lime slurry pressure injection sites. 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, and 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.[3] The native plants will not thrive in soil of this pH. Thus, one 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 visible effects it will have locally.

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 (i.e., some surfactants may be toxic 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. Injecting the slurry into the ground does give rise to a significant risk of well water contamination. To minimize this risk, any additive that is to be mixed into the slurry should be one considered safe for use in water supply systems.

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.

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 researched 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. Controlled applications will be the essential mechanisms of averting adverse environmental impact.

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2.6. FINITE ELEMENT ANALYSIS

Introduction

The accurate determination of the stresses, strains, deformations, and modes of failure of the track structural system is a difficult task for the railroad civil engineer. Better concepts for static and dynamic load spectrums, climatic and environmental criteria, load-history, stress-dependent soil properties, and computerized structural analysis tehniques are necessary for a proper solution of this complex problem. The railroad track system--composed of soil, ballast, ties, joints, and rails--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 essentially 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, development, and application experience will be required prior to the final development of the method as a railroad track analysis tool.

It is only recently that the powerful finite element method has been applied to the static stress analysis solution of the railroad track system. These efforts, conducted principally by graduate engineering students at the Universities of Illinois and Arkansas, were supported by university, industry, AAR, and FRA research funds. In this section, a brief history of track analysis efforts and an introduction of the finite element method are presented.

Analysis Background

A notable track analysis advancement occurred in the years 1914-1925 as a result of the work of the Special Committee to Report on Stresses in Railroad Track, which had Prof. Arthur N. Talbot as its chairman. The work[1,2] of the committee has been the basis for the engineering analysis of track systems for the past 50 years. That this effort should have served so excellently for so long is mainly due to the time and resources allotted 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 complexitiy 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 trial, 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 1920s train-track problems using 1920s engineering technology, and it served the railroad industry well for several years. Today we can better analyze the problem with our more complete geotechnical knowlege, digital computers, new laboratory testing equipment, and new instrumentation for determining in-place field soil measurements. We must apply current engineering technology in the economic and operational environmental of today's railroad industry for the accurate analysis of the modern track structural system.

Railroad Finite Element Analysis

The first paper that specifically addressed the finite element analysis of the railroad track system was written in 1970 by J. R. Lundgren,[3] a graduate student at the University of Illinois. 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 was little activity in this area for several years.

Lundgren's paper offered a good beginning point for the development of a new finite element program under this contract. One other parallel work was in progress in 1974-1975 by S. Tayabji of the University of Illinois.[4] Tayabji developed a two-dimensional program (ILLI-TRACK) that was adapted to analyze the three-dimensional railroad embankment problems by analyzing the longitudinal and transverse cross-sections. His method is similar to that which was developed at the GIT. However, there are significant differences in the finite element constitutive relationships used to model the nonlinear soil effects.

This work was conducted in the beginning principally by B. J. McAlister, a graduate student whose thesis[5] utilized the nonlinear finite element method of analysis. The computer program he developed used a two-dimensional finite element model that approximated the real three-dimensional structure by first analyzing the transverse section of the track structure, Figure 2.6-1, and then logitudinal section, Figure 2.6-2. The effect of the two-phase analysis was to generate stresses, strains, and deflections that were representative of those under the track, which is in reality a three-dimensional solid system. A computer program that was written by Sogge and Richard[6] of the University of Arizona for the analysis of bulkhead stresses, strains, and displacements was selected to be modified for railroad analysis. 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.[7] The SSI computer program was modified by McAlister to allow for the analysis of the track structure system. The modification included the addition of an automatic node generator to help simulate the track system. Actual railroad soil properties obtained from the Rock Island test site were utilized for the stress-dependent material representation.

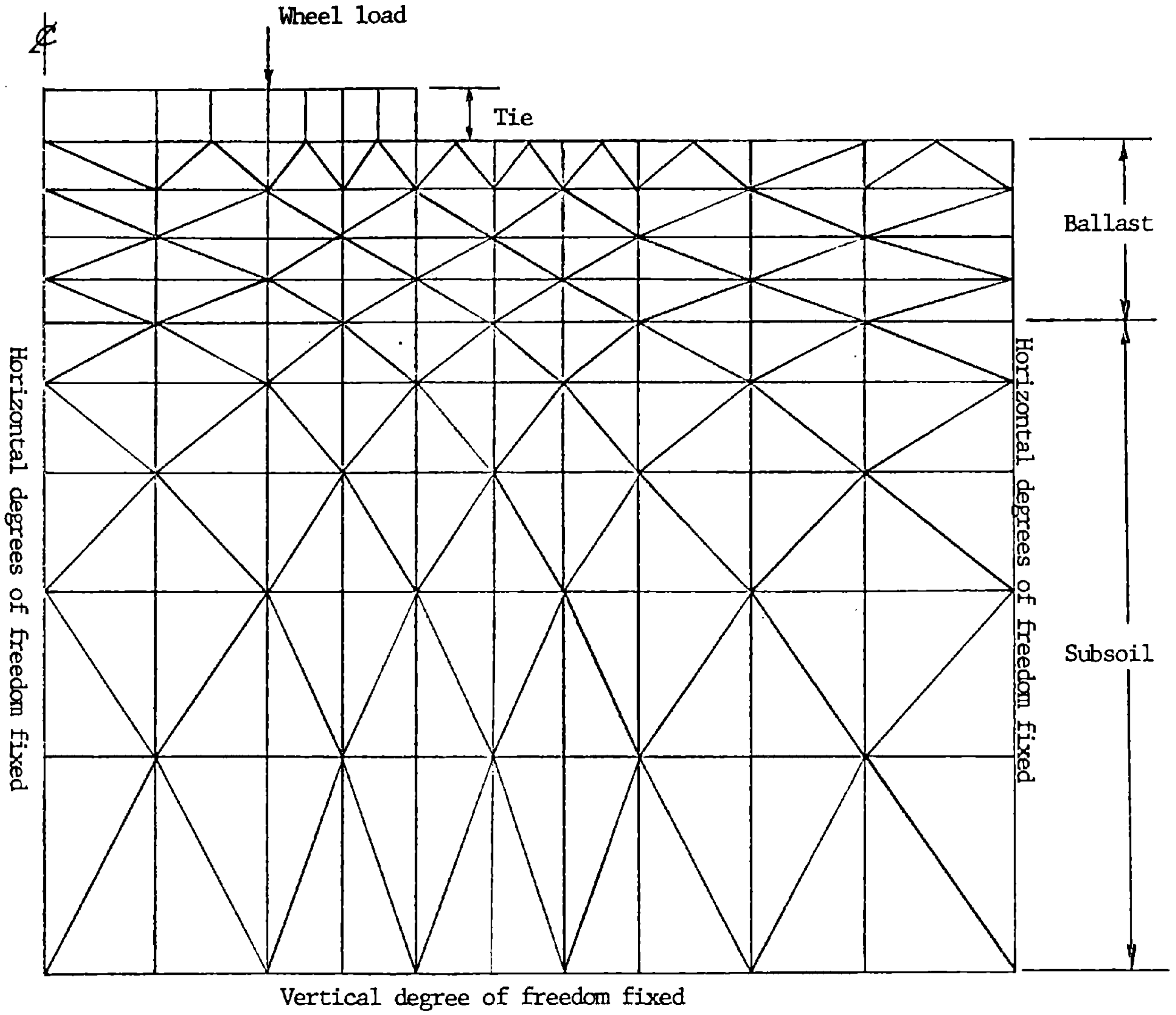

Fig. 2.6-1. Railroad subgrade--transverse simulation.

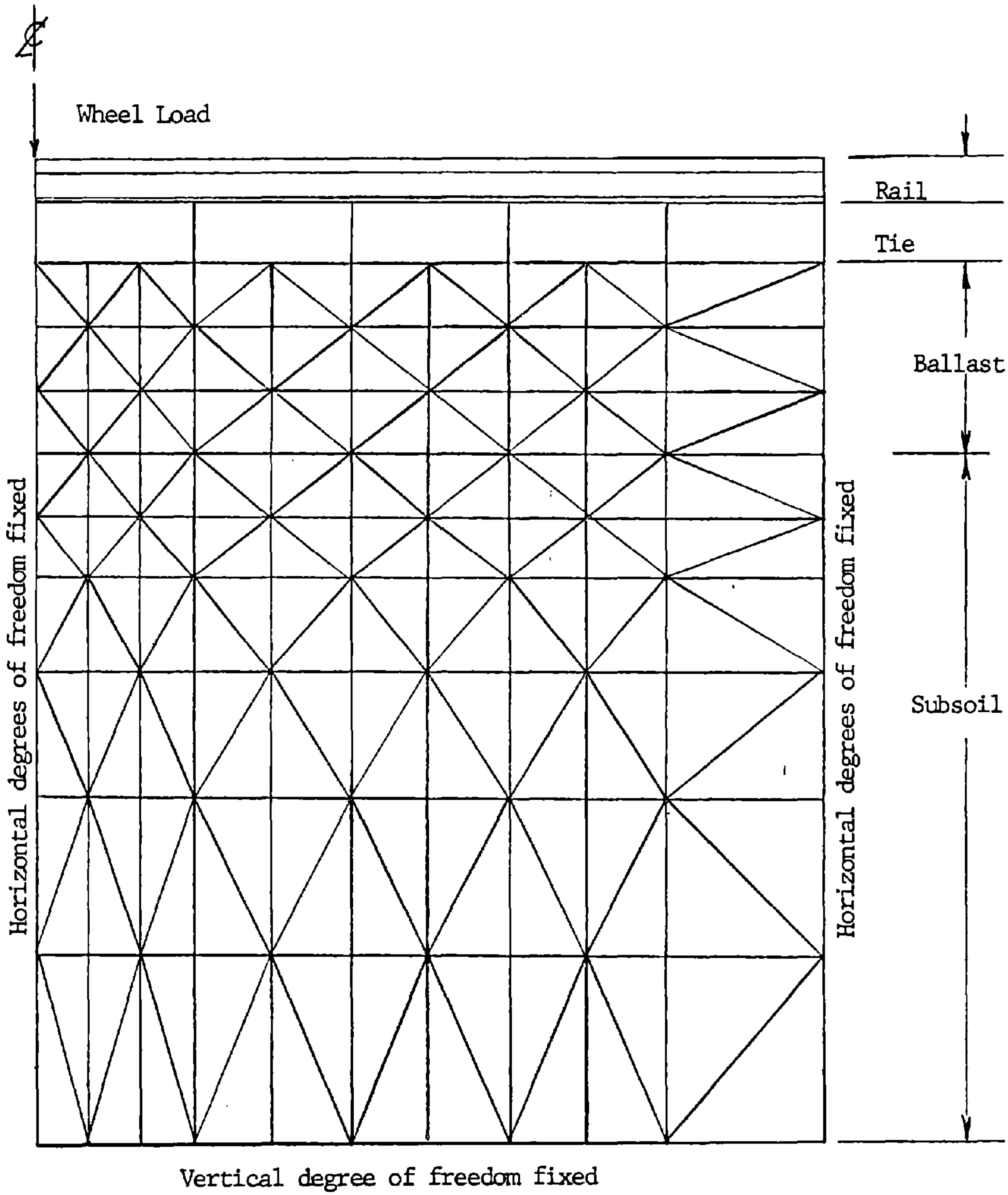

Fig. 2.6- 2. Railroad subgrade--longitudinal simulation.

McAlister's principal accomplishments were the development and validation of the quasi-three-dimensional method and the study of its use with the SSI program to analyze the track system. It should be noted that McAlister's roadbed simlulation for the transverse section utilized only 195 finite elements interconnected by 118 nodes and that the basic finite element used was a consant stress triangle. A considerable improvement in the accuracy of the solution is 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 can also be expected to improve the accuracy of the solution.

The program SSI was next modified in 1975-1976 to include a plane-strain rectangular finite element to be used in place of the plane-strain triangular elements which were the only plate-type elements in the original program. The problems which were run initially by McAlister were rerun using the rectangular elements. These stress distribution solutions were recognized to be less than satisfactory in two major areas. First, it was noted that, because of the limited size of the computer programs, the grid was still too coarse in the areas which contained high stress gradients and the bottom and side boundaries were too close to the point of load. Second, the drastic shortage of test data made it difficult to validate this new method of analysis. Since the Lundgren paper solves the two-dimensional nonlinear problem, the solutions could not be readily compared. The results from the Tayabji research with the ILLI-TRACK program were not available at the time this work was completed.

Grid Refinement Method

In 1976, Chris H. Lawson, a graduate student at the GIT, undertook the further development of the SSI program to study in depth the quasi-three-dimensional procedure and to develop the grid refinement method for examining the stress distribution in areas of high stress concentration. The results from Lawson's study[8] showed that the grid refinement method which had been utilized and developed in the civil and aerospace industries had useful application for the more efficient and accurate analysis of the railroad track problem. This eliminated the first of the less-than-satisfactory areas listed above; and as the program now stands, it is ready to be tuned and validated as soon as test data becomes available. Once this is accomplished, the second less-than-satisfactory area will be eliminated and the program can be distributed as a productive engineering tool for the analysis of the railroad track structural systems.

Future Development Plans

The finite element method of analysis may provide a suitable engineering analysis tool for the rational static stress analysis of layered lime-injected railroad track subsoil systems. A portion of the improvement associated with injection of lime slurry is a result of strength increases 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 surface square foot and including the strength and stiffness properties of the soil-lime interface, it should be possible to develop a rational finite element method of analysis capable of predicting the behavior of lime-injected roadbeds subjected to static loadings. The development of a special finite element for lime-soil interface will be necessary to proceed with this approach. Thus far, this work is in the planning stages only. An extensive labortory testing program will be required in conjunction with the theoretical analysis method 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 of the track system caused by the injection of lime. It could provide the capability for stress analysis and failure prediction for lime injection stabilized track systems.

References

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2. A. N. Talbot, "Fourth Progress Report of the Special Committee to Report on Stresses in Railroad Track," Transactions of the American Society of Civil Engineers, Vol. 88, 1925.
3. James R. Lundgren, "A Simulation Model of Ballast Support and the Modulus of Track Elasticity," University of Illinois, Urbana, Illinois, Master's Thesis, 1970.
4. S. D. Tayabji, "Considerations in the Analysis of Conventional Railway Track Support Systems," University of Illinois at Urbana-Champaign, Urbana, Illinois, Ph.D. Thesis, 1976.
5. B. J. McAlister, "An Investigation of Stresses and Deflection in Train-Track Systems Using the Nonlinear Finite Element Method," University of Arkansas, Fayetteville, Arkansas, Master's Thesis, 1975.
6. Robert L. Sogge and Ralph M. Richard, "Documentation of Computer Programs SSI for Finite Element Stress Analysis," University of Arizona, Tucson, Arizona, 1974.
7. J. M. Duncan and C. Y. Chang, "Nonlinear Analysis of Stress and Strain in Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM 5, September 1970, pp. 1629-1653.
8. Chris H. Lawson, "Finite Element Analysis of Conventional Railway Track Support Systems," a report submitted in partial fulfillment of the

requirements of the degree of Master of Science in Engineering, Graduate Institute of Technology, Little Rock, Arkansas, February, 1977.

3. CASE STUDIES

To assist in evaluating the field performance of lime injection under railroad loading conditions, a case study program was initiated in the spring of 1975. The purpose of the program was to collect quantiative and qualitative information about injection procedures and track roadbed and site conditions before and after lime injection stabilization. The information was acquired from the participating railroads and the lime injection contractors and through field inspection trips and soil exploration and testing. Reports of the track condition before injection were requested from the railroad division engineers. The lime injection contractors were extremely helpful in providing injection information. Visits were made to most sites whenever possible both before and after lime injection and during soil sampling. Soil samples and subsurface information were obtained at selected sites utilizing soil drilling trucks equipped with hy-rail wheels provided by funds from the research contract.

Several case study projects were initiated during the first five quarters of the research program. Seven of the studies progressed beyond the initial phase (see Table 3.0-I), and special reports were prepared for each.

The special case study reports have been filed in the GIT technical library. Short summaries of each case study are presented in Sections 3.1 through 3.7. The remaining studies were considered as preliminary only, and no additional tasks were performed.

TABLE 3.0-I

Fully Developed Case Studies

Railroad	Location	Investigator
Rock Island	Forrest City, Arkansas	Sheaff & Welch
Missouri Pacific	Pine Bluff, Arkansas	Sheaff & Kumar
Burlington Northern	Chillicothe, Missouri	Sheaff & Kumar
Santa Fe	Cresson, Texas	Welch
Chicago & Northwestern	Belle Fouche, S. Dakota	McNutt & Gremillion
Rock Island	Forrest City, Arkansas	McNutt & Lawson & Blacklock
Valley Drive*	Little Rock, Arkansas	Lawson

*The Valley Drive case study was performed on a lime injection city street.

3.1. FORREST CITY--ROCK ISLAND
LIME INJECTION CASE STUDY, PHASE I

The purpose of this study was to identify possible problem soil strata that were responsible for the "continual subsiding of the subgrade" reported by the Rock Island Railroad and to determine the potential effectiveness of lime slurry pressure injection (LSPI) as a means of stabilizing the embankment soils.

The main east-west line of the Rock Island between Memphis, Tenn., and Little Rock, Ark. 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 in most of the 10-mile-long problem areas. Typical daily rail traffic consists of over 800 cars weighing nearly 50,000 tons.

Three areas on the main Rock Island track between Forrest City and West Memphis, Ark., were selected for soil exploration and sampling: one at Round Pond, one at Heth, and one on a county-road crossing between Round Pond and Heth. Each of these areas is on fill and is frequented by slides caused by excess moisture and unstable clay soils.

The soil-testing firm of Barrow-Agee Laboratories, Inc. was directed to obtain full-depth Shelby tube samples at the three test sites. The number of borings was to be determined from the conditions found as sampling proceeded. Barrow-Agee was to prepare a boring log for each hole and package the undisturbed samples for delivery for testing. As shown in the plan of borings (Figure 3.1-1), five borings were made to a depth of 20 feet using a standard highway rubber-tired truck (Figure 3.1-2). Boring was completed on Dec. 20, 1974. A typical boring log is shown in Figure 3.1-3. Occasional layers of

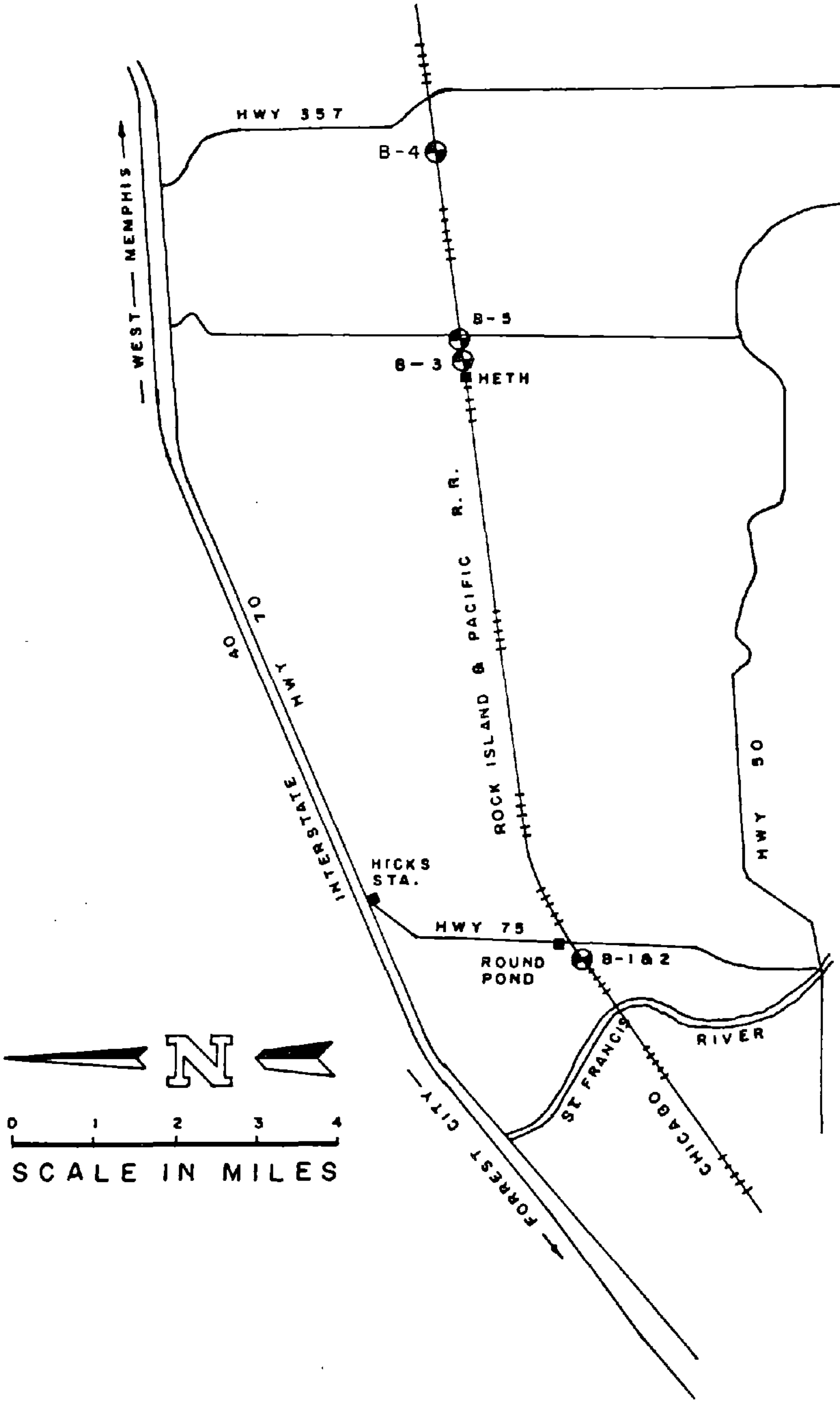


Fig. 3.1-1. Plan of borings.

