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