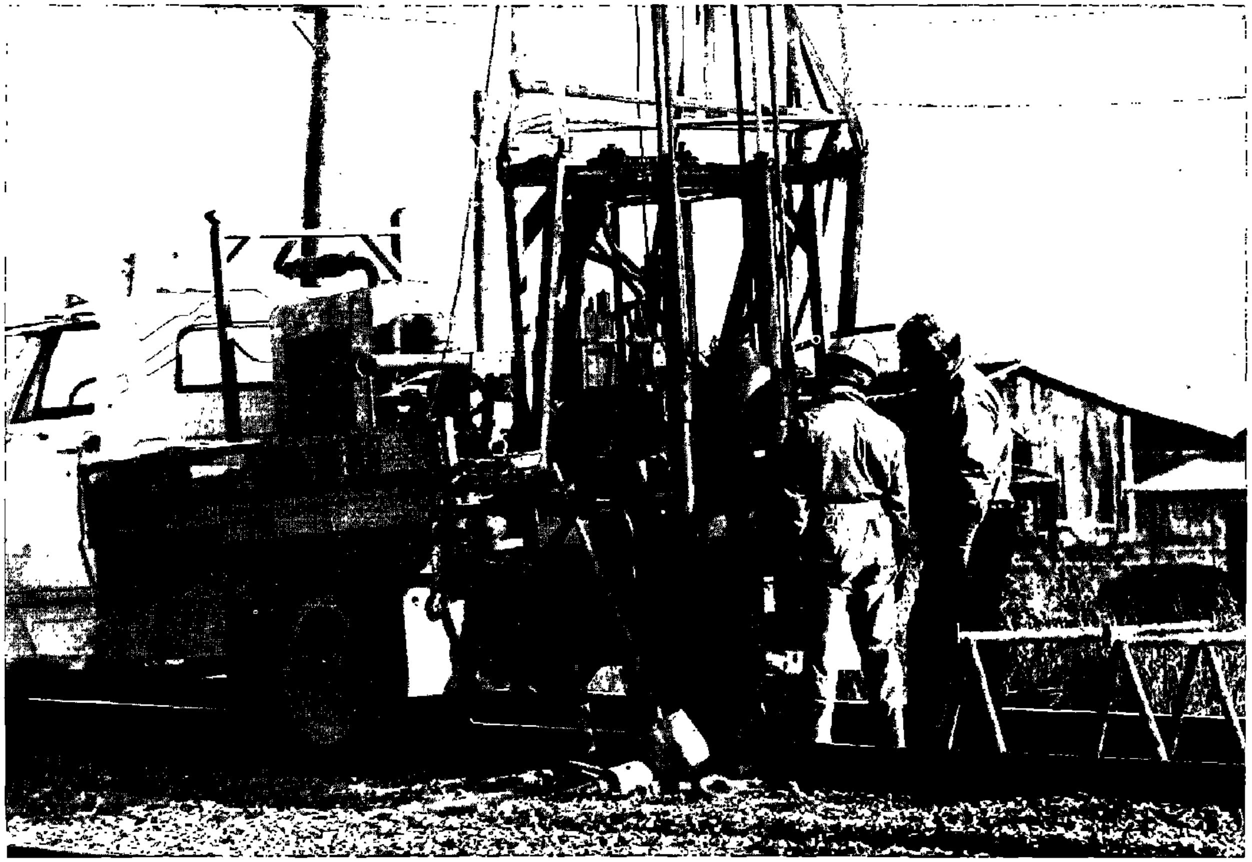
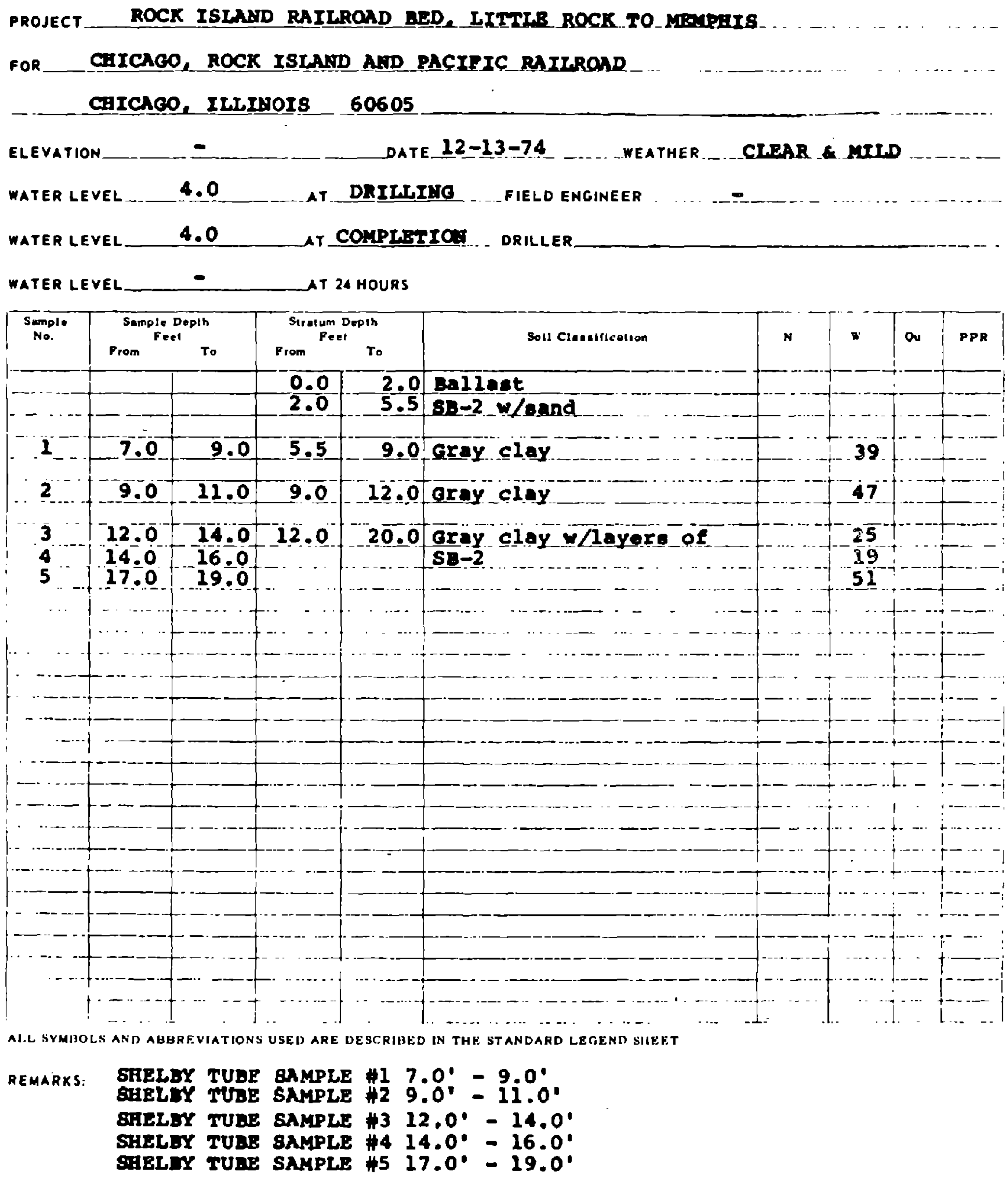


Fig. 3.1-2. Boring with standard highway truck.



PROJECT ROCK ISLAND RAILROAD BED, LITTLE ROCK TO MEMPHIS

FOR CHICAGO, ROCK ISLAND AND PACIFIC RAILROAD

CHICAGO, ILLINOIS 60605

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

WATER LEVEL 4.0 AT DRILLING FIELD ENGINEER -

WATER LEVEL 4.0 AT COMPLETION DRILLER

WATER LEVEL - AT 24 HOURS

Sample No.	Sample Depth Feet From	Sample Depth Feet To	Stratum Depth Feet From	Stratum Depth Feet 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'

Fig. 3.1-3. Typical boring log.

cinder, sand and sandy clay were encountered at several locations.

Stability problems were observed at numerous locations. The track was out of cross-level and alignment, and slide areas where the embankment had failed were apparent. The lengths of the "soft spots" varied from approximately 50 feet to several hundred feet. The site shown in Figure 3.1-4 is an unstable area near the Heth crossing.

A limited laboratory testing program was conducted 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.

To determine the effect of lime on certain physical-chemical properties of the soils, three types of lime reactivity tests were conducted on selected samples. The effect of lime on the plastic and liquid limits (Atterberg limits) was determined on five samples at 0 and 3 percent lime. The amount of shrinkage was measured in bar shrinkage molds of four samples at lime contents of 0, 2, and 6 percent. The minimum lime content necessary to complete the soil reaction was determined by the Eades Quick Test. This test indicates the minimum percentage of lime necessary to result in a pH of 12.4, which is reported by Eades and Grim as essential for lime stabilization to occur. The increase in shear strength of lime-treated soil was examined by performing unconfined compression tests on remolded samples with 0, 2, 4 and 6 percent lime cured for 7 days. The results of the tests are presented in the report.

To aid in classification and identification of the natural soils, plastic and liquid limit tests and mechanical grain size analysis tests were performed on most samples. The results are tabulated in Table 3.1-I.

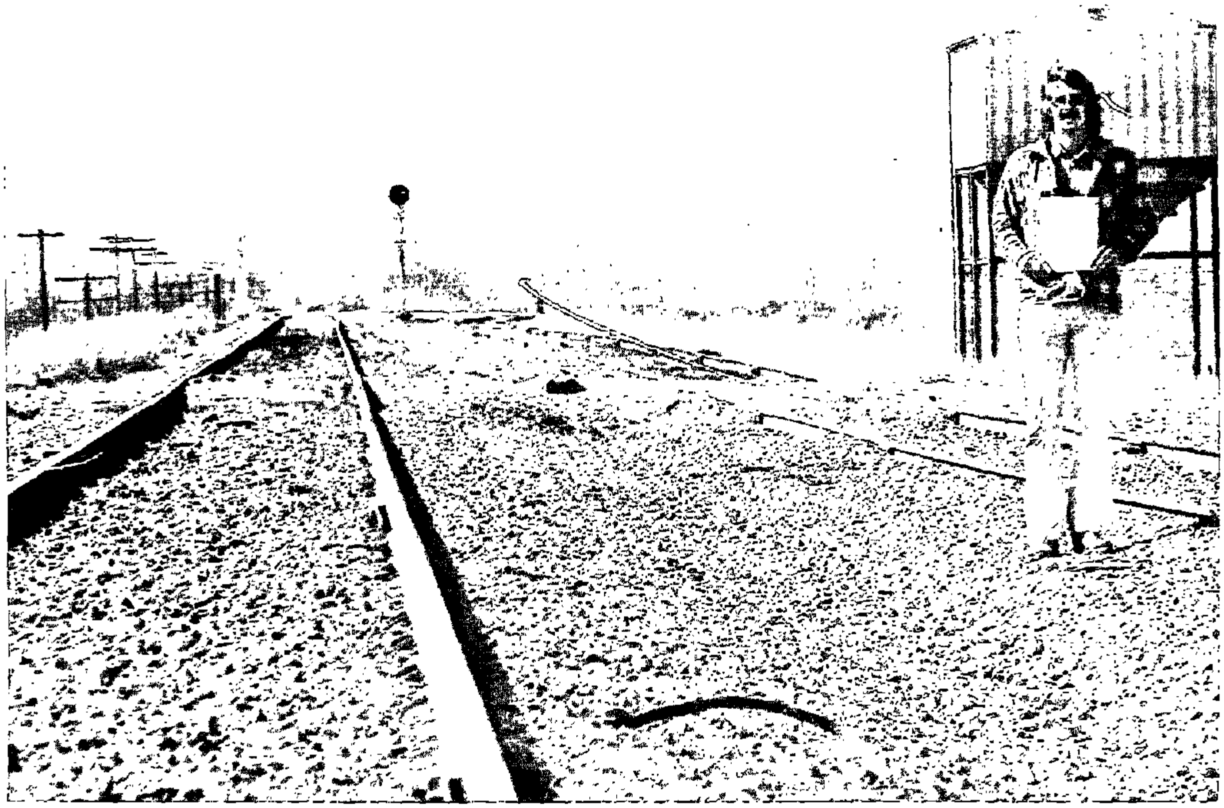


Fig. 3.1-4. Track roadbed at Heth.

TABLE 3.1-I

Chicago, Rock Island and Pacific Railroad

Soil Classification Data Summary

Boring No.	Depth (ft.)	Atterberg Limits		% Passing #200 Sieve	AASHO Class
		P.L.	L.L.		
1	7-9	20	26	--	A-4
1	12-14	27	68	--	A-7-6
2	3-5	--	--	40	A-4
2	5-7	19	26	53	A-4
2	9-10	--	--	40	A-4
3	7-9	39	91	--	A-7-5
3	12-14	30	89	73	A-7-5
4	3-5	19	27	33	A-2-4
4	9-11	25	80	88	A-7-6
4	12-14	19	40	--	A-6
4	15-17	39	105	--	A-7-5
5	4-6	23	37	98	A-6
5	8-10	27	71	--	A-7-6
5	14-16	--	--	71	A-4

Recommendations and Conclusions

The purpose of conducting the laboratory tests discussed in the report was to determine whether the soils in the problem railroad embankment were reactive with hydrated lime. The results of the soil-lime reactivity tests conducted on the cohesive soils indicated that properties such as plasticity and shrinkage were improved. These positive results were interpreted to indicate that the injection of a hydrated lime slurry would improve the behavior of the subgrade soils in this section of the rail line. It had been evident from previous experience that successful lime injection requires a positive reaction of the intermixed lime and soil. Other stability parameters were considered by the engineers on the research project; and based on the information available, it seemed reasonable to expect that lime injection would successfully improve the performance of the track structure system.

Because of the variability in subgrade conditions and length of the problem area (approximately 10 miles of railway), it was recommended that a test section be lime-injected and evaluated. The test section should be situated where there was some knowledge of the subsurface conditions and where track stabilization was a problem. It should also be of a length sufficient to represent the subgrade conditions found over most of the problem area. A test section 4000 feet long located in the general vicinity of the Heth siding would satisfy these criteria. It was estimated that the approximate cost of the test section (including lime and installation) would be $23,000.

A suggested specification for the lime injection work was provided. 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 was encountered, (2) the total quantity of lime injected should be between 85 and 105 lbs per lineal ft, and (3) the injection spacing should not exceed 5 feet between injections. It was further recommended that the LSPI work be observed by a member of the GIT team. The behavior of the injected test section was to be monitored periodically by the research tem. The final recommendation would have then been prepared based on the performance of the test section.

Summary

According to the plan laid out at the initiation of the Forrest City case study, the Rock Island was to follow the recommendations and provide funds and designate a section of track for an LSPI stabilization test to be accomplished in the summer of 1975. The cost of the injection would have been paid by the Rock Island.

After the roadbeds had been stabilized, the success of the project would have been determined based on surface conditions. The university would then have taken additional soil borings and performed engineering tests to complete the cycle of testing and to provide recommendations for the Rock Island. The additional soil exploration and analysis work would have been funded under the FRA program. This would have concluded the planned research activities for the Forrest City-to-West Memphis track on the Rock Island line. Unfortunately the Rock Island filed for bankruptcy, and the case study was halted in April, 1975.

3.2. PINE BLUFF--MISSOURI PACIFIC LIME INJECTION CASE STUDY

During March, 1975, an investigation of the feasibility of using LSPI for the stabilization of problem subsoils on the Missouri Pacific Railroad track near Pine Bluff, Arkansas was initiated. The USDA Soil Classification Association for the area is Morganfield-Keo-Rilla. These soils are described as deep, well-drained, moderately to slowly permeable, level and undulating, acid, loamy bottom land soils.

The problem track was on the Pine Bluff siding, which had been constructed in 1972 and graded at least 12 inches higher than the main line. The performance of the main line (constructed much earlier) was reported as satisfactory, but the siding had required frequent releveling and had been put out of commission on March 25, 1975.

Nine preliminary borings were obtained in March, 1975. Laboratory testing indicated an optimum line content (from index property tests and the Eades-Grim[1] Quick Test) of 3 percent. Subsequently, 12 borings to a depth of 15 feet were obtained near MP 391 in July, 1975. The soil profile is shown in Figure 3.2-1.

A study of LSPI field records indicated that the average amount of lime injected was 0.5 percent (by weight). However, since the lime distributes into seams and pockets, this average value is considered a lower limit for lime reactivity evaluation.

Classification and strength tests were conducted on the natural soil containing 0, 0.5, and 2 percent of lime by weight. Three index properties were used as a measure of ion exchange and the resulting flocculation-agglomeration: (1) amount of 2-micron clay particles, (2) liquid limit, and (3) plasticity index. Unconfined compression strength was used to measure the

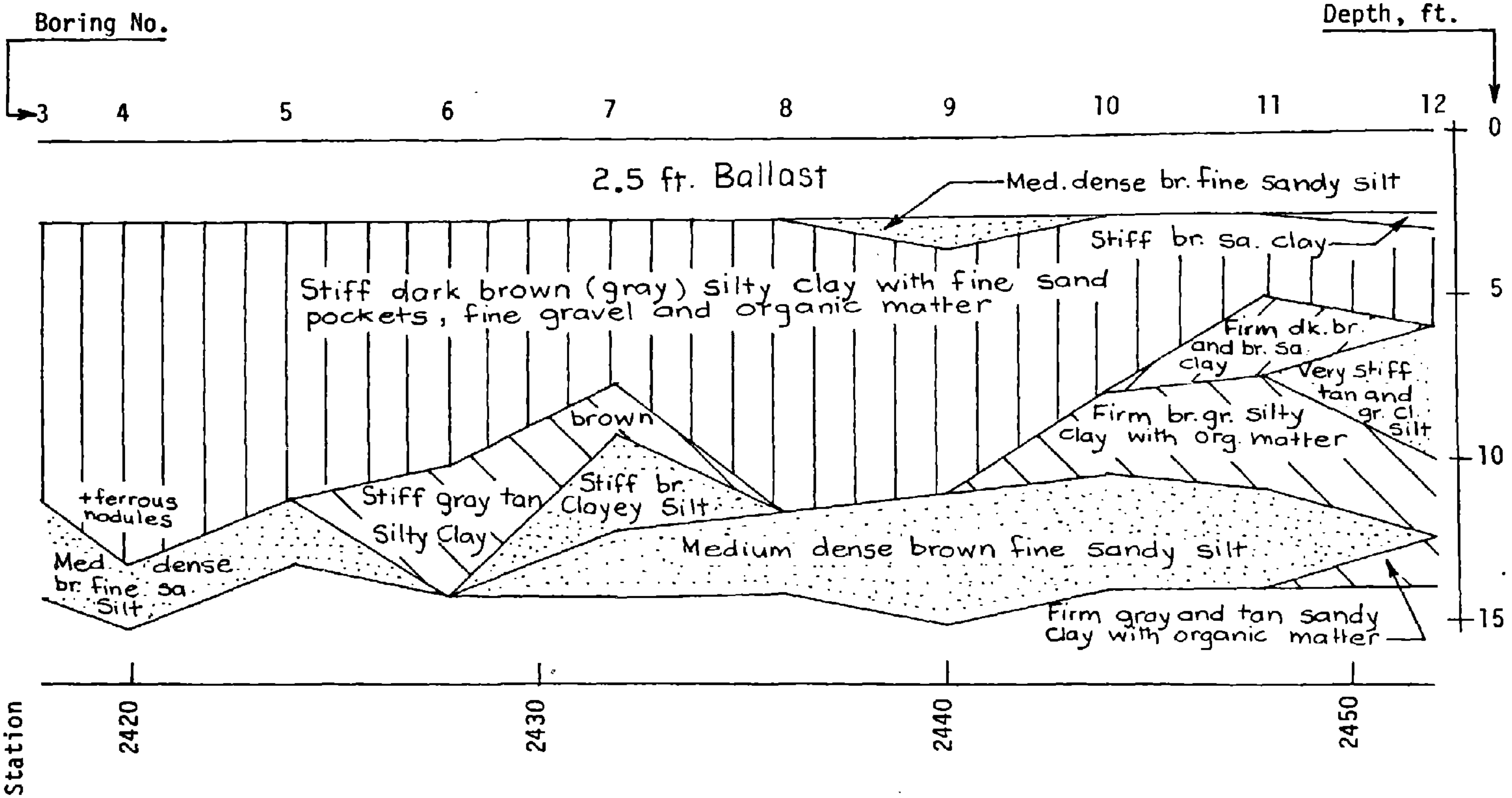


Fig. 3.2-1. Soil profile of the Pine Bluff siding.

cemeting effect of the pozzolanic reaction.

The data revealed that, with an increased lime percentage and curing period, both the index properties and strength properties showed improvement, indicating that the soil is lime reactive. On the basis of the data, LSPI using a lime-water ratio of 2.5 lb/gal to a depth of 10 feet every second tie was recommended.

Reference

[1] Eades, J. L., and Grim, R. L., "A Quick Test to Determine Lime Requirements for Lime Stabilization," Highway Research Record No. 139, Highway Research Board Washington, D. C., 1966, pp. 61-72.

3.3 CHILLICOTHE--BURLINGTON NORTHERN LIME INJECTION CASE STUDY

This case study involved an investigation of the effect of lime slurry pressure injection (LSPI) on a portion of railroad track in the Brookfield-Richmond area on the Burlington-Northern line. The site is in Carroll County, Mo., in the vicinity of Chillicothe. The track had been injected during September and October, 1973. By August, 1975, some areas of weakness had developed even though most of the track was in a fairly good condition.

The soils of the area are highly clayey and slowly permeable. These soils become excessively wet during wet seasons and require drainage. During the summer, these soils tend to become too dry.

Foundation boring were made at 13 locations in three different areas during August, 1975. The portion of track including borings 1 through 5 was underlain primarily by noncohesive material; therefore, no test samples were obtained from those borings. Soil profiles for the other two areas are shown in Figure 3.3-1.

During boring operations, an inspection of the track indicated the following:

1. Mud pumping near borings 2 and 7. Some cribbing and reballasting had been done in the area of boring 7, and the drainage was improved.
2. Track near boring 11 had been reballasted and was in good condition.
3. A portion of the track between borings 7 and 8 was in bad condition.

Due to the site conditions and the nature of boring equipment, it was

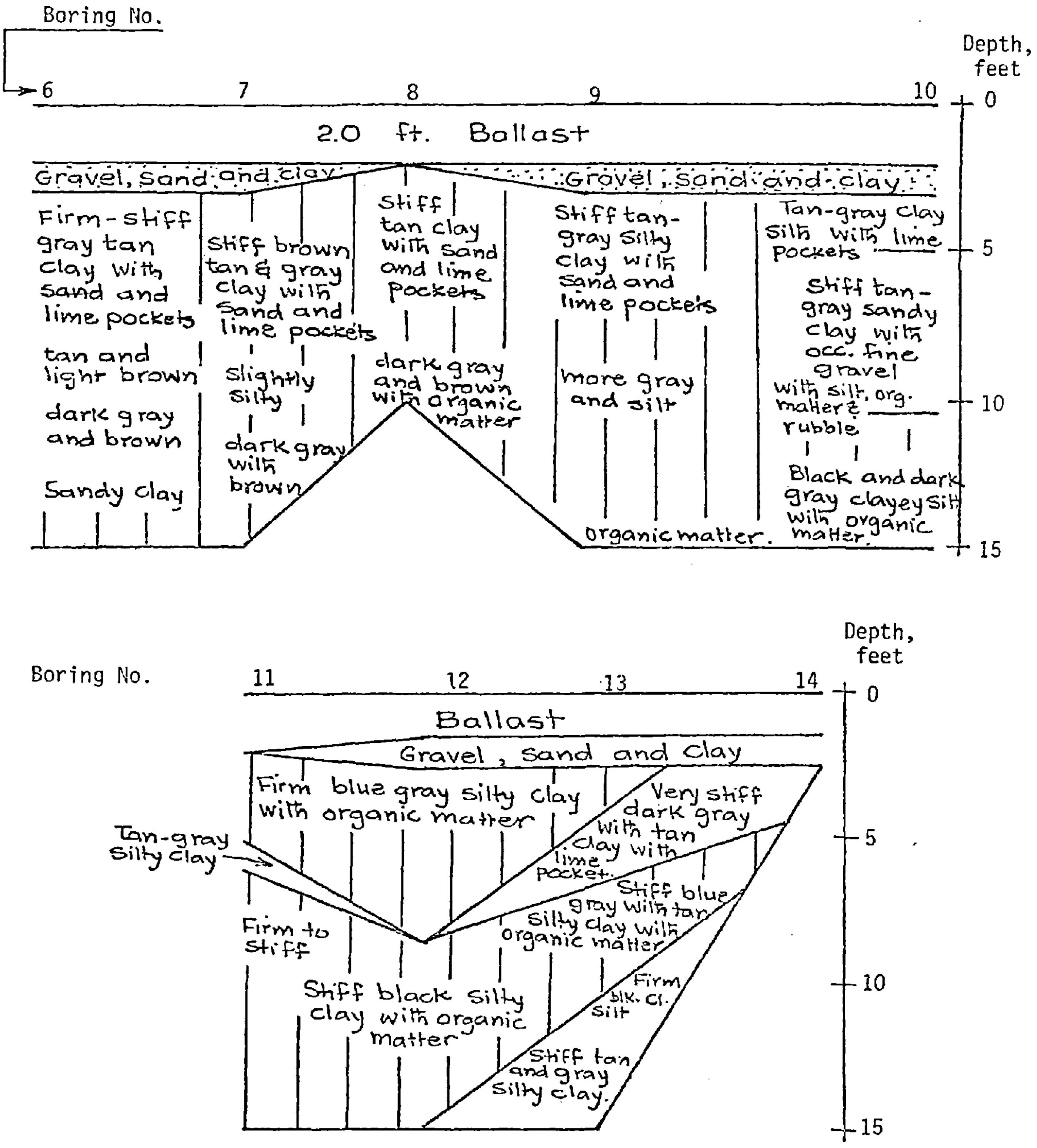


Fig. 3.3-1. Soil profiles.

not possible to obtain samples from the areas on the sides of the railroad track. Because of this, the samples obtained from borings 6 through 14 were divided into three catagories:

1. Treated--those from the expected lime injection zone.
2. Possibly treated--those from the depths just below the expected lime injection zone. The exact nature of the sample treatment was to be determined from the differences, if any, in the actual test data.
3. Untreated--those from a depth well below the zone of expected lime injection treatment.

The following testing program was adopted:

1. Determine the natural moisture content and in-place density.
2. Perform consolidation testing and determine the preconsolidation stress. The stress range was from 0.25 tsf to 16.00 tsf.
3. Perform unconfined compressive strength testing at the slowest possible speed (0.05 in/min).
4. Determine particle size distribution using the AASHTO designation T-88-57.
5. Determine liquid limit and plastic limit in accordance with the AASHTO designations T 89-68 and T 80-70, respectively.

The results of consolidation and unconfined compression tests are shown in Table 3.3-I; and those of particle size distribution, liquid limit, and plastic limit are shown in Table 3.3-II.

Almost all the samples showed discontinuities and heterogeneity. The heterogeneities in the treated samples were due to the presence of lime or its reaction products. In certain cases, the samples contained lenses of coarse-grained material in the otherwise clayey sample. In some cases, slickensides,

TABLE 3.3-I
Results of Consolidation and Unconfined Compression Tests

Boring Number	Sample Depth[a] (ft)	Precons[b] Stress (tsf)	Maximum Stress (psi)	Yield[c] Stress (psi)	Elastic Modulus (psi)
6	3.5	--	23.9	15.3	1285
	6.0	1.0	20.3	14.8	1178
	11.0	1.3	30.8	23.8	1194
7	3.5	--	20.9	15.7	1191
	6.0	1.8	14.1	13.1	990
	14.0	2.4	26.1	19.6	1377
8	2.5	--	37.4	21.3	1681
	6.5	--	34.5	25.0	1631
9	3.5	--	30.6	16.8	1910
	5.5	--	28.8	20.0	1082
	9.0	1.2	29.8	20.0	1677
	14.0	7.8	--	--	--
	14.5	2.1	34.2	21.5	1867
10	3.5	--	--	--	--
11	4.0	--	21.3	16.3	931
12	5.5	1.8	--	--	--
	14.0	2.8	22.9	20.0	837
13	6.0	2.7	--	--	--

[a] Depth to the top of a 6-in. sample.

[b] Preconsolidation stress using Casagrande method.

[c] Stress at the end of the initial straight line portion of the stress-strain curve.

TABLE 3.3-II
Index Property Data of Soil Samples

Boring Number	Depth[a] (ft)	%[b] Sand	%[c] Silt	%[d] Clay	Liquid Limit	Plastic Limit	AASHTO[e] GI
6	3.0	6	48	46	62	35	38
	5.5	2	48	50	68	42	48
	7.5	5	53	42	53	27	29
	11.0	12	59	29	42	23	21
7	9.0	1	51	48	62	21	29
	14.0	6	63	31	43	12	14
8	3.0	8	58	34	43	21	21
	9.0	2	59	39	52	31	34
9	4.0	12	64	24	42	18	17
	6.0	3	60	37	51	31	33
	9.5	0	62	38	54	15	22
	14.5	6	61	33	51	32	32
11	4.0	5	55	40	50	18	21
12	3.5	3	58	39	50	14	19
	14.0	6	58	36	52	21	24
13	3.0	10	77	13	45	14	15
	5.5	10	60	30	40	11	12

[a] Depth to the top of a 6-in. sample.

[b] Particle size greater than 75 microns.

[c] Particle size between 2 and 75 microns.

[d] Particle size smaller than 2 microns.

[e] American Association of State Highway and Transportation Official's Classification Group Index.

planes of weakness, and inclusions of alien material seemed to be present. The effects of discontinuities and heterogeneity on individual tests were not always apparent. In most cases, however, the initiation of crack formation in the unconfined compression test could easily be attributed to the presence of lenses, inclusions, cracks, and small particles.

The consolidation tests were run primarily to determine if the pressure induced by lime injection would cause some preconsolidation of the soil material. The preconsolidation stress was determined using the Casagrande method. The data from the unconfined compression tests were sued to determine yield stress and the elastic modulus of the material.

From the boring information, the effect of lime injection was found to be absent below 7.5 ft. The differences in the data obtained from the samples above and below this depth, however, were not significant.

Conclusion

The only conclusion that could be drawn from this test program was that, two years after the treatment, the tests did not show any significant differences between the samples of the treated zone and those of the lower untreated zone. In light of the fact that some railroad maintenance agencies have found LSPI to be an effective and worthwhile method, it is possible that the conventional soil testing program--the type of which was used in this case--is not suitable for bringing out the potential of the LSPI method.

3.4. CRESSON--SANTA FE
LIME INJECTION CASE STUDY

In 1973, after experiencing unstable track between Cresson and Cleburne, Texas, due to a deteriorating subgrade, the Atchison, Topeka and Santa Fe Railroad adopted a test program to determine if lime injection would stabilize the problem subgrade. Fourteen locations, totalling 13,743 feet of track, were lime injected during July and August, 1973. During July, 1975, permission was obtained from the Santa Fe for research project personnel to make soil borings and obtain samples from lime-treated areas and adjacent untreated areas. These borings were made in September, 1975.

The objectives of this study were to obtain samples from lime-treated areas and adjacent untreated areas, perform tests to measure the engineering properties, and determine if there is a significant difference in properties between the treated and untreated areas. The comparison included: (1) moisture content, (2) dry density, (3) Atterberg limits, (4) gradation, (5) undrained shear strength, and (6) pH. The results of the laboratory testing program show:

(1) No discernable moisture difference exists between treated and untreated areas.

(2) Six of 33 samples from treated areas had reduced plasticity due to lime treatment.

(3) No change in gradation due to lime treatment was observed.

(4) The pH of several samples from treated areas was 1 to 1.5 units higher than the pH of samples from untreated areas.

(5) The resuts of the shear strength tests showed some scatter, with no apparent difference between treated and untreated areas.

The conclusions reached in this study are:

(1) The soil properties in the treated areas were not modified to any great extent due to lime injection. Some samples showed an increase in pH, but less than 20 percent of the samples showed a reduction in plasticity. No other changes were apparent.

(2) The injected lime was not well-distributed throughout the soil mass. This conclusion is supported by the lack of observed lime seams in the samples from treated areas and by the small number of samples exhibiting changes in properties due to lime modification.

(3) The study method used (i.e., testing of small samples from the injected areas) may not be the appropriate method for determining whether any overall improvement in roadbed performance has been achieved. The presence of lime seams, although not distributed throughout the soil mass, may provide moisture barriers or increased stability of the total roadbed structure.

As a result of the insight gained from this case study, it is recommended that future studies of the possible improvement of lime-injected embankments should be done on a full-scale performance basis. Measurement of long-term deformations due to moisture change and consolidation and short-term, perhaps dynamic, load-deformation properties should be made for injected sections and control sections.

3.5. BELLE FOURCHE--BURLINGTON NORTHERN LIME INJECTION CASE STUDY

Introduction

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 responsible for "sink holes" and difficulty in maintaining the cross-level of the track on the Bentonite spur. The spur was originally constructed as a "temporary" line, to serve the new Bentonite plants, with a life expectancy of about 20 years. After more than 25 years of continuous use, the roadbed had deteriorated to the condition shown in Figure 3.5-1. Because of the continued demand for use of the spur, it was necessary to repair and upgrade the roadbed without stopping traffic.

In June 1974, approximately 12,000 feet of unstable track on the Bentonite spur was injected with lime slurry. The average depth of injection was approximately 10 feet as measured below the top of the rail. The treated areas varied from 100 feet to 1350 feet in length.

In the summer of 1975, a staff member visited 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 appeared that the stabilization had improved the roadbed. In the summer of 1975, the track section injected in 1974 was chosen as a case study site. In August 1975, soil samples were obtained at three injected and three non-injected locations in the treated area. After the test program had begun, it became apparent that these samples were from three different soil formations, thus making any conclusive lime injection evaluations impossible. The decision was then made to revisit the most

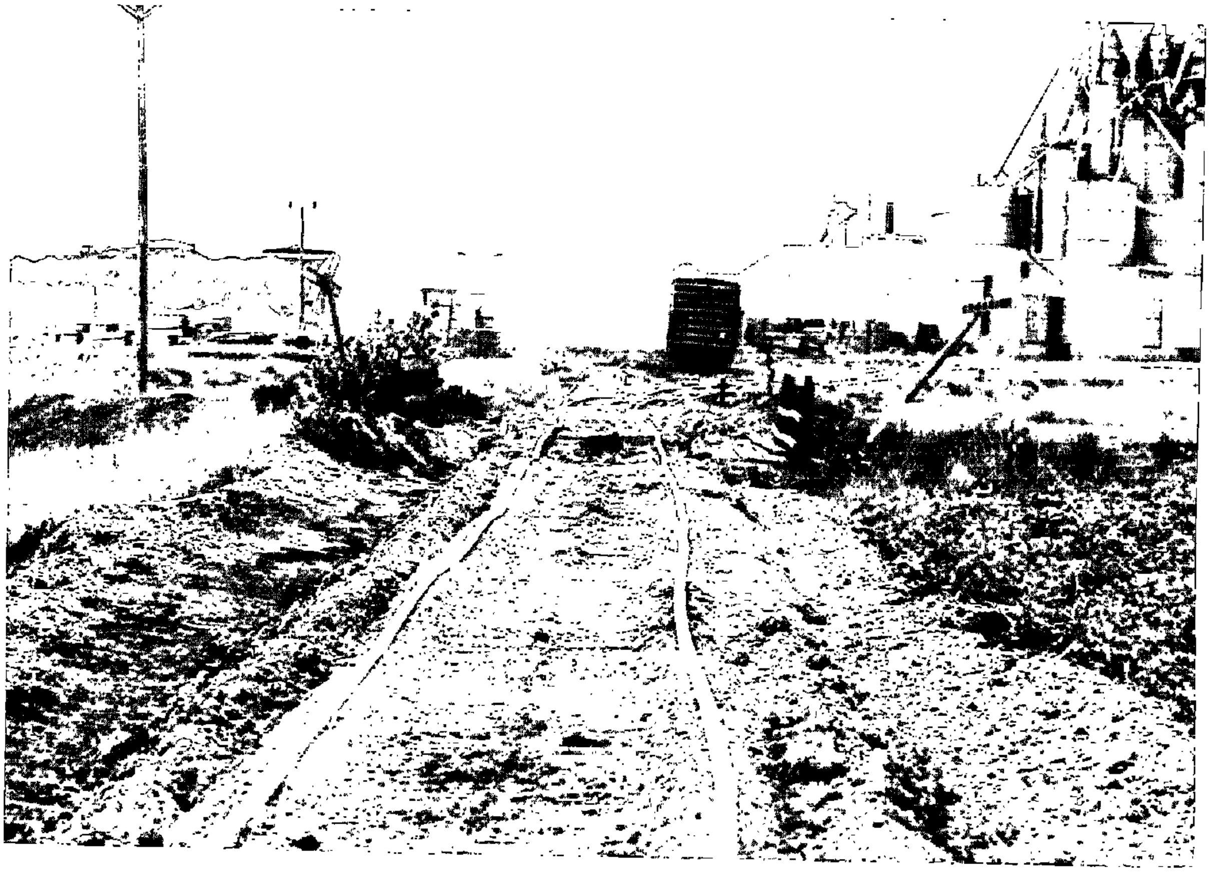


Fig. 3.5-1. Bentonite spur roadbed shortly after injection.

promising of these three areas based on the findings of the August samples and make additional borings and take additional samples. This was accomplished in November 1975 under the direction of Mr. William T. McNutt, subcontract research engineer.

Forty undisturbed soil samples were obtained from ten borings in a treated area of soil and ten borings in an adjacent parallel untreated area 40 feet away from the track. Two 30-inch Shelby tubes were pushed in each boring, one starting at the surface and one bottoming out at 7 feet below the surface. The land was flat in this area of the track, and the samples in the treated and untreated area were carefully obtained at the same elevations. A sketch of the site, boring locations, and cross-section is shown in Figure 3.5-2. (Note: Lime seams were observed in several of the upper-level samples.)

These forty new samples were utilized in an extensive case history study program. Each soil sample was to be subjected to soils laboratory testing and some to mineralo-chemical and X-ray diffraction analysis.

This case study was conducted in two principal parts by two investigators, one an engineer and one a chemist; therefore, it is summarized in two parts. Part I, written by Mr. W. T. McNutt, includes the results of the statistical soils engineering portion of the experiment; Part II, written by Dr. A. F. Gremillion, includes the results of the mineralo-chemical and X-ray diffraction analyses.

Part I

Soils Laboratory Experiment

The experiment for this previously lime-injected subgrade on the Chicago and Northwestern Railroad's line from Belle Fourche to Colony was designed to (1) objectively evaluate the functional effectiveness of LSPI, (2) develop

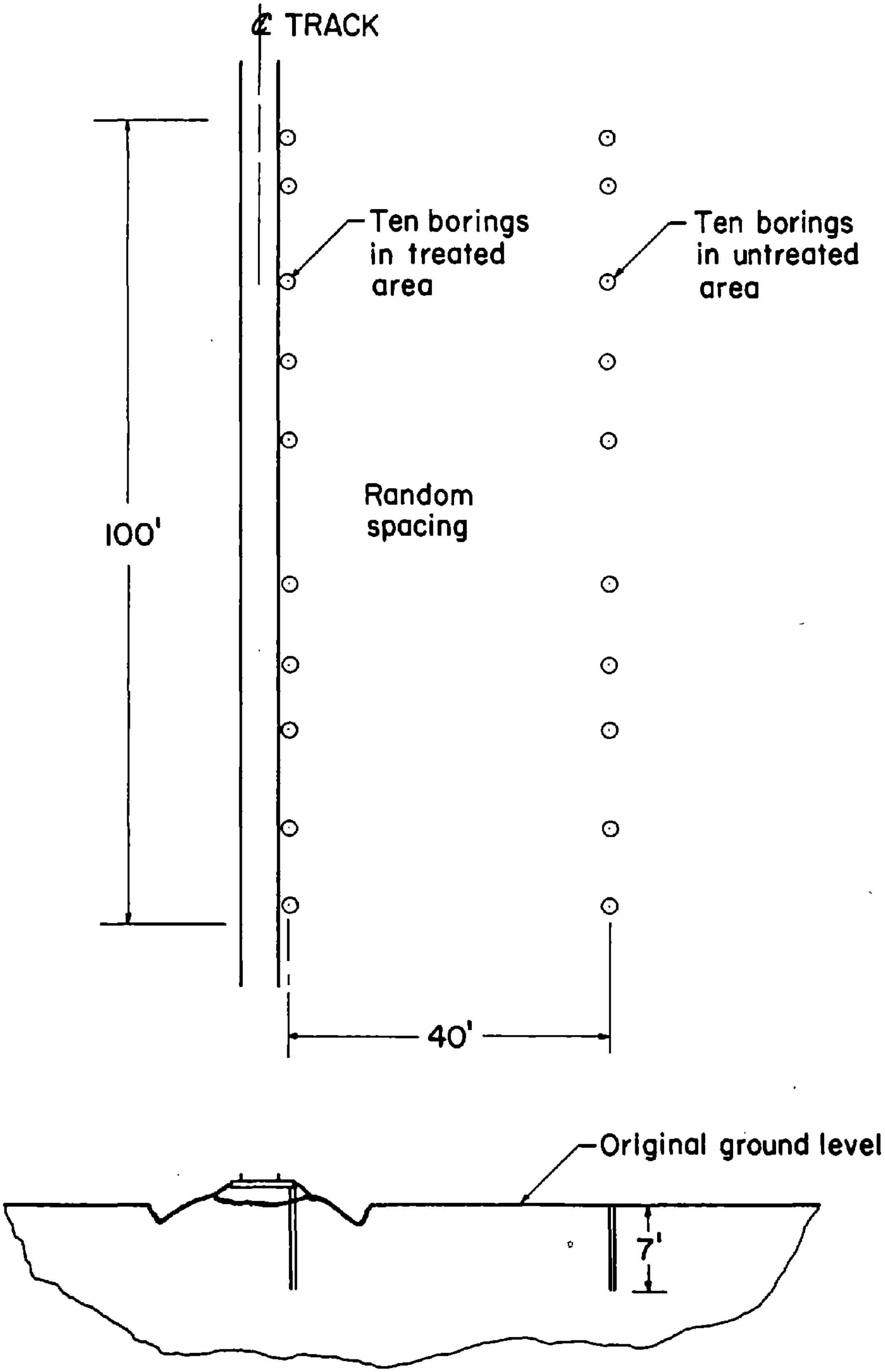


Fig. 3.5-2. Belle Fourche soil sampling plan.

tests for lime slurry pressure injection of railroad subgrades comprising expansive clays, and (3) superficially explore areas of promise for further research.

The test site was selected after a field inspection of the entire length of roadbed from Belle Fourche to Colony. A Chicago and Northwestern Railroad field crew staked the boring locations and ran the topographical survey. Francis-Meador-Gellhaus, Inc., drilled the site, recovered the Shelby tube samples, extruded the Shelby tubes, and logged and sealed the samples and shipped them to GIT.

Observations made during the drilling operation disclosed an extensive, though not complete, thin coating of hardened lime slurry at the ballast-subgrade interface. A few of the injection probe holes were located, and in no instance was the subgrade surface coating continuous between the probe holes.

When the top Shelby tube of boring No. 19 was pulled, the lower 3 1/2 inches were missing, and lime seam was partially covering the exposed end. The missing section was retrieved from the boring by hand. Apparently, the torqueing of the tube to shear the sample had sheared the sample properly at the bottom of the Shelby tube but had also sheared the sample at the lime seam, allowing this piece to drop from the tube during withdrawal.

It was noted that there were some pockets of fine white quartz in many of the samples that possibly could be misconstrued as lime seams. Small amounts of any material that visually might be lime were treated with HCl and were logged as a lime seam if they evidenced frothing.

There were a few lime seams logged from the top Shelby tubes, but none was logged from any of the bottom Shelby tubes. No significance can be given to this as the number of Shelby tube samples is too small for a non-parametric test.

The laboratory testing program was as shown on the flow diagram (Figure 3.5-3). All of the equipment required for the test was recalibrated, and a set of expendable samples was run through the complete testing program to refine laboratory techniques prior to testing of the experiment samples. The order of testing of the experiment samples was random.

All statistical inferences in this experiment were tested at the 95 percent significance level. A single failure criterion was used: differential deformation. Whatever the causative mechanism or mechanism--as evidenced by slope slides, pumping, track squeeze, etc.--the subgrade has failed when the differential deformation exceeds a set boundary. The differential deformation boundary used in this study was 0.75 inch. This one failure criterion need only be made time dependent to validate the full range of probabilistic design techniques.

Most, if not all, subgrades on expansive clays will transgress the differential deformation boundary in time without traffic due to long-range climatic variations. Portions of all such subgrades will transgress this boundary without traffic under seasonal climatic variations.

Conclusions From Laboratory Soil Tests

The differential movement of the lime slurry pressure injected subgrade was significantly less than the untreated control. The Dyadic specific gravity tests suggested that with substantiation they would be rewarding as an acceptability test for lime slurry pressure injection and as a research tool.

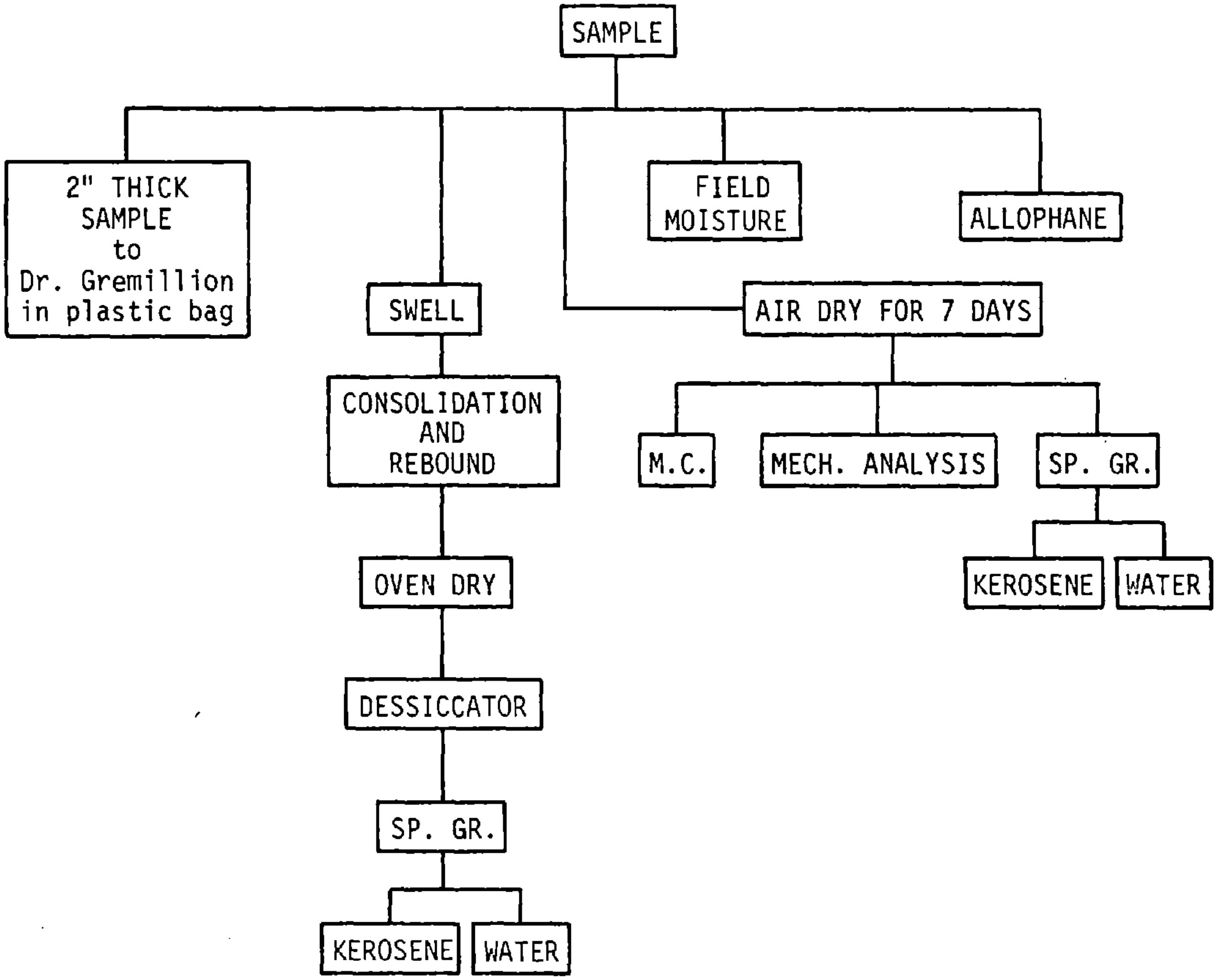


Fig. 3.5-3. Soil testing for Belle Fourche, South Dakota.

Part II

Chemical and X-Ray Diffraction Activities

Four soil samples from the Belle Fourche were studied by chemical and X-ray diffraction methods. The conventional methods employed to study the samples have been described in substantial detail by Jackson in an early publication[1] upon which he has more recently elaborated.[2] The purpose of the soil treatments employed is to clean up a soil sample to separate the clay portion of the soil from the other soil constituents in a manner that minimizes denaturation of the final products. Following the chemical clean-up procedures, the sand, coarse silt, medium silt, fine silt, coarse clay, medium clay, and fine clay fractions are separated from each other by use of a centrifuge and certain suspending solutions. The particle size range for each of these fractions is indicated with some of the data below. At this point, any of these fractions can be subjected to the proper procedures for X-ray diffraction identification of crystalline mineral phases present. The procedures for the preparation of clay fraction samples for X-ray irradiation are substantially different from those employed for the silt and sand fractions.

In this type of work, it is only in the case of a fine clay fraction that it is necessary to remove the suspended matter from its suspension by flocculation with an electrolyte, after that fraction has been separated from other fractions of the soil sample. Apparently, in some cases, the retention of flocculating electrolyte by the fine clay fraction can be so great as to obviate the procedure indicated above as a tool for determination of fine clay content. This appears to have happened with two soil samples (see 3 and 4 in Table 3.5-I).

Sand, Silt, and Clay Content of Soil Samples

The results from the analysis of soil samples for sand, silt, and clay by the procedure referred to above are given in Table 3.5-I.

TABLE 3.5-I

Particle Size Distribution in the Sand-Silt-Clay Portion of Soil Samples Shown, After Soil Clean-Up

Soil Fraction	Percentage of Total Soil and Sample Number			
	1	2	3	4
Sand (+50)	5	3	17	3
Coarse Silt (-50 +20)	8	12	6	7
Medium Silt (-20 +5)	15	18	13	27
Fine Silt (-5 to +2)	8	6	6	1
Coarse Clay (-2 to +0.2)	18	17	16	17
Medium Clay (-0.2 to +0.08)	21	18	15	23
Fine Clay* (-0.08)	15	21	--	--
% Recovery	90	95	--	--

*Percentage of fine clay for samples I6T3 and C10T5 unavailable by methods employed because of very high retention of NaCl from flocculation. Probable values here are about 15 to 20 percent.

X-Ray Diffraction

The X-ray diffraction by various fractions from the soil samples listed above was performed in accordance with the procedures given by Jackson[1,2] using copper K_α radiation and a Philips Geiger counter diffractometer with a scan rate of 1°/min.

Both magnesium saturation-glycerol solvation and potassium saturation-glycerol solvation methods were used with samples mounted on microscope slides. Certain heat-treated samples were also submitted to X-ray diffraction.

The fine clay fractions of soil samples 1 and 2 proved to be principally montmorillonite. Similar fractions for samples 3 and 4 were amorphous. The medium clay fractions from all four soil samples were largely montmorillonite, but quartz was also a prominent component. All patterns from the four coarse clay fractions showed these fractions to be overwhelmingly quartz.

Conclusions From Chemical and X-Ray Diffraction Activities

The principal conclusion that can be drawn from the data of the X-ray diffraction patterns is that, except for the fine clay fractions of soil samples 3 and 4, the four samples studied were very similar. Of equal notworthiness is the fact that, of all the X-ray patterns recorded, only those for the fine clay fractions of samples 3 and 4 indicated only amorphous material. This could explain the failure to recover the fine clay material of either sample 3 or 4 in the quantitative determination of these fractions.

Conclusions

The Belle Fourche Case Study was the first case study conducted which included the combination of a statistically designed soils engineering experiment and a mineralo-chemical and X-ray diffraction research study. This case study was much larger than those previously conducted; and in reflection, the results obtained have shown the value of this approach over the previous approaches used on other case studies conducted by the GIT.

Unfortunately, the time required to conduct a case study which includes the above two areas plus the other necessary areas for the completion of a comprehensive experiment is almost too large to be acceptable in the present "fast" climate. The Talbot Committee spent in excess of 10 years to complete its study.

On the positive side, it should be noted that the approach used for this case study has proved effective, as exhibited by the positive findings of McNutt in his statistical analyses work noted in Part I. The work accomplished by Gremillion, as noted in Part II, is considered to be positive in its procedure; however, the limited time and funds expended precluded any large or significant contribution to the actual case study findings. It is felt, however, that if a similar approach were to be utilized fully on a lime injection case study project, the usefulness of the work as done in Part II of this study would add significantly to the findings.

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2. Jackson, M. L., "Soil Clay Mineralogical Analysis," in Soil Clay Minerology: A Symposium, eds. C. I. Richard and G. W. Kunze (University of North Carolina Press, Chapel Hill, 1964) pp. 245-257.

3.6 FORREST CITY--ROCK ISLAND
LIME INJECTION CASE STUDY, PHASE II

The Rock Island Phase II case history study was an extension of the original contract beyond the completion of all other activities. It was designed to take advantage of the opportunities afforded to the railroad research team by the near-by lime injection stabilization of 10 miles of Rock Island mainline track. Permission was given by Rock Island to conduct a study of the lime injection stabilization effort on their Briark-Brinkley track located in eastern Arkansas.

Funds were requested and approved in early 1976. The field portion of the case study was begun in June, 1976, and concluded in November, 1976. The laboratory portion of the program was begun in June, 1976, and terminated due to lack of funds in August, 1976, prior to the actual testing of any of the preinjection or postinjection samples from the two test sites injected. The subject report does not contain any conclusions or results from the experiments. However, the report does contain approximately 200 pages of raw data, and this information should prove useful to those who might attempt to carry out a similar experiment in the future.

3.7. VALLEY DRIVE

LIME INJECTION CASE STUDY

In July, 1973, Valley Drive, a Little Rock, Ark., subdivision access road, was treated with LSPI. Similar LSPI treatment of foundation soil at an adjacent apartment complex, Huntington Place, had shown signifcant improvement of the consolidation parameters. This prompted LSPI treatment of Valley Drive to a depth of 7 feet to stabilize the upper silty-clay (CL) layer.

Valley Drive is on a fairly flat alluvial plain with good vegetation cover. The road exists mostly as cut to a maximum depth of 3 feet. The road surface was in essentially good condition, except for one ponded area. The road in this section was not trafficable by conventional vehicles.

A total of 18 soil borings were made over a distance of 1893 feet with 5 borings taken 50 feet from the road center line, these providing control samples. These samples were examined in the field and subsequently transported to the GIT Soils Laboratory. The drilling was performed by Grubbs Consulting Engineers, a division of McClelland Engineers, during June, 1975, about 22 months after completion of the LSPI treatment. Figure 3.7-1 shows the schematic layout of the boreholes. The soil profile is shown in Figures 3.7-2a and b. There are essentially three soil layers, as shown by these figures and the boring logs.

The top layer (Soil A) has a liquid limit (LL) of about 40 and a plasticity index (PI) of 20, giving a unified classification of CL. The middle layer (Soil B) had an LL of 80 and a PI of 40, giving it an MH or MH-CH classification. In the lower layer (Soil C), the LL is 96 and this PI 59, giving it a CH classification.

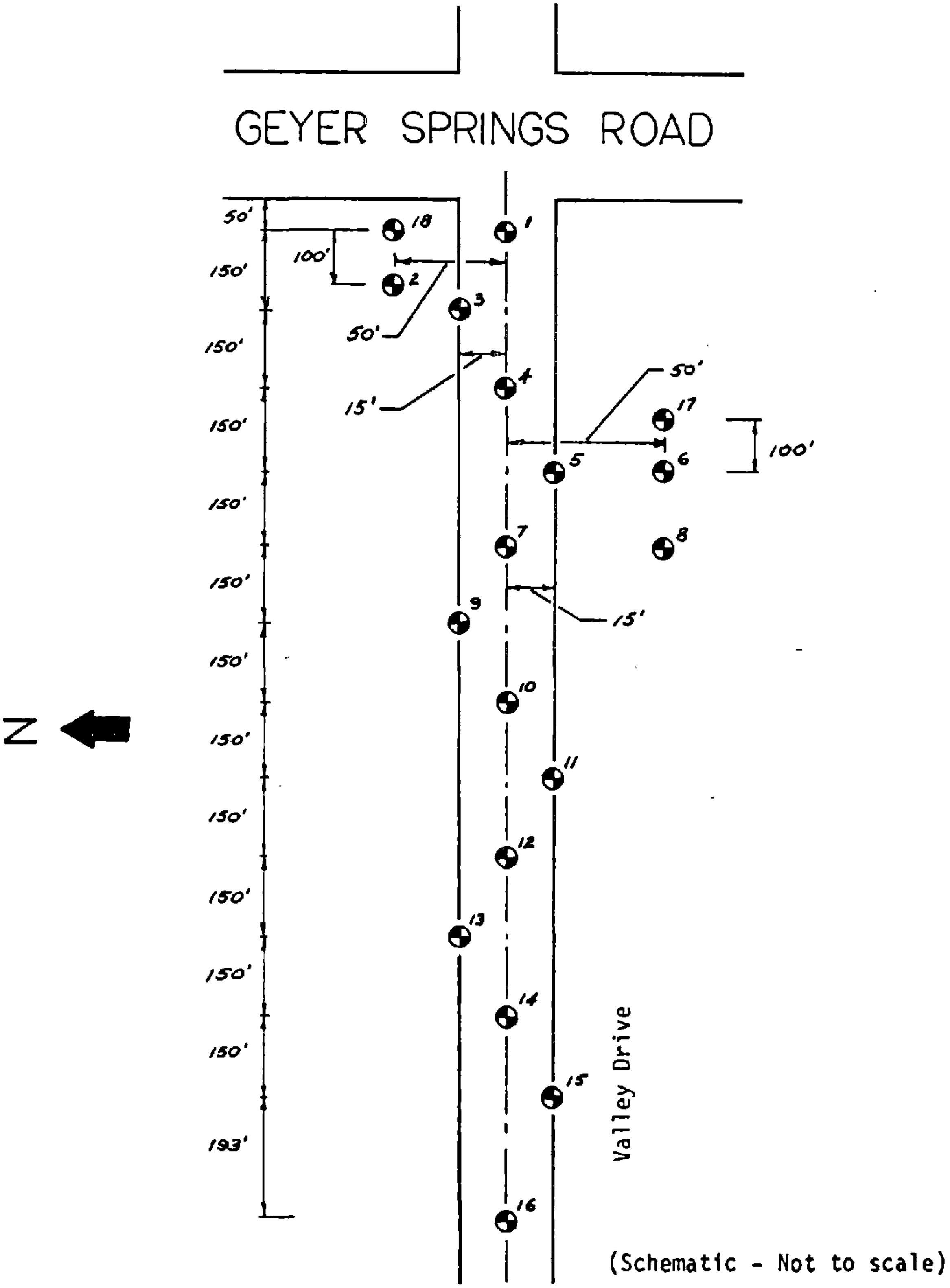


Fig. 3.7-1. Boring location plan.

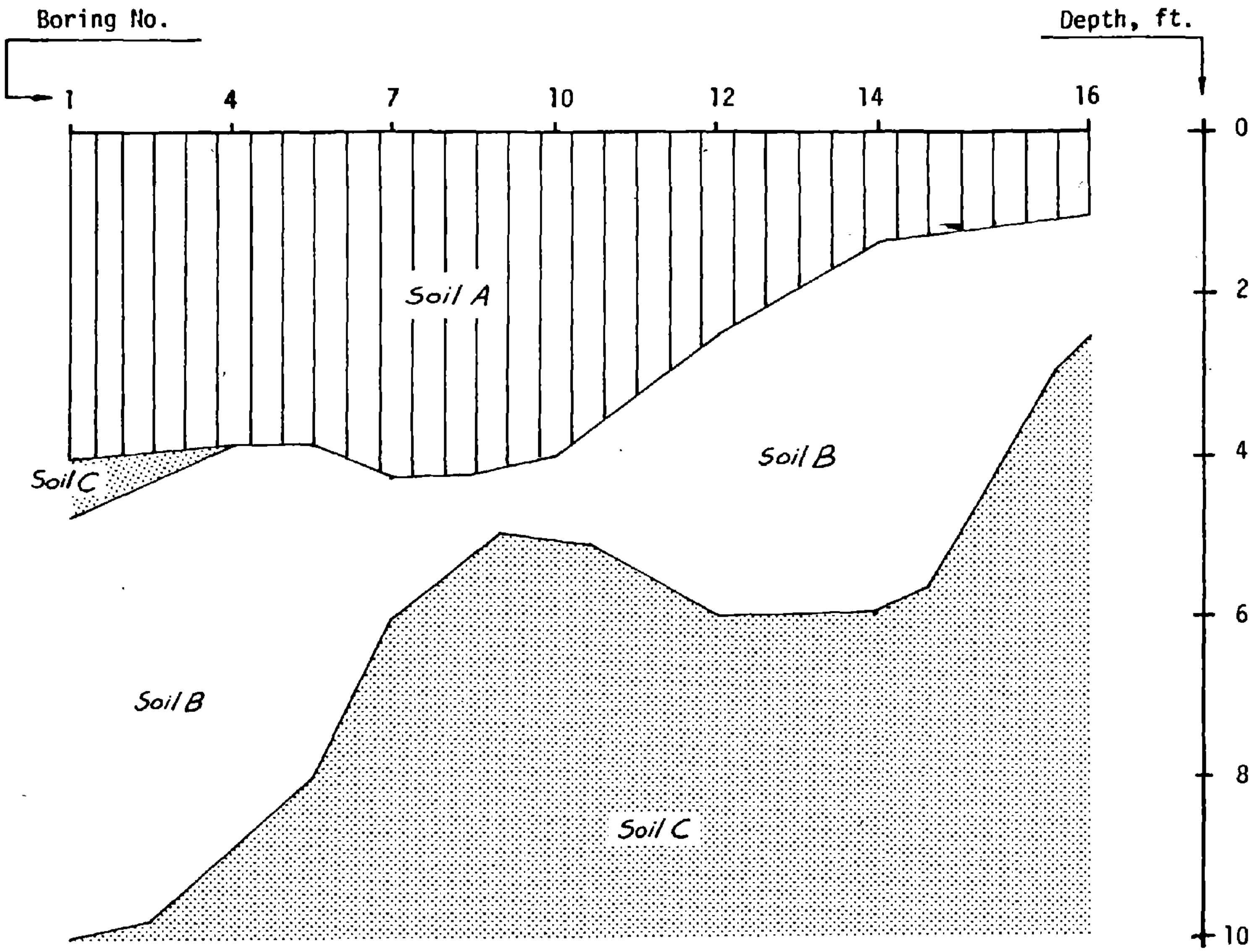


Fig. 3.7-2a. Longitudinal soil profile.

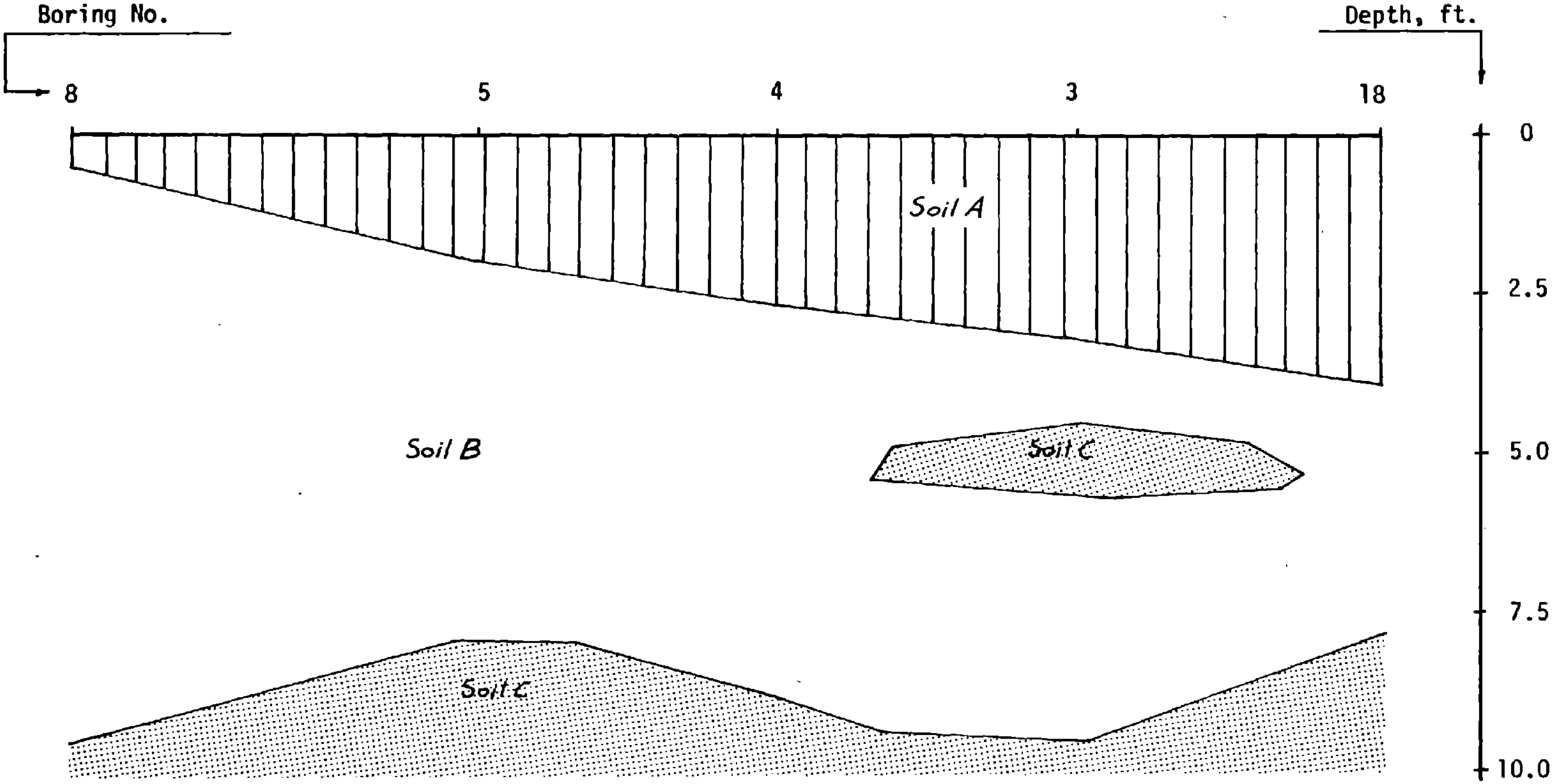


Fig. 3.7-2b. Transverse soil profile.

Soil C was over-consolidated, probably due to dessication. It is commonly called "Midway Clay" and is characterized by a grayish color and a blocky structure. The size of the blocks increased with depth, following the general trend of the Midway formation. The in situ water content variation can generally be summarized as:

1-3 ft.	20-25%
3-4 ft.	25-30%
4 ft.	30-42%

After further examination, a test program was designed and samples were allocated for the particular tests.

Tests were conducted in accordance with ASTM Part 11, 1964, and the Corps of Engineers Publication EM 110-2-1906, "Laboratory Soils Testing." The following tests were conducted: liquid and plastic limit, grain size analysis, water content, specific gravity, consolidation, and triaxial compression.

Boreholes 2, 6, 8, 17, and 18 were classified as untreated. The remainder were classified as treated to a depth of approximately 7 feet. Samples below this depth were classified as untreated.

Classification tests (LL, PL, specific gravity, grain size) were conducted on samples in all layers. The data revealed no discernable difference between the treated and untreated zones. There was no difference in pH of the treated and untreated soil.

Three sets of consolidation tests were preformed: (1) on treated soil, (2) on untreated soil, and (3) on untreated soil with the top and bottom surfaces smeared with lime slurry. These samples were smeared with

0.5 percent lime (by weight) in the form of a slurry and allowed to cure in a controlled atmosphere (100% RH, 72° F) for two weeks. A specific gravity of 2.80 was assumed for all samples; subsequent testing showed this to be high, but for comparison purposes, this assumption should be adequate.

A comparison of the test results revealed no trends evident in consideration of variation of initial moisture, initial dry unit weight, initial saturation, and preconsolidation load.

The coefficient of consolidation showed a marked increase in the case of the smeared samples. This is in accord with results from intimate mixing of lime with soil in which cohesion is decreased and particle size effectively increased, thus hastening consolidation. When viewed in comparison with the values for the treated soil, this could indicate a possible "fall-off" in the effects of lime treatment with time, though by the nature of the tests, this is certainly inconclusive.

The results from the free swell test might indicate that any form of lime treatment (LSPI or smearing) would decrease free swell. However, again this is inconclusive.

Consolidated-undrained triaxial tests (with and without pore-pressure measurement) were performed on both treated and untreated samples. The results of these tests were scattered and, therefore, difficult to interpret. This is partly due to the limit on the number of samples available for testing. Where possible, a Mohr-Coulomb failure envelope was drawn; but in some cases the tests had to be teated individually and the data, therefore, are not significant. Any grouping was done using classification data, in situ density, etc.

There was no detectable difference in the strength parameters due to the LSPI treatment.

The purpose of the LSPI treatment was to stabilize the upper 7 feet of soil, which is predominantly a silty-clay. This depth included the interface with the Midway formation.

None of the tests indicated that there is any conclusive detectable difference (22 months after LSPI treatment) between the untreated soil and the treated soil. However, this does not mean that the treatment was unsuccessful. The treatment of the same type of soil at the adjacent Huntington Place Apartments was shown to be quite successful. Visual inspection of the site used for this investigation shows that the LSPI treatment was successful. The surface of this road has remained unpaved since its construction, and though now rutted to depths of 4 to 6 inches, is still stable and generally trafficable despite relatively poor drainage in the immediate area.

4. RESEARCH REPORT SUMMARIES

The scope of the research that was proposed under this contract included the utilization of several graduate assistants, all of whom were working to complete the requirements for the degree of Master of Science in Engineering or Applied Mathematics. In the process of working on the lime injection research program, the students independently wrote several theses and special project research reports. Five such reports--by Blenden, Lawson, McAlister, and Greeson--are generic to the research project and are available through the Graduate Institute of Technology Library in Little Rock, Arkansas. These reports have been summarized in this section of the final report in subsections 4.1 through 4.5. Each of these reports is listed in the bibliography in the back of this report.

4.1. A STUDY OF THE EFFECT OF A LIME SEAM ON THE UNCONFINED COMPRESSION CHARACTERISTICS OF A SOUTH DAKOTA CLAY

by Chris H. Lawson

Introduction

Lime stabilization, an established art for more than 5000 years, is now an established science. In the development of this science, most of the knowledge has been gained in "conventional" lime stabilization. Lime slurry pressure injection (LSPI) is a modern approach[1] to soil stabilization and is used principally when the surface must remain intact or when stabiization at depth is required. Although LSPI originally was developed for use in rehabilitating houses constructed on expansive clay soils, it now is also used in pre-construction situations.

Detailed knowledge of the mechanisms by which LSPI improves a soil mass or renders it stable is still being developed. Current LSPI theory suggests that the treatment mechanisms are:[2]

1. Prewetting (due to the large volume of water injected),
2. Development of soil-lime moisture barriers (lime seams) in existing cracks, fissures, bedding-planes, and root lines and in cracks formed by the jetting action of the lime slurry,
3. Effective swell restraint with the formation of limited quantities of soil-lime reaction products, and
4. Strength increase due to soil-lime pozzolanic reaction.

The soil type which most favors successful LSPI treatment is one containing an extensive fissure and crack network into which large quantities of the lime slurry can be injected. Wright[3] has reported: "The lime slurry is deposited in horizontal sheet like seams, often interconnected with vertical or angular veins." This is especially true in a layered deposit, but in most instances the distribution is likely to be a maze of inter-connecting seams[4] (Figure 4.1-1).

Many properties are attributed to lime seams, the predominant one being that they form moisture barriers which tend to stabilize the moisture content of the soil mass. One important LSPI property which has received very little attention is the influence of lime seams on the strength of the soil mass. Material characteristics determined from unconfined compression testing of laboratory-manufactured lime seam specimens have been used in this study[5] in an effort to determine the influence of the lime seam on soil mass strength. Three types of specimens were tested in unconfined compression: intact specimens, specimens manufactured with a transverse crack at mid-height, and specimens manufactured with a transverse lime seam at mid-height.

Lime Seam Manufacture

Figure 4.1-2 shows a typical lime seam specimen. The mold used to manufacture the specimens is shown in Figure 4.1-3. The following steps were used in molding the specimens:

1. Mix two batches of soil and water (one for each half of the mold).
2. Place in plastic bags for equilbration for 7 days.
3. Apply a thin film of silicone grease to the inside of the mold.
4. Assemble the mold with the spacer in the center.
5. Fill one end of the mold with a batch of mixed soil and cap it.
6. Fill the other end of mold and insert a piston.
7. Replace the cap of step 5 with a piston.

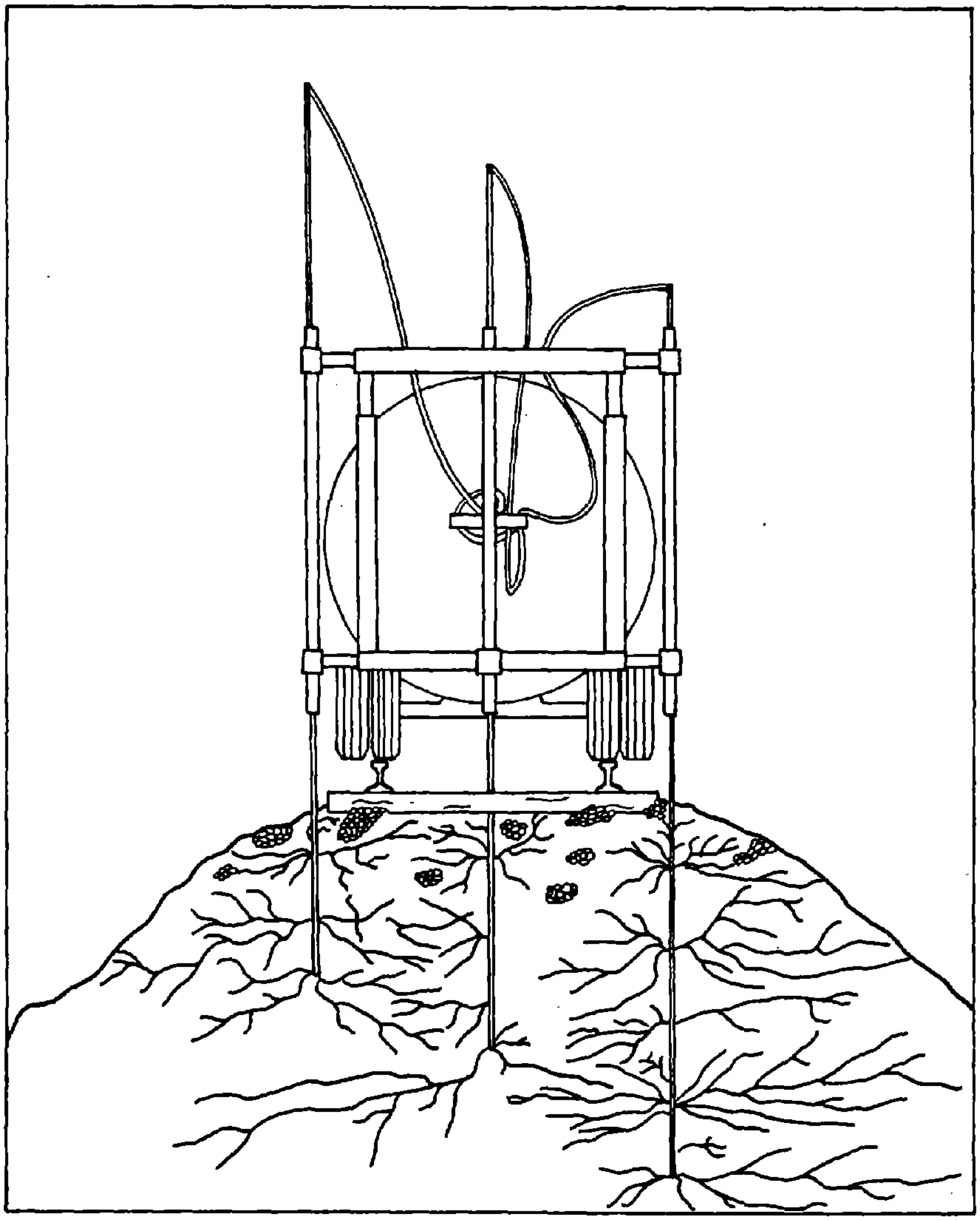
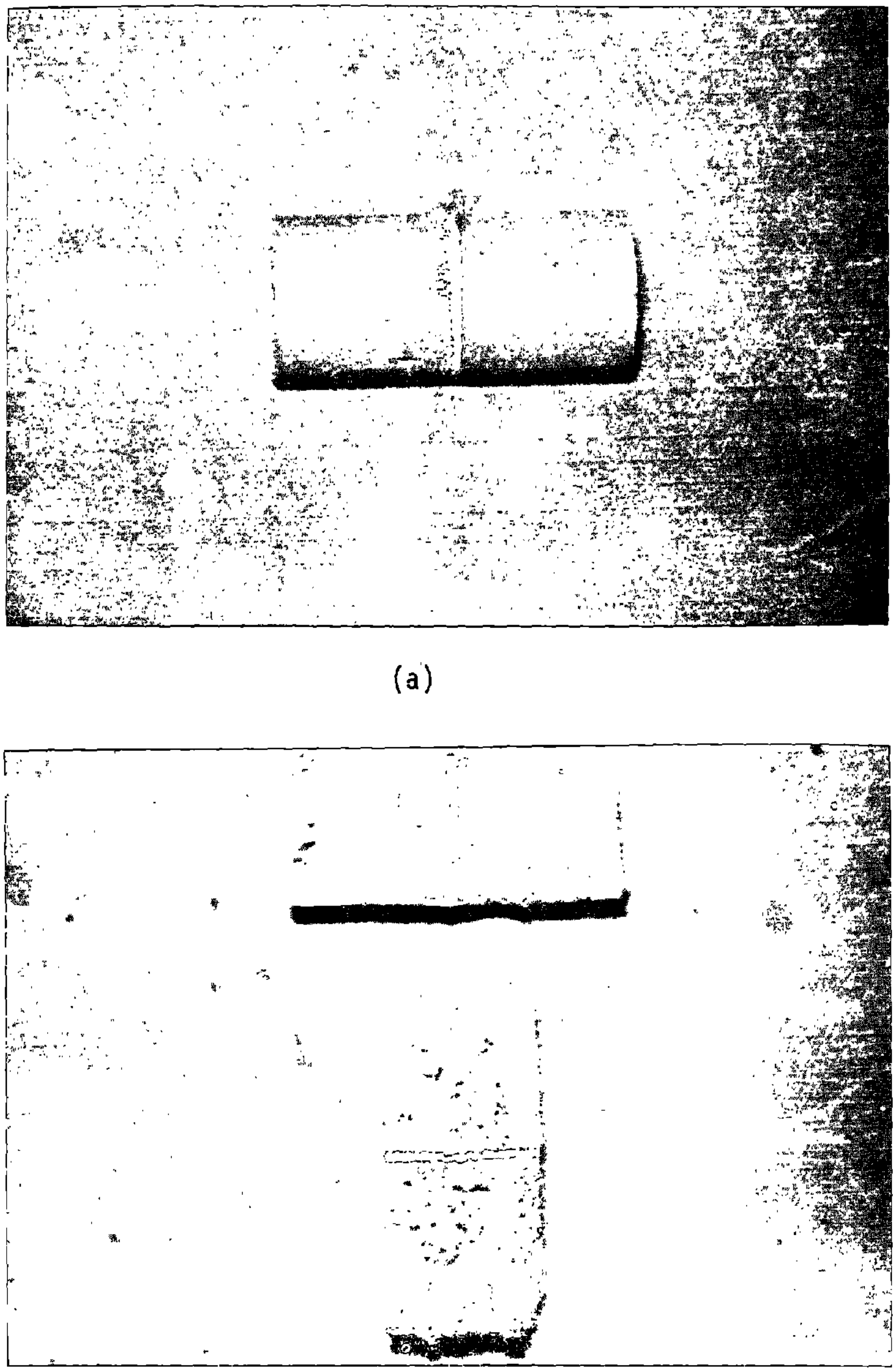


Fig. 4.1-1. Typical lime seam formation and distribution.



(a)

(b)

Fig. 4.1-2. Lime seam specimen.

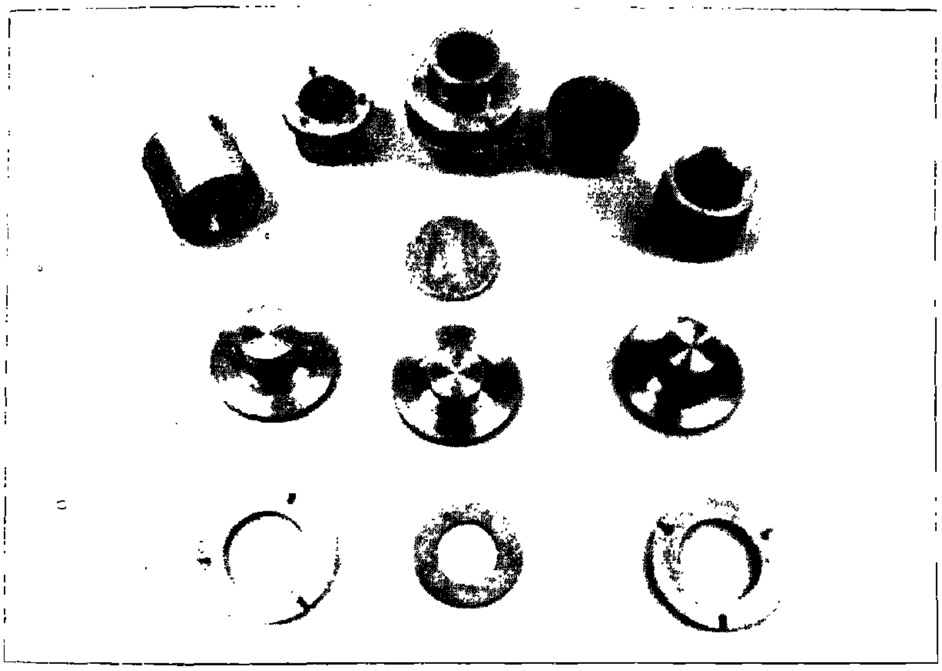


Fig. 4.1-3. Final lime seam mold design.

From Top Left:

Row 1 - Head Piece, Half Mold, Assembled Mold, Half Mold, Head Piece.
Row 2 - Spacer Plate.
Row 3 - Piston, Cap, Piston.
Row 4 - Collar, Lime Seam Ring, Collar.

8. Compress the soil by applying a load to the pistons with a hydraulic jack and leave it under load until soil relaxation is complete (approximately 2 minutes).
9. Remove the pistons and head pieces.
10. Disassemble the mold and remove the spacer.
11. Lightly scratch the surface of the soil which is to form part of the lime seam.
12. Place the lime seam ring on the mold.
13. Mix the lime paste and apply to the specimen until the lime seam ring is filled.
14. Reassemble the mold, squeezing out all the excess lime paste, and wipe the mold clean.
15. Place in a plastic bag and store under controlled moisture and temperature conditions for 24 hours.
16. Extrude the specimen.
17. Weigh, wrap, and mark the specimen, and place it in the controlled atmosphere for curing.

In manufacturing intact specimens, step 4 is modified in that the spacer is not used. Steps 5 and 6 are combined and steps 10 through 14 ignored. In manufacturing specimens with a formed cracked, step 12 is deleted and step 13 modified in that 1 ml of water, rather than the lime paste, is placed in the crack.

Soil Type

The soil was obtained adjacent to the Chicago and Northwestern Railroad track near Belle Fourche, S.D. The index properties of this soil--a dark brown, plastic clay--are described in column 1 of Table 4.1-I.

An experiment was conducted to determine the effects of lime on various properties of the soil. Two percent (by weight of the oven dry soil weight) lime was used in all cases.

TABLE 4.1-I
Soil Paremeters

Parameter		No Lime	2% Lime
In-Situ Density (pcf)		84.4-113.9	...
In-Situ Moisture Content (%)		10.3- 27.3	...
pH		4.0- 4.5	...
Grain Size (% Finer than)	No. 10[a]	99.9	...
	20	99.5	...
	40	98.6	...
	60	96.6	...
	100	93.2	...
	200	90.1	...
	2 microns	50.0	...
Liquid Limit		55	63
Plastic Limit		26	35
Plasticity Index		29	28
U.C. Strength (psi)		37.97	60.50[b]
Unified Classification		CH	CH
AASHTO Classification		A-7-6(19)	A-7-6(19)

[a] U.S. Standard Sieve,
[b] T-test detects a difference of 18.53 psi at = 0.05.

The data for the soil-lime mix is summarized in column 2 of Table 4.1-I. While the lime did not reduce the plasticity index, it did increase the plastic limit (PL) which is generally indicative of a potential strength increase. Unconfined compression testing indicated a 50 percent strength increase due to the lime treatment.

Presentation and Discussion of Data

A total of 96 specimens (6 blocks of 16) were tested in unconfined compression. Specifications for the experiment were:

Block 1 = Intact, 7 day cure

Block 2 = Intact, 28 day cure

Block 3 = Formed crack, 7 day cure

Block 4 = Formed crack, 28 day cure

Block 5 = Lime seam, 7 day cure

Block 6 = Lime seam, 28 day cure

Design Dry Density = 90 pcf

Design Moisture Content Range = 19.0 to 26.0 percent

Specimen Height = 3.00 inches plus lime seam (where necessary)

Specimen Diameter = 1.35 inches

Design Strain Rate (average) = 0.0015 inches/minute

Lime Paste = 2 gm of lime, 4 ml of water

Molding and testing sequence = Random

The analysis showed that quality control was good. Two-way analysis of variance was used to investigate possible differences in the unconfined compression strength (peak stress, S_p) and Young's modulus of elasticity (least-squares modulus, LSM), but the analysis revealed no meaningful information.

Multiple linear regression was then used to model each of the dependent variables as a function of all the independent variables. Unconfined compression strength (S_p) was the only dependent variable that showed good correlation.

Analysis of the individual blocks showed that S_p was strongly linearly dependent on moisture content (MC) (see Table 4.1-II):

$$S_p = C_1(MC) + C_2, \tag{1}$$

which was to be expected because the moisture range determined at testing (17.0 to 24.0 percent) was less than the PL. The moisture range was less than that specified because of a lack of humidity control in the laboratory.

TABLE 4.1-II
Coefficients for Equation 1

Block	C_1	C_2	CD[a]
1	-3.3348	113.71	0.835
2	-2.5513	96.15	0.616
3	-3.2403	109.55	0.906
4	-3.8209	122.16	0.910
5	-2.6987	97.56	0.791
6	-2.2690	88.47	0.693

[a] CD = Coefficient of Determination

The blocks were further analyzed in groups of two to determine the effect of curing period, crack, and lime seam. Four comparisons were made:

(1) 7 - day vs. 28 - day curing

(2) intact specimens vs. cracked specimens

(3) intact specimens vs. specimens with a lime seam

(4) cracked specimens vs. specimens with a lime seam

Equation 2 gives the general first model used:

$$S_p = C_1(DD) + C_2(MC) + C_3(SE) + C_4(ST_p) + C_5 + C_6(D_i)^* \qquad (2)$$

where

S_p = peak stress or unconfined compression strength.

DD = dry density

MC = moisture content

SE = strain energy absorbed by specimen in reaching peak stress

ST_p = strain at peak stress

C_1, C_2, ..., C_5 = regression constants

In all instances, the effect of parameter D_i on S_p was very small (from none to 4.5 percent).

During the manufacture of the cracked specimens, 1 ml of water was added to enable some bond to form. The effect of increased curing on these specimens was to "heal" the crack. After 7 days curing, the cracked specimens were not as strong as the intact specimens; but after 28 days, there was no detectable difference in their strengths. The same trend was evident for the lime seam specimens. Though at no time during the test period were they as strong as the others, the difference was very small. This information is summarized in Table 4.1-III.

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*For i = 1 (i.e., first block in the comparison), D_i = -1. For i = 2 (i.e., second block in the comparison), D_i = 1. Thus, for the four comparisons respectively, D_i = -1 for (1) 7 day curing, (2) intact specimens, (3) intact specimens, and (4) cracked specimens, and D_i = 1 for (1) 28 day curing, (2) cracked specimens with a formed crack, (3) specimens with a lime seam, and (4) specimens with a lime seam.

TABLE 4.1-III
Influence of the Crack or Lime Seam on
the Strength of the Specimens

	Influence of d_i after 7 days during (%)	Influence of D_i after 28 days curing (%)
Intact/Cracked	2.5	0.1
Intact/Lime Seam	4.5	2.1
Cracked/Lime Seam	0.4	1.0

The coefficients for all analyses are given in Table 4.1-IV.

TABLE 4.1-IV
Linear Model Coefficients for Equation 2

Blocks	C_1	C_2	C_3	C_4	C_5	C_6	CD^a
1 & 2	-1.847	-2.238	10.172	-11.350	258.31	-1.321	0.789
3 & 4	-1.396	-3.579	2.324	- 1.320	241.13	0.664	0.924
5 & 6	0.928	-2.227	7.221	- 6.638	2.08	0.258	0.835
1 & 3	-1.269	-2.857	5.066	- 5.013	216.98	-1.822	0.904
2 & 4	-1.281	-2.405	4.831	- 9.543	214.16	0.738	0.800
1 & 5	0.863	-2.558	7.750	- 9.055	18.96	-1.224	0.892
2 & 6	-1.087	-2.057	8.839	- 7.309	180.12	-1.798	0.760
3 & 5	0.318	-2.395	6.030	- 6.981	63.12	-0.178	0.894
4 & 6	-0.493	-3.169	3.808	- 2.174	149.47	-0.682	0.836

[a] CD = Coefficient of Determination.

Conclusions

The purposes of this investigation were to study the effect of a lime seam on the strength of a soil specimen and to establish a method for conducting further research in this field.

The difference in curing periods had no effect on the strength (peak stress) of the intact specimens. However, it did affect those specimens with cracks and lime seams. Those with cracks showed an increase in strength with additional curing, 28 days being sufficient for the crack to "heal." Specimens with lime seams showed the same effect but to a lesser degree.

Specimens with cracks did not model a fissured soil as planned, but rather a fissured soil in which the cracks had filled with water and subsequently closed. Specimens with cracks or lime seams did not have greater strength than the intact specimens, though they may have had equal strength. Specimens with lime seams always had lower strengths than both the intact specimens and those with cracks. This difference, however, became less evident with increased curing time. The strength difference was never very great, which indicates that a lime seam may do as well in repairing the cracks in a soil mass as the normal seasonal fluctuations in moisture content which, at certain times of the year, render the soil mass effectively intact.

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4.2. A STUDY ON THE EFFECTS OF OF A SATURATED SOLUTION FROM LIME SLURRY ON THE STRENGTH OF A SOUTH DAKOTA CLAY

by Robert E. Blenden

Introduction

Many inferences concerning the mechanisms of LSPI have been based on data compiled from the intimate mixing and recompaction of lime and soils. Although some research specifically dealing with LSPI has ben performed and some LSPI theories have been proposed, few definite conclusions have been drawn explaining the principal causes of the beneficial effects that are attributed to LSPI.

The primary functional difference that sets LSPI apart from conventional grouting techniques and associates it with mechanical mixing of lime and soil is the intent to use the soil chemistry in reaction with lime to transform soil properties. Furthermore, the instances in which lime injection and in which chemical grouting are used differ. LSPI is applicable mainly to fine-grained soils, whereas grouting is usually associated with problems encountered in rocks and coarser-grained soils.

The in-place treatment by LSPI may represent an economical alternative to replacement or structural compensation. But more questions must be answered and more data gathered to enable the proper engineering application of the technique. It is important to know how lime injection does or does not affect the soil.

Theory of lime Injection

At present, it is generally accepted that lime slurry which is forced into the ground under pressure will follow cracks and other planes of weakness, thereby forming layers oriented in various directions.[1,2] These layers vary in thickness and number, depending upon the lime-to-water ratio of the slurry and also on the nature and condition of the soil.[3]

The lime seams lose moisture through permeation of the water into the surrounding soil. At the same time, lime particle migration occurs, and chemical reactions initiate between the soil and the lime.[4,5] The lime seam is defined, for the purpose of this investigation, as the material layer, the boundaries of which are determined by the maximum distance of migration of undissolved particles of lime away from the original seam of lime slurry. Thus, a lime seam will contain suspended particles of lime-soil by-products and the soil within the defined boundaries.

All information presently available shows that lime particles do not permeate a great distance from the original slurry seam in fine-grained soils.[1] The distance depends upon the relative size of the lime and soil particles and is generally no more than one or two inches, even with the sustained use of high pressures.[1,4,6] The distance between lime seams has been seen to vary from several inches to several feet, often within a single injection site.[7] In general, the quantity of lime specified to be injected amounts only to approximately 0.25 percent to 1.50 percent of the soil mass by dry weight. With this relatively small proportion of the soil mass consisting of actual seams, an important question to be answered is: Will the bulk of the soil mass lying between lime seams be significantly affected by the inclusion of the seams, and if so, in what manner?

The solubility of calcium hydroxide, $Ca(OH)_2$, is approximately 1.70 grams per liter of water at room temperature, with a slight variation that is inversely proportional to the temperature.[8] Thus, an increase in the moisture content of a soil by 5 percent due totally to the addition of a saturated solution of calcium hydroxide would represent the addition of approximately 0.0085 percent lime by weight. It should be noted that this value is an overall value of moisture content. Increases in the moisture content of the soil immediately adjacent to lime seams (in this experiment adjacent to the point of application) are much higher. There is little in the literature that attempts to explain the mechanism and effect of small amounts of lime in a soil.

Some experiments were conducted using strontium hydroxide, $Sr(OH)_2 \cdot 8H_2O$, which is an active base chemically similar to calcium hydroxide but with a solubility approximately three times that of calcium hydroxide. It was included as an additive in a solution used for treatment of samples in one test group. It was felt that perhaps the increased solubility of the strontium would tend to show more pronounced effects on the strength of the soil samples. Additionally, strontium had been used in a lime injection experiment on the Rock Island Railroad, and it appeared that there might be a chance to corroborate test results.

Laboratory Investigation

This investigation was directed at answering fundamental questions about the injected slurry and the surrounding soil--in particular about the products of this interaction in the zone adjacent to but not including a lime seam, that is, the zone permeated by the lime supernatant solution.

The benefits attributed to lime injection include dewatering, stabilization of moisture content, reduced swelling potential, and increased strength.[9,4,2] This investigation is concentrated on the strength-increase phenomena. In the case of mechanical intimate mixing of lime and soil, this is probably the best documented benefit. The main soil parameters that were measured for analysis during this investigation were the shear strength, Q_u, and Young's modulus of elasticity, E. These were measured by performing unconfined compression tests on molded samples. The initial tangent modulus was used for evaluating the modulus of elasticity of the specimens.

The primary solution used in this experiment was the clear liquid decanted from a settled lime slurry mixture. The liquid contained water and any dissolved material resulting from the physical mixture of 2 1/2 pounds of lime per gallon of distilled water. A second solution was also used in this study--the supernatant liquid resulting from the mixture of lime, distilled water, and strontium hydroxide.

In virtually all previous studies concerning lime-soil mixtures, the test specimens were molded after the lime had been mixed with the soil and, in many cases, allowed to equilibrate for a period of time. In this experiment, all samples within a comparative group were molded identical in size, composition, density, and moisture content; and the lime supernatant treatment was effected a short period after compaction.

This method of treatment of the specimens was adopted because it resembled as closely as possible that which occurs in the field and because reactions take place immediately when lime comes into contact with reactive soils. Some of the benefits due to these reactions may be lost by delayed compaction after the lime and soil are intermixed. Further, because some soil properties that effect the molding procedure are changed prior to molding, an additional variable is introduced. Thus, differences found from strength tests may be due to different molding characteristics between control samples and those treated prior to molding with lime. This testing has no direct relation to LSPI, in which no remolding or removing occurs subsequent to the treatment with lime. Tests presently conducted as an engineering approach to the use of LSPI consist mostly of intimate mixing of lime, lime slurry, or supernatant. They are used only as an indication of the lime reactivity of the soil in respect to certain soil properties and are not directly indicative of the _in situ_ results to be expected from LSPI. The molding-treatment procedure followed here is intended to more closely represent the actual LSPI treatment conditions.

The soil used in this study was a lime reactive, dark brown, plastic, montmorillonitic, highly expansive clay from the Chicago and Northwestern track roadbed in Belle Fourche, S.D.

The molded specimens were allowed access to a given quantity of solution, with no attempt to force the fluid into the sample. Thus, the suction characteristics, as well as the permeability of the soil, determined the rate at which the fluid entered the sample and the distance it penetrated. The device used to support and seal the samples during treatment consisted of a clear Plexiglas tube split down the side to allow easy insertion and removal of the sample. The ends were sealed with brass plugs that had fittings for adding the solution.

To introduce the solution into the sample, a 7/64-inch-diameter hole was drilled lengthwise down the center of the sample. The volume of soil removed by this process was only .60 percent of the sample by volume. The solution was administered by filling the hole and reservoir tube (Figure 4.2-1 and 4.2-2).

The data for all test groups were analyzed using two-way analysis of variance, Duncan's multiple range test, and the T-test.[11,12] A significance level of $\alpha = 0.05$ was used. To fund the initial tangent modulus values, the stress-strain curve was plotted, and the slope of the

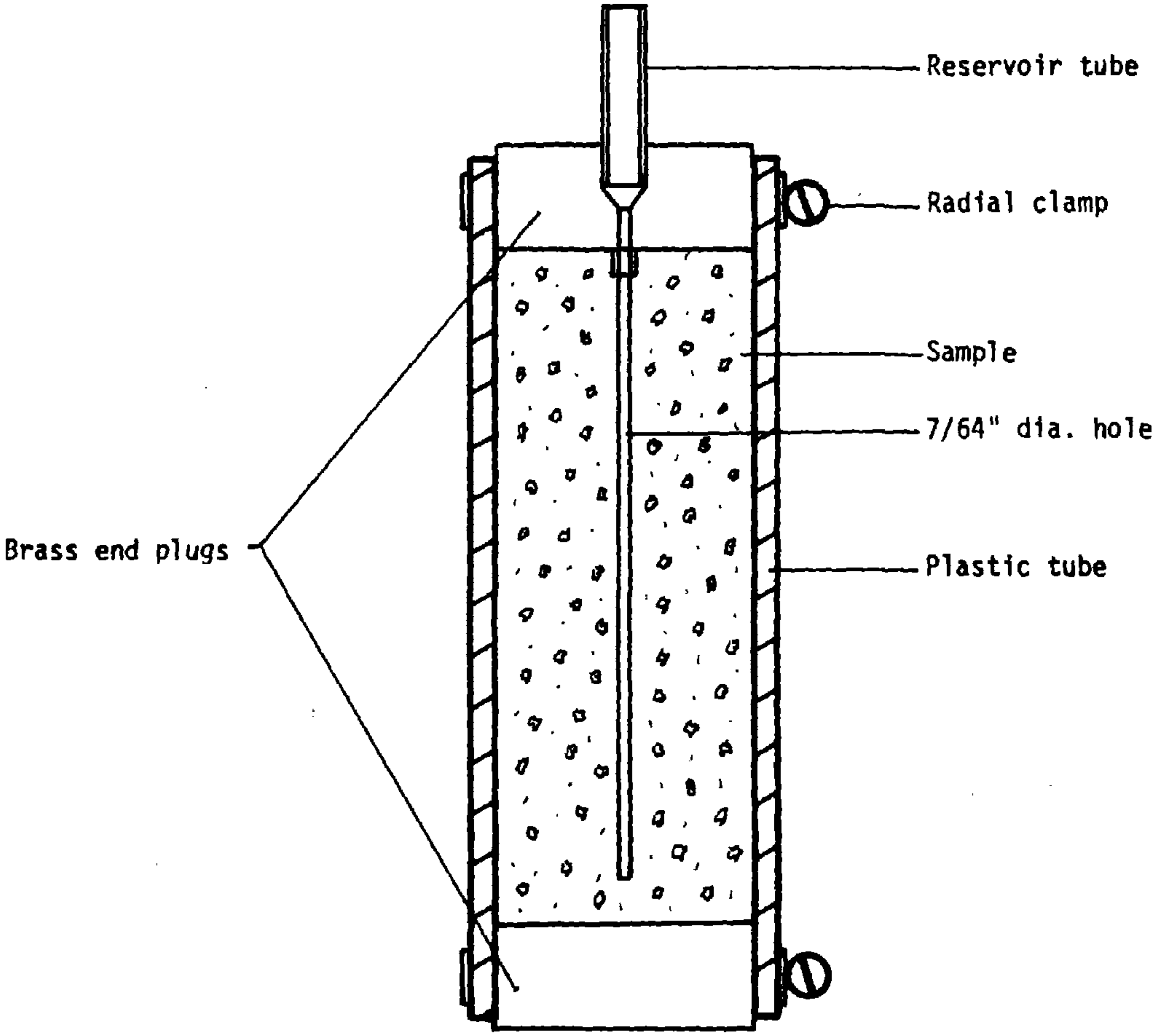

Fig. 4.2-1. Cross-section of treatment apparatus.

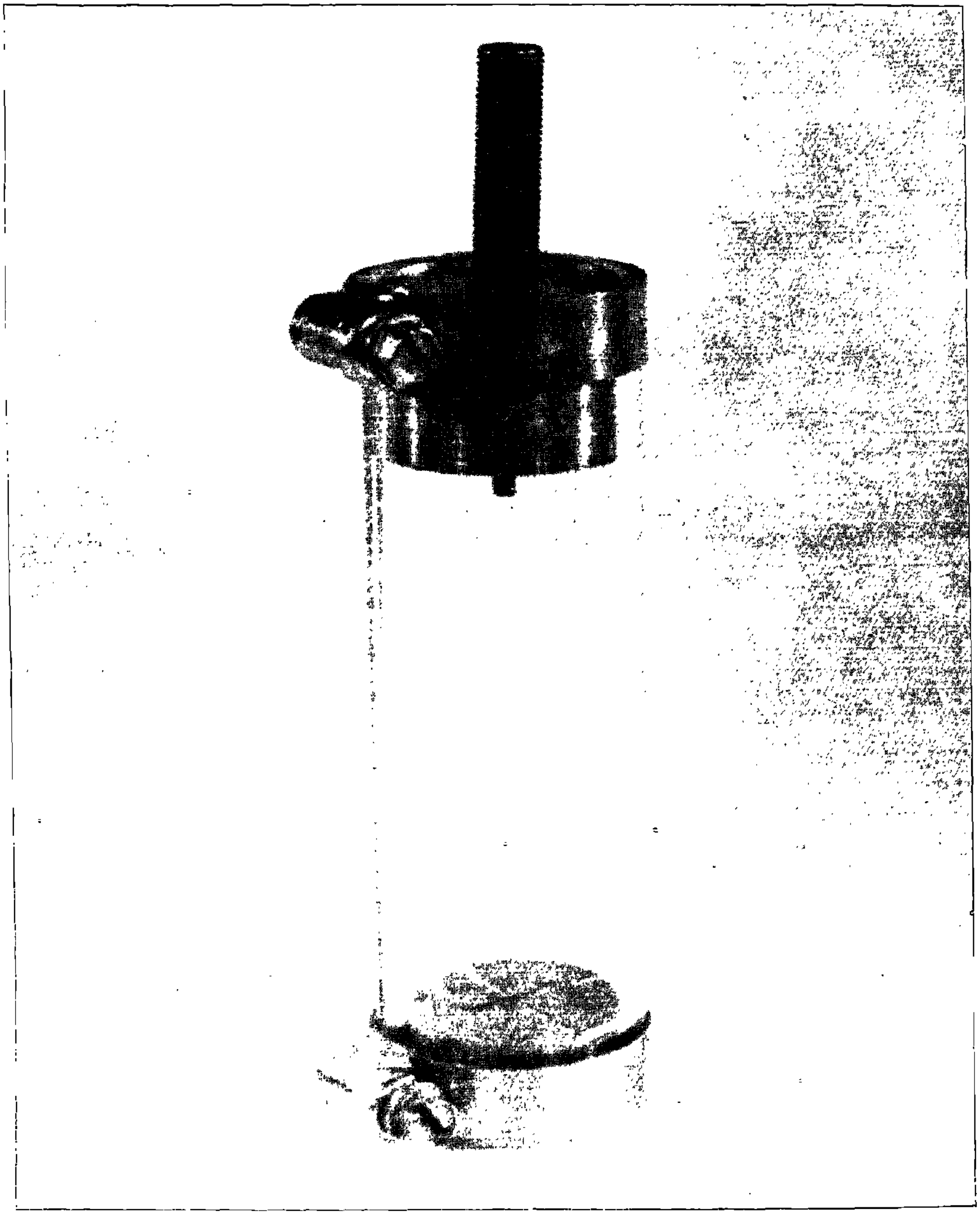

Fig. 4.2-2. Apparatus for internal application of solution.

straight line portion was found by the least squares method.

Summary and Conclusions

Interpretation of the results of analysis indicates that there was very little, if any, strength increase brought about by the addition of a lime slurry supernatant liquid to this soil. The compatability test showed that the addition of 2 percent lime brought about a strength increase of about 50 percent, while the addition of 18 percent by weight of the calcium-saturated solution did not significantly affect peak strength, although a slightly higher mean value of peak strength was noticed. The internal addition of the same saturated solution did not affect peak strength values. Although the means of the initial tangent modulus values were generally higher in the supernatant-treated samples, this difference was not significant, as demonstrated by the analysis of variance.

Another result of this experiment was the demonstration of a new laboratory technique that may be of further use in the study of LSPI. This technique proved to be a satisfactory method of putting a supernatant solution into contact with a compacted soil sample. Data compiled using this method is more directly related to LSPI than data compiled on the intimate mixing and recompaction of lime and soil.

Both the Atterberg limits test with lime and the lime compatability tests indicate that the Belle Fourche soil is only slightly to moderately lime reactive. This was not expected; and it is very probable that, if this study for strength reactivity data had been conducted in the preliminary planning phase, a soil better suited for the program would have been chosen. The determining factors in the choice of the Belle Fourche soil were that it was being used in several parallel studies in the GIT and that X-ray diffraction had shown the presence of a significant amount of montmorillonite.

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4.3. FINITE ELEMENT ANALYSIS OF CONVENTIONAL RAILWAY TRACK SUPPORT SYSTEMS

by Chris H. Lawson

[EDITOR'S NOTE: The development of the railway track structural analysis program is outlined in Section 2.6. Lawson[1] continued the development work of McAlister,[2] Blacklock,[3] and others in studying the quasi-three-dimensional procedure and developed the use of a grid-refinement procedure to enhance the analysis capabilities of the program RR.]

A brief series of parametric studies was conducted to study the influence of different tie and ballast stiffnesses and the influence of the longitudinal element thickness. Figure 4.3-1 is a schematic representation of the track support system.

The analysis of the influence of tie and ballast stiffnesses determined that two limiting states exist:

1. For a completely flexible tie (zero stiffness) on an infinitely stiff ballast, the wheel loads are effectively point loads applied to the ballast, and the highest stress concentration occurs under the loads.
2. For an infinitely rigid tie (infinite stiffness) on a ballast of zero (or very low) stiffness, maximum shear will occur at the ends of the tie, and the highest stress concentrations will occur three also.

For any other combination of stiffnesses, the peak stress will occur at any point under the tie other than at the ends. Analysis showed that it is possible for the peak stress point to move inside the load (i.e.,

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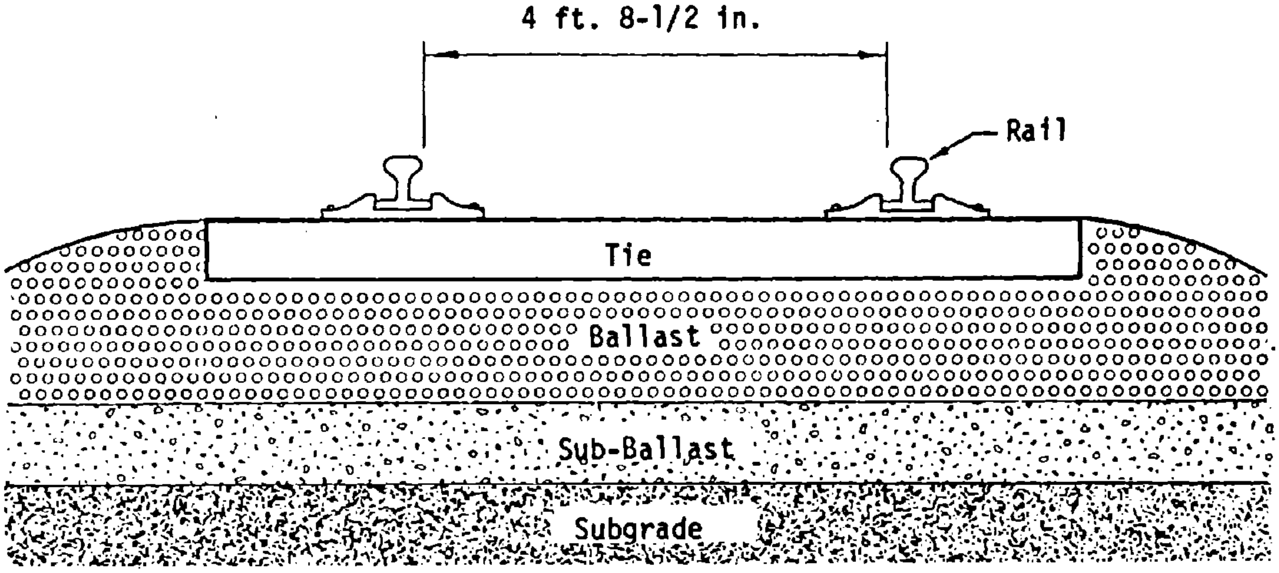
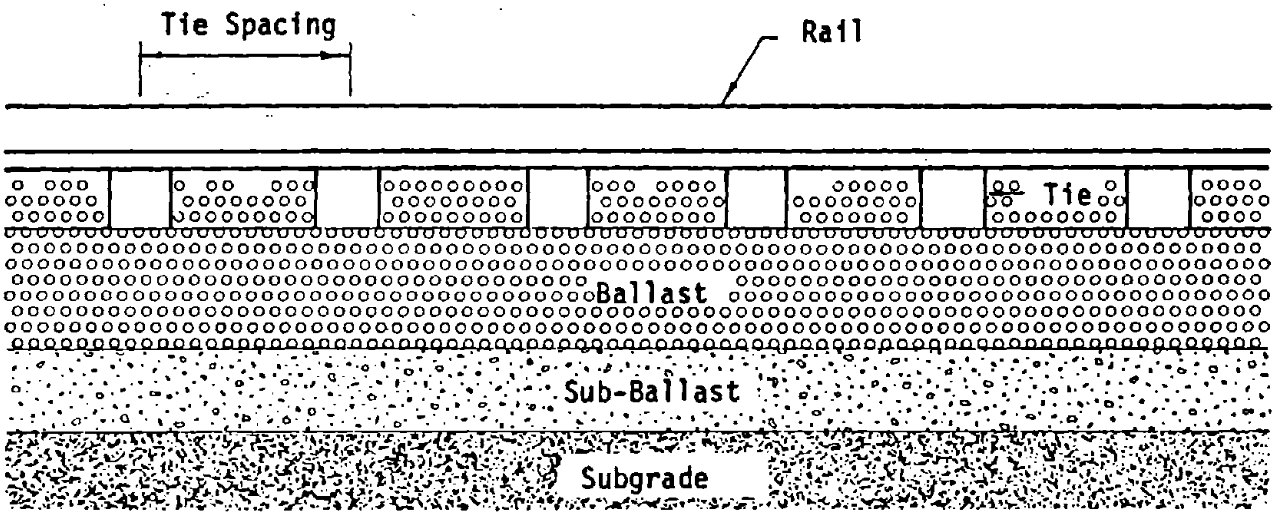


(a) Cross-Section



(b) Longitudinal Section

Fig. 4.3-1. Typical cross-sectional and longitudinal profiles of a conventional railway track.

between the rail and the track center-line), and it is envisioned that a peak could occur at the center-line

In the quasi-three-dimensional procedure for determining stresses in the longitudinal section, element thicknesses are determined from analysis of the transverse section analysis as:

$$\mathrm{LET} = (\mathrm{LAS}/\mathrm{FLAS})\mathrm{FLET},$$

where

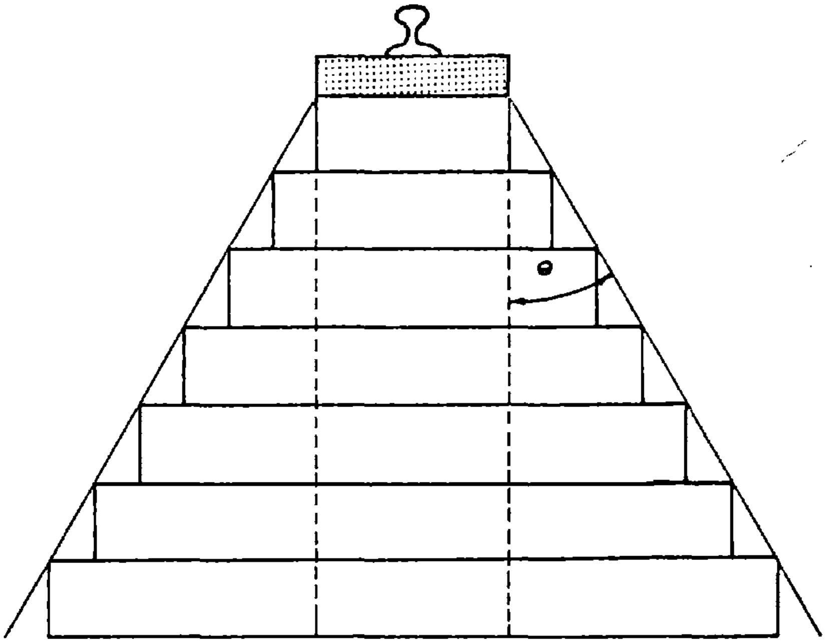
LET = Layer element thickness for logitudinal section

LAS = Layer average stress from transverse section analysis

FLET = First layer element (nominal) thickness for logitudinal section

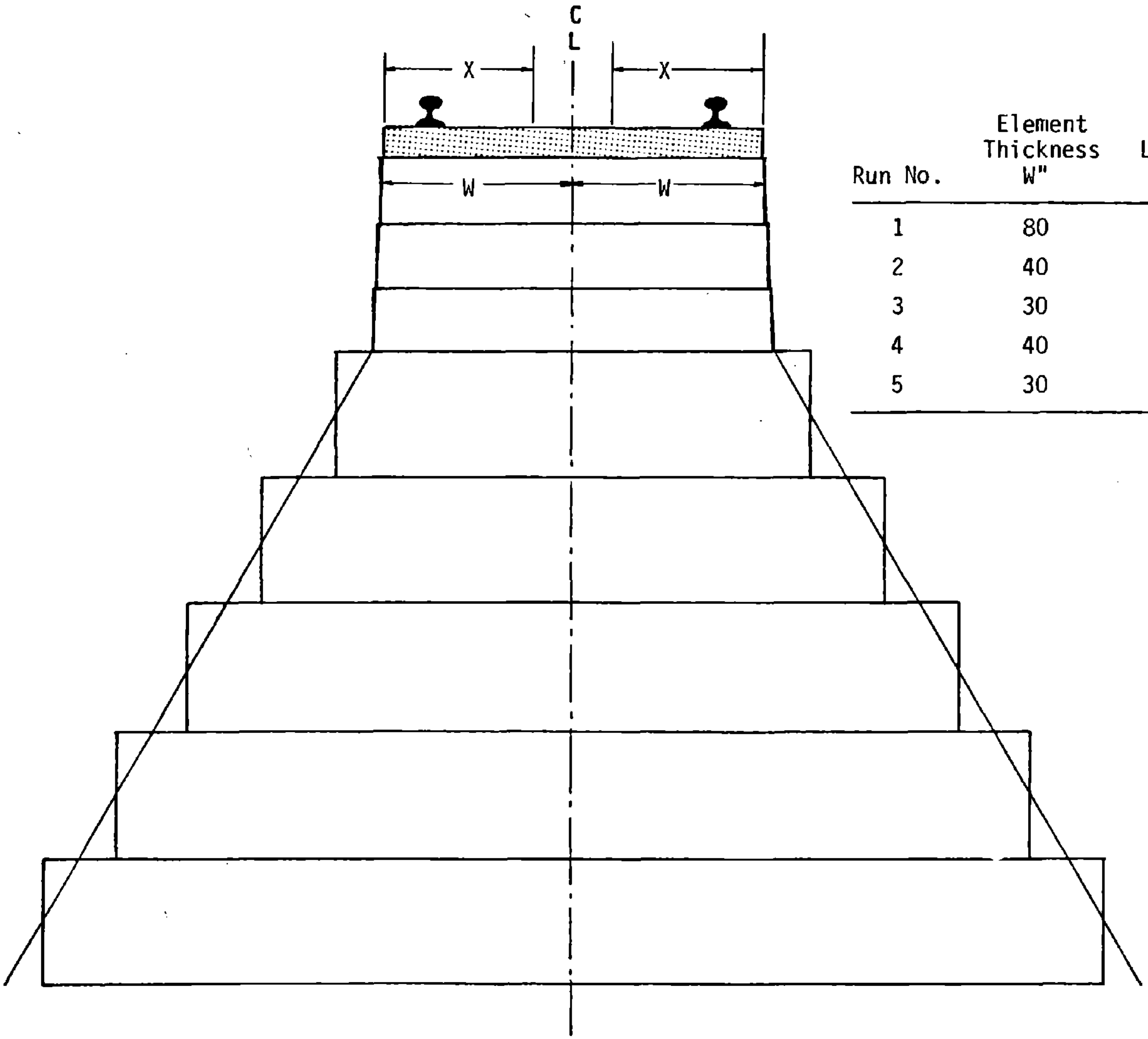
FLAS = First layer stress from transverse section analysis

Lawson describes the method in more detail, but it is obvious that, since stress decreases with depth, thickness of the element increases with depth. Tayabji[4] (Figure 4.3-2) used the assumption that the element thickness should increase at the rate of h tan 30^0, where h is the element height and 30^0 is an angle subtended with a vertical axis through the transverse simulation. In his analyses, Lawson (Figure 4.3-3) determined that the 30^0 assumption was useful for the subgrade but that an angle of 2.6^0 was required for the ballast. However, the agreement is remarkable since Tayabji used repeated load data while Lawson used static load data. The transverse section simulation is given in Figure 4.3-4. The vertical boundaries are supported against horizontal movement while the lower horizontal boundary is supported against vertical movement. Rotation is prevented at the tie center-line.



(θ = 30°)

Fig. 4.3-2. Representation of element thickness spread (after Tayabji).



Run No.	Element Thickness W"	Tie Length X"
1	80	80
2	40	40
3	30	30
4	40	80
5	30	80

Fig. 4.3-3. Simplified variation in element thickness with depth.

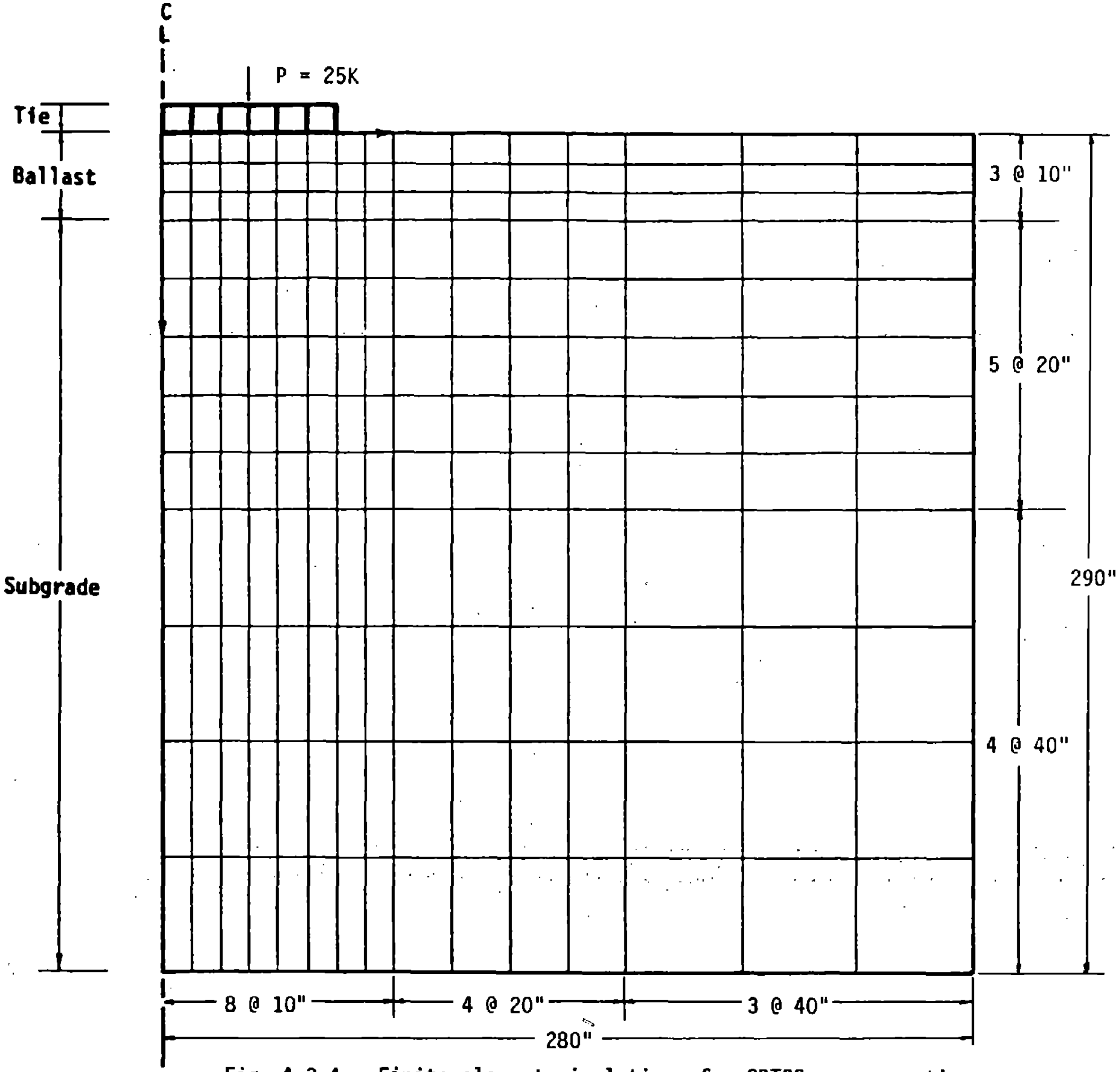


Fig. 4.3-4. Finite element simulation of a CRTSS cross-section.

The thickness of the first layer element (FLET) in the logitudinal simulation (Figure 4.3-5) was varied to study its effect on the vertical stresses due to the wheel loads. Figure 4.3-6 shows stress contours for a FLET of 80 inches that are typical of those obtained for all values of FLET. Peak stresses for 80, 40, and 30 inch FLET values were 4.9, 9.1, and 11.8 psi, respectively. The relationship is not linear, which is to be expected due to the nonlinearity of the problem.

Since this program has not been calibrated against field data, the exact magnitude of the stress at any point in the system is unknown, but the FLET parameter is one parameter which can be varied to ensure exact calibration.

Grid refinement was found necessary for two reasons:

1. The core available for the program was not large enough to allow a sufficiently accurate simulation.
2. Finer mesh grids would allow analysis of stress concentrations and other points of interest, such as the tie-ballast interface.

One further advantage of grid refinement is that a large simulation containing a great variation in element sizes (from very large to very small) can lead to ill conditioning of the stiffness matrix. This would be avoided.

The fine grid analysis gives the stress contours shown in Figure 4.3-7. The refined grid accents the concentrations of stress, showing that there are large concentrations under the ties, which is to be expected. It is thus concluded that the grid refinement gives a more detailed picture of the actual stress state.

The stress state at the ballast-subgrade interface was also investigated (Figure 4.3-8), and it can be seen that the fine grid analysis

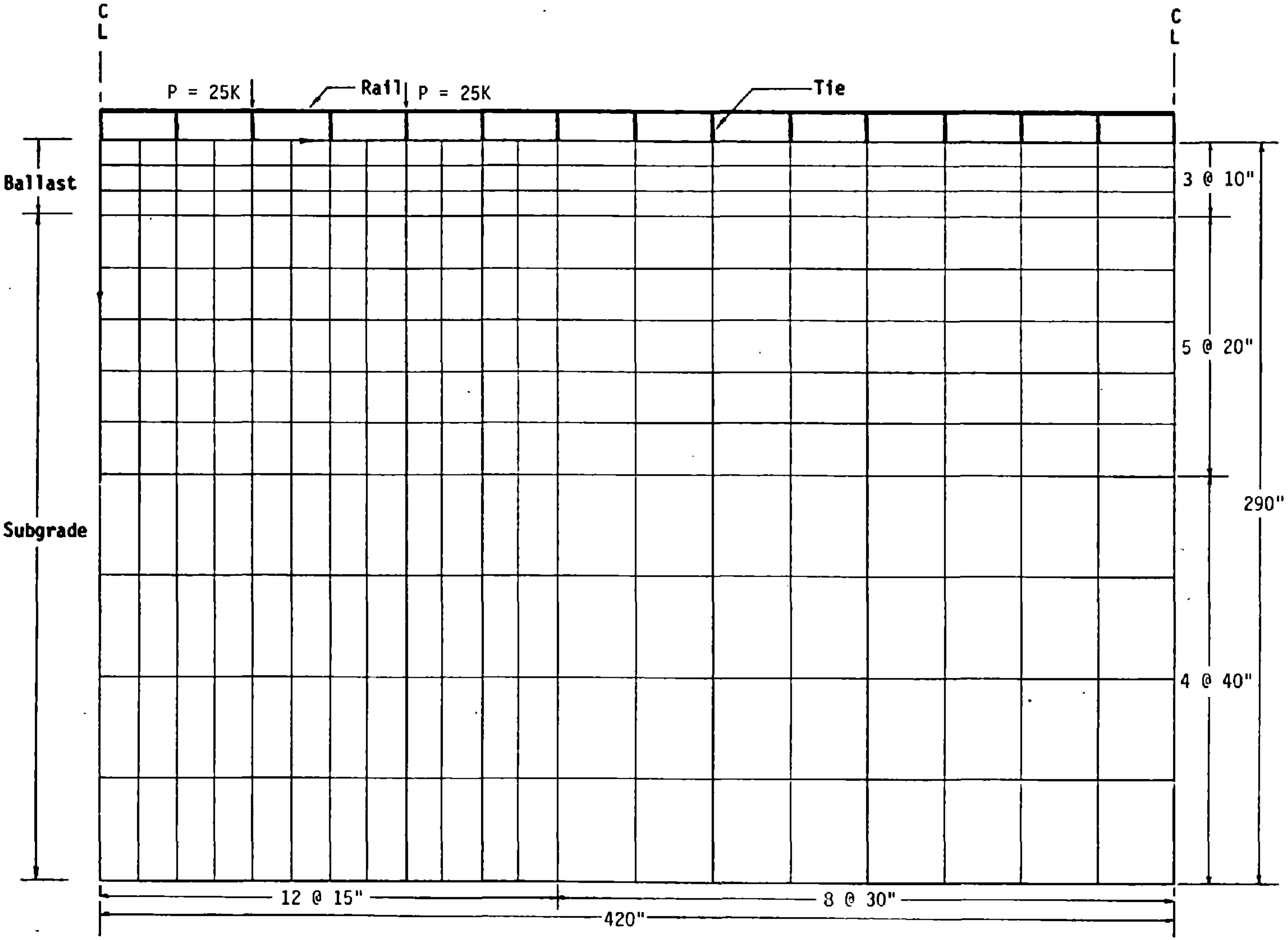


Fig. 4.3-5. Finite element simulation of longitudinal section.

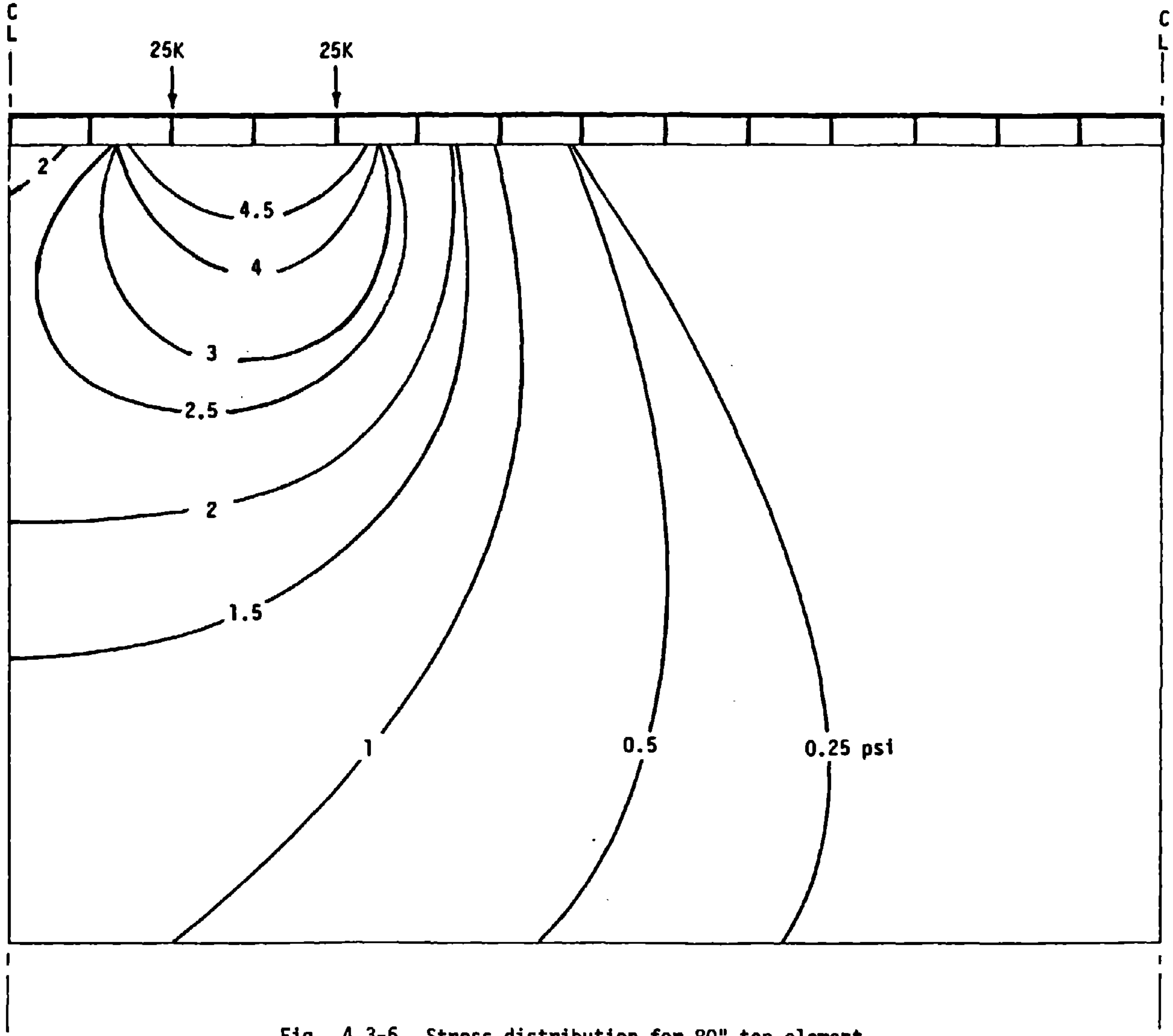


Fig. 4.3-6. Stress distribution for 80" top element.

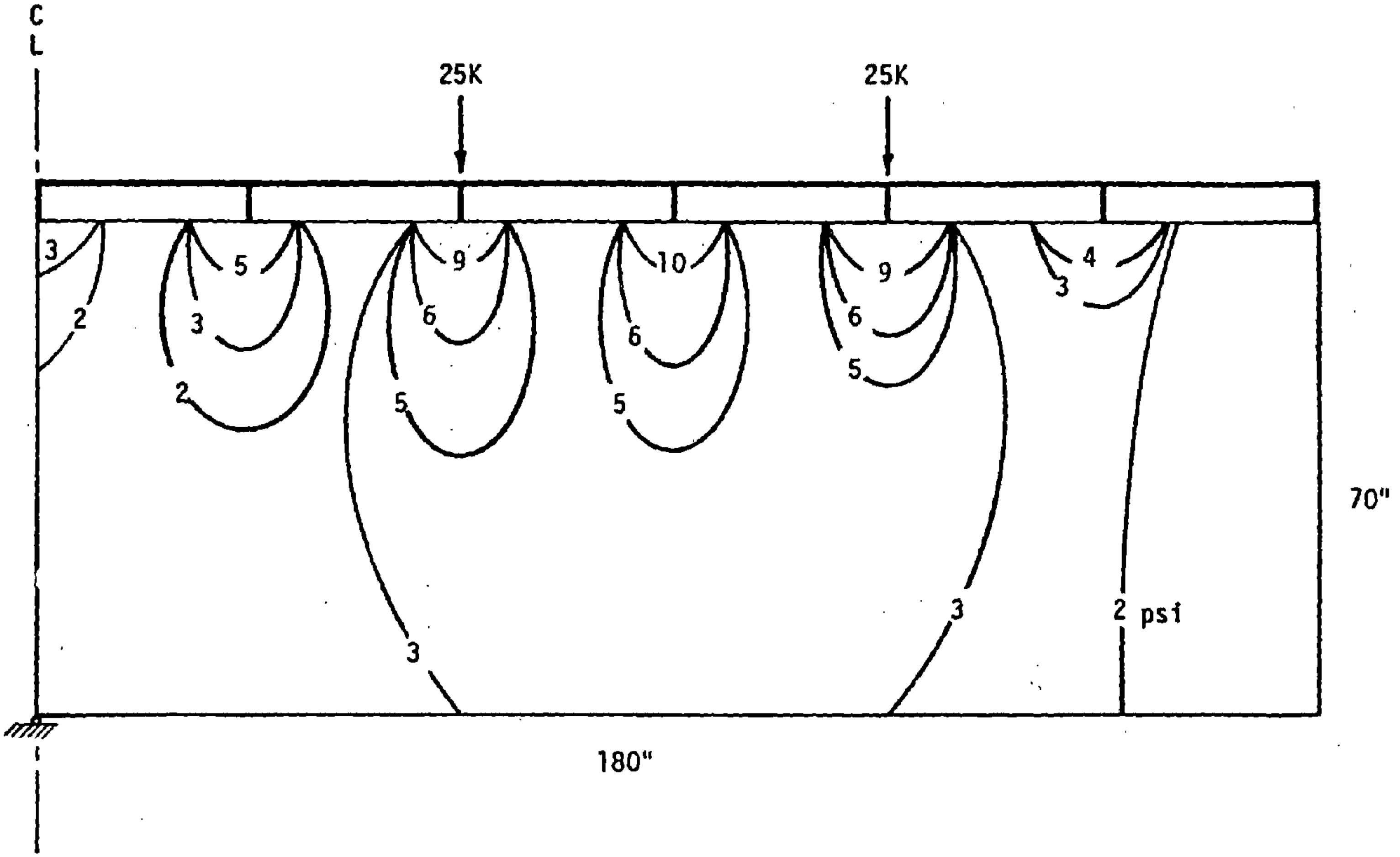


Fig. 4.3-7. Stress contours for refined grid analysis.

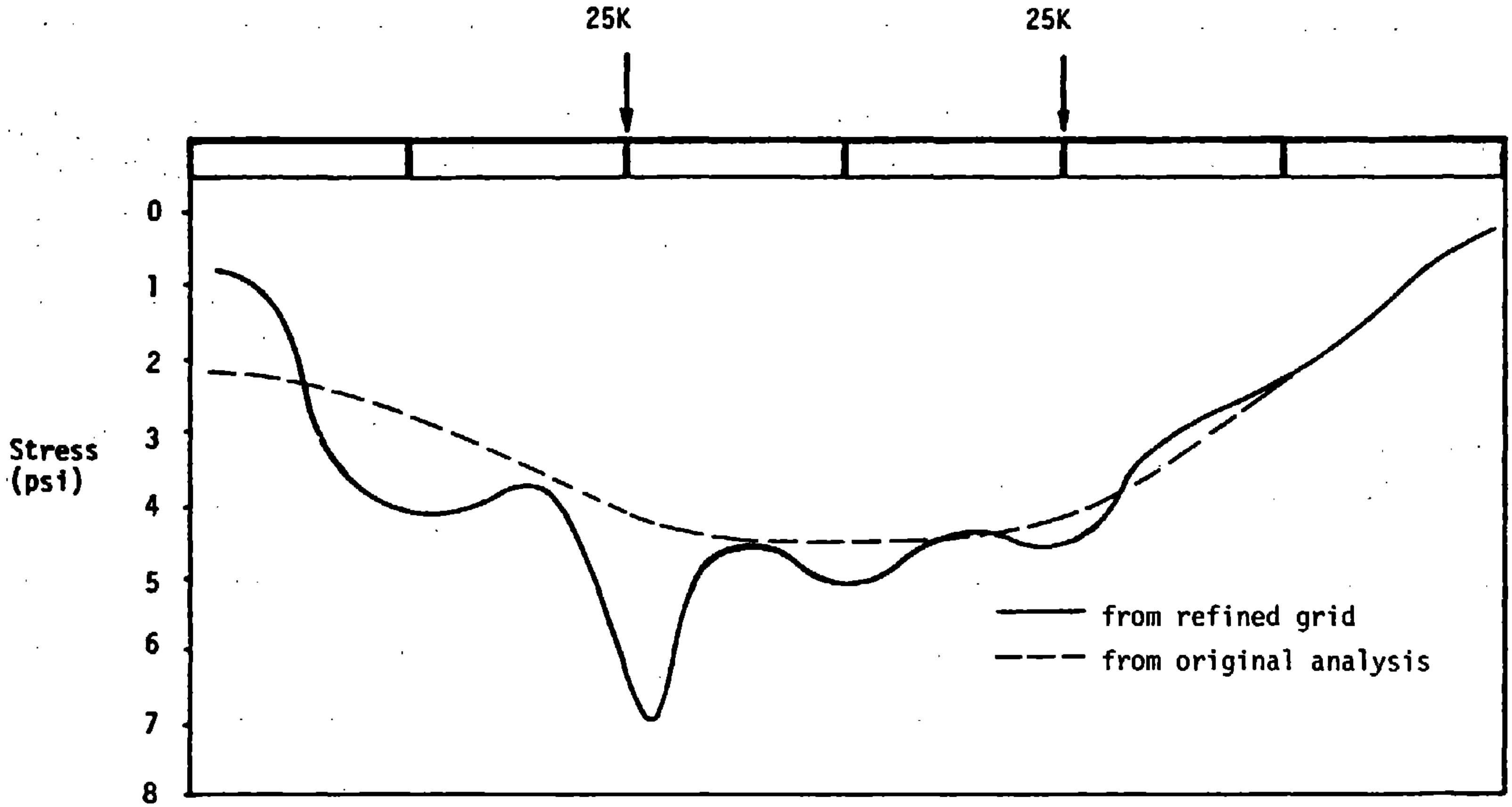


Fig. 4.3-8. Stress at ballast-subgrade interface from refined grid analysis.

pin-points the concentrations which are not shown in the original coarse analysis (dashed line). The conclusion again is that grid refinement produces a more detailed picture of the actual stress state.

The finite element program, RR, thus developed from SSI is a powerful tool in the analysis of any CRTSS. Presently, it can acommodate 300 elements and 300 nodes, having a total of 620 degrees of freedom with a bandwidth of 66. Also, 25 materials, 50 loads, and 75 supported coordinates can be specified. These dimensions require a memory capacity of 58k words in a UNIVAC 1106.

The element library includes the bar, beam, triangular (TRIM 3), and rectangular elements. Both plate elements are plane strain elements. Rectangular elements may be automatically generated by the program, but all other elements require specific manual input. Support conditions on horizontal and vertical boundaries may also be generated automatically.

It is possible to obtain reactions at internal nodes to enable grid refinement analyses. These reactions will be obtained after dead load application and after application of all applied live loads. To enable analysis of a refined grid, these reactions can be input as applied loads at the fine grid simulation boundaries during the dead load and applied load application portion of the analysis.

The output of the program RR also includes: an echo check of the input data, stress data after dead load application, stress state after last load interaction, equilibrium checks, and a summary of the final vertical stresses due to the applied loads.

A listing of program RR and a user's guide are included in the appendix of Lawson's report.

The full capabilities of program RR have yet to be utilized. It is presently capable of performing an overall track structural system analysis and also of refining the analysis to concentrate on zones of higher stress. Although not calibrated against field data, it can be useful in performing parameter studies to determine stress, strain and deformation conditions and trends within the track structural system. Validation and calibration will broaden its horizon to enable the user to perform complete and accurate structural analyses.

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4.4. AN INVESTIGATION OF STRESSES AND DEFLECTIONS IN TRAIN-TRACK SYSTEMS USING THE NONLINEAR FINITE ELEMENT METHOD

by Billy Jack McAlister

[EDITOR'S NOTE: The objective of Mr. McAlister's research was to develop and document a finite element track analysis method that would include the influences of the track structural components and the nonlinear properties of the ballast and subgrade to determine stress and strain distribution in the ballast and subgrade layers due to a static wheel loading. A parallel study was in progress by S. D. Tarabji at the University of Illinois during the same time frame, and it appears that the methods used by each student were very similar although significantly different with respect to the stress-dependent model selected for soil representation and the method developed for the quasi-three-dimensional analysis using successive two-dimensional calculations.]

A two-dimensional finite element approximation of the train-track structure was used, and a plane strain state was assumed to exist in the structure. The finite element idealization will accept values for subgrade and ballast properties which are stress-dependent to best represent the behavior of the actual track structural system being studied. The finite element method transforms the problem of a continuum structure into an assemblage of idealized finite elements which are used to formulate the representative equation of stress, strain, and deformation.

The method of analysis used for the train-track problem proved to work exceedingly well. Initially, the transverse cross-section of the track was simulated and analyzed by the two-dimensional program SSI using a constant thickness for the tie, ballast, and subgrade elements. Based on the stresses developed in the transverse cross-sectional analysis, the thicknesses of the layers of elements in the longitudinal finite element representation of the train-track structure were calculated using a direct inverse ratio. This method of attack was used after it had accurately been shown to determined the distribution of effective stresses in the classical problem of a point load on a semi-infinite half-space. For the railroad problem, it was assumed that the vertical stresses could be safely used instead of the effective stresses that were used in the validation to indicate the thickness variations in longitudinal element layers.

In previous finite element analyses of the track structural system, only a longitudinal section of unit thickness along the vertical centerline of the rail had been considered. That analysis procedure did not consider the fact that the load spreads out with increasing depth into the soil. With the load covering a larger area, the soil pressure at any depth below the surface will be smaller than at the surface. The accepted Boussinesq, Westergaard, and 2:1 slope methods for evaluating soil pressure at various points in a soil stratum demonstrate this fact; therefore, it was felt that a more accurate analysis of the train-track structure could be made if this decreasing pressure phenomenon could be included in the analysis procedure for using the finite element method.

One method of including the decreasing-pressure effect in a soil mass is to increase the thickness of the finite elements in the simulation as the depth from the surface increases.

In summary, the finite element method of analysis is a very powerful procedure for stress analysis that has been used successfully in several industries for many types of structural analysis. In this report, the finite element method has been adapted to the study of the stresses in a track-roadbed structural system. The finite element method can be used to determine the behavior of the track system analyzed for different types of ballast and subgrade and also for various sizes and configurations of track structural components. The stresses calculated from the model differed only slightly from the measured values for stress, indicating that this finite element method of analysis could provide an approximation to the true stress state in a train-track structure.

It was found that the constant stress triangular finite element is very limited. Large numbers of elements would be required to accurately analyze the rapidly changing stress gradient of the track structure with the constant stress triangular finite element. It was therefore recommended that a more sophisticated finite element, such as the rectangular element, be included in SSI before any additional analyses are conducted. The stresses can vary linearily in the rectangular element; therefore, fewer rectangular elements would be necessary for accurate analysis of the stress disribution in the track roadbed structural system.

The simulation of a three-dimensional problem with a two-dimensional solution procedure requires a method of varying the element thickness to represent the effects of the third dimension. A method for accomplishing this was developed during this study.

The vertical stresses in the cross sectional model did not spread out as quickly as expected. This points to the need for the ability to simulate a deeper section of roadbed with better finite elements and for more empirically determined stress data to be able to accurately model the true problem.

Future studies were recommended to determine variations of stresses caused by changes in the structural components of the track system and in the stress dependent material properties of the ballast and subgrade.

4.5 LABORATORY TESTING OF LIME SLURRY PRESSURE INJECTED SOILS

by William C. Greeson

This investigation was essentially a comparison of the soil properties of an assumed LSPI-treated soil with those of a control soil from the area immediately around the injection site.

The injection site was Valley Drive, a subdivision access road located in the southeastern part of Little Rock, Ark. Valley Drive was stabilized by lime slurry injection in July, 1973. Approximately 180 tons of hydrated lime were mixed with water and injected into the roadbed to a depth of 7 feet. The road was not fully developed, and approximately 1800 feet of the stabilized road was not surfaced immediately.

The main purpose of the lime injection treatment of Valley Drive was to stabilize the top soil stratum, or the CL soil layer. One of the reasons for choosing LSPI for the CL layer was that significant soil improvement was noted at an adjacent apartment complex, Huntington Place, after LSPI was used. A report written by McNutt[1] indicated that consolidation parameters were improved by a significant amount in lime-injected cuts for loads of two tons and under.

The soil samples were obtained by the use of a 2.9-inch Shelby tube. Eighteen borings were drilled, and from 6 to 10 samples were taken from each hole. A plan of borings is presented in Figure 3.7-1.

Two basic soils were found in the boring operation. The first, a stiff tan or brown silty clay, was located at the surface and varied in thickness from 2 to 4 feet. The weathered surface soil was of moderate plasticity and was classified as CL, according to the Unified Soil Classification System.

The second type of soil encountered was a stiff gray and red or tan silty clay with a well-defined blocky and slickensided structure. The common name for the soil is "Midway Clay." This stratum extended from immediately below the first stratum to below the boring depth of 10 feet. This highly plastic clay, which was classified as CH under the Unified Soil Classification System, seemed to be over consolidated due to dessication. The blocky structure of the soil showed a distinct change with depth: At 5 feet, the size of the fractured particles was small, about .125 inch in cross-sectional distance; but below 7 or 8 feet, the cross-sectional distances of the blocky particles varied from .25 to .50 inch.

This stiff clay caused a perched water table, which in turn caused the instability in the stiff tan or brown silty clay near the surface.

Only the CH layer of soil was used in the laboratory investigation. No investigation of the CL layer was performed because insufficient undisturbed samples had been obtained.

The samples taken from borings outside of the 30 foot roadway were considered as untreated. In the investigation, it was assumed that lime was injected into all of the soil to a depth of 7 feet. It was possible, however, that some of the samples that were assumed to have been treated were never in contact with the lime slurry due to the lack of penetration in the lower soil layer.

The classification tests performed at the University of Arkansas consisted of the liquid limit test (ASTM-423-66),[2] plastic limit test (ASTM-D424-59),[2] and the grain size analysis test (ASTM-0422-63).[2] The results show that there was little significant change in the classification data due to the LSPI treatment.

A pH comparison test was performed on both treated and untreated soils in their natural pH condition. The pH was determination according to the Eades and Grim[3] "quick test" except that lime was not added to the samples to determine optimum lime content. The tests indicated that, even though lime seams were known to exist, a range of pH from untreated to treated could not be determined after a period of two years. In other research on LSPI, the Louisiana Department of Highways[4] also found that change in pH could not be detected after two years.

Two triaxial tests were performed on both treated and untreated samples. There were four different sets of triaxial data. The two tests were the consolidated-undrained triaxial test with pore pressure measurement and the consolidated-undrained triaxial test without pore pressure measurement. The following conclusions were drawn from the test results:

(1) Average initial moisture content did not vary over 2 percent due to LSPI treatment

(2) Wet and dry unit weights were consistent on treated and untreated soils.

(3) Initial void ratios were consistent on treated and untreated soils.

(4) Porosity was consistent on treated and untreated soils.

(5) Initial saturation was consistent on treated and untreated soils.

(6) The angle of internal friction (both $\bar{\phi}$ and ϕ_{cu}) increased significantly due to the LSPI treatment.

(7) The cohesion (both $\bar{c}$ and c) of the soil was reduced due to the LSPI treatment.

(8) No significant change in strength was shown on the samples tested due to LSPI treatment.

Three sets of consolidation and swell tests were performed: (1) a set taken from an LSPI-treated section of soil, (2) a set taken from an untreated section of soil with a .5 percent (by weight) lime slurry smeared on both top and bottom of the consolidation sample, and (3) a set taken from an untreated section of soil which was used as a control section. In the "smear" test, an attempt was made to simulate what might happen under actual field conditions. The smeared samples wre cured for two weeks in a moisture room with a constant humidity of 100% and a constant temperature of 72°F.

The consolidation tests were performed according to testing procedures of the Corps of Engineers "Laboratory Soils Testing Manual."[5] The swell test consisted of allowing the sample to free swell with a small load increament (.1 psi) immediately before the consolidation test was run. The swelling of the soil usually began after water was placed around the sample. This is in accordance with the free swell test described by Holtz.[6]

The following conclusions were drawn from the consolidation test results:

(1) Average initial moisture content did not vary over 2 percent due to treatment of the soil by LSPI or due to smearing of a consolidation sample.

(2) Initial wet unit weight was consistent on treated, smeared, and untreated samples.

(3) Only the smeared sample showed a lower initial saturation (S_i) percentage. The lower S_i number (92.8 for the smeared sample compared with 96.0 and 96.6 for the untreated and the treated samples, respectively) was probably due to drying of the sample

during the curing period.

(4) The average preconsolidation load was found to decrease slightly due to the injection of lime. The smeared sample showed an increase. The conclusion was that the deviation resulted from dessication of the smeared sample in curing and the graphical procedure from which the preconsolidation load was determined.

(5) The comparison index (C_c) showed a small increase due to the injection of lime and the smearing of lime on a sample (.006 to 0.12). It does not seem reasonable that the injection of lime could cause such an increase if it is assumed that lime increases strength.

(6) The treated and the untreated samples had approximately the same consolidation coefficient (c_v) at both 1.25 tsf and 4.0 tsf. The smeared sample had a c_v value at each load of about six times the treated and the untreated values. Therefore, no substantial results could be formulated.

(7) The percentage free swell showed a decrease due to injecting the lime slurry and smearing the samples. The results show that swell is decreased by one-third due to injection of lime.

(8) Specific gravity was assumed to be 2.8 for all tests. In comparison of the results of many samples on the same soil, this one constant was believed not to influence the results to any great extent.

The laboratory results discussed in this report were obtained from soils which were not the primary target of LSPI stabilization. The soils tested were below the level of primary concern. It is possible that some of the soils which were assumed treated in the comparison analysis were never LSPI stabilized. Reasons include that the injection rods never reached the assumed 7-foot injection depth or that refusal was reached before lime was injected into all cracks, fissures, or root lines. However, since the soil which was the primary target of LSPI injection was not obtainable in the testing program, the soil immediately below the primary target and within the assumed injected zone was tested. The following conclusions were drawn from those samples and research done in the preparation of this report:

(1) Comparison of treated and untreated classification tests showed no significant differences in LL, PL, PI, or grain-size.

(2) The "quick test" procedure for determining the pH of a soil is not a good method for checking for lime after as much as two years time.

(3) A comparison of triaxial test results of treated and untreated soils indicated that cohesion is decreased and the angle of internal friction is increased due to LSPI stabilization. No strength change was indicated in the laboratory analysis; that is, the results were inconclusive.

(4) The compression index determined by the consolidation test showed a small increase due to injection of lime and smearing of lime on an untreated consolidation sample.

(5) The free swell test indication that LSPI will reduce swelling potential.

(6) Smearing samples to simulate field conditions does not compare well with samples taken from injection sites.

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5. SUMMARY AND CONCLUSIONS

This research program was proposed under the University Grants Program, and it was originally contracted under that program. The initial go-ahead for the research activities was given in July, 1974. Later, as the program was expanded, it was transferred to the FRA. All subsequent activities were sponsored and directed by the FRA.

The success of the program is due largely to the cooperation of the railroads and the lime injection contractors. Without the help of these people, the program would not have been possible. The funding and the support of the FRA and the University of Arkansas were also indispensable. Therefore, it was through the joint efforts of many that the work was accomplished.

As planned, the program was partially manned by University of Arkansas graduate students. Their participation had a mutually beneficial effect on both the program and their career development. Four students wrote master's theses while working on the program, which follows the purpose of the University Grants Program under which the work was first proposed.

All objectives of the original proposal were fully met. The research program examined the ability of the LSPI stabilization technique to improve the in-place subgrade soils of problem track roadbeds. It was found that the qualitatively described success of the LSPI method was difficult to prove using quantitative data from routine engineering laboratory and field testing procedures. It was determined that meaningful case studies of track stabilization projects were very difficult to conduct, and it was only at the end of the program--during the Belle Fourche and Rock Island

Phase II case studies--that the research team felt that their experiments were showing data which accurately reflected the field performance of LSPI. The program was concluded with the definite knowledge that the LSPI method can be used to improve unstable and low-strength track roadbeds but that it is only fully effective when properly applied.

The methods for pre-injection engineering investigations are explained in detail in the lime injection handbook. When these methods are incorporated into a railroad's decision process for the utilization of LSPI, they should insure better utilization of the LSPI method.

There are many questions that remain to be answered, however. These questions will require more research-and-development work, which will cost more in both money and time. It is, therefore, important that the progress that has been gained thus far not be lost and that the information in the handbook and the reports be put to practical use. It is only through implementation of these research results that the value of the research program will be established.

While there are several areas of prime interest for future study, the one area of research that most likely would result in big gains for the railroads is a study of the benefits of lime slurry additives. There are known additives which should be analyzed and tested to evaluate their field performance with LSPI.

The results of the environmental study were very encouraging. Previous studies in this area were nonexistent, and it was feared that environmental problems might be discovered which would limit future application of LSPI. Fortunately, this was not the case; however, a few warnings were issued in the handbook regarding potential environmental hazards.

The cost of LSPI was studied with the aid of the data collection, storage, and retrieval system. Reliable figures were developed for predicting the cost of injecting a ton of lime. The cost effectiveness of LSPI, however, is tied mainly to the dollars saved when compared with an alternate system of track improvement, and only scant data was developed in this area. It was learned that this type of information is essentially not publicly available; therefore, future work in this area would be best accomplished by the individual railroad companies.

With few exceptions, the railroads that were making the most use of LSPI in 1974, when this program began, are still using LSPI. Several new users have been added. Most railroads, however, are using LSPI with some reservations, and they are still seeking better information concerning its use. Many improvements have been noted in LSPI equipment and techniques during the past three years, and improved equipment currently is being developed. The biggest breakthrough in the past year has been the development and utilization of new equipment to inject deep slurry projects. One successful project that utilized 40-foot-deep injections over a 900-foot section of track has been conducted. This work was too recent to be included in this study.

The current weak link in the LSPI area is the lack of understanding of the soils engineers and track engineers who are relied upon to make decisions regarding the use of LSPI. It is very difficult for the contractors or the railroads to obtain responsive technical decisions regarding the use of LSPI to stabilize a problem section of track. The factors which affect this are time, money, and established techniques. Before the LSPI method will be used routinely and properly, it will be necessary to allocate the necessary time and money for a thorough investigation, and it will be necessary for the engineers to establish their LSPI expertise.

It is, therefore, recommended that the future research for subsoil stabilization for the railroads continue those areas of study begun during this program and follow new directions in other methods of chemical injection for roadbed stabilization. Even though the railroads are not gaining 100 percent effectiveness from their LSPI projects, the cost is so far below that of most of the alternatives that they have been inclined to specify LSPI stabilization and allow for some unknown percentage of failures. Even when LSPI is not fully effective, it still appears to afford a temporary measure of improvement. It is this uncontrolled success or failure that must be understood to insure that the railroads gain maximum effectiveness from their maintenance dollars.

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☆U.S. GOVERNMENT PRINTING OFFICE: 1978-261-264/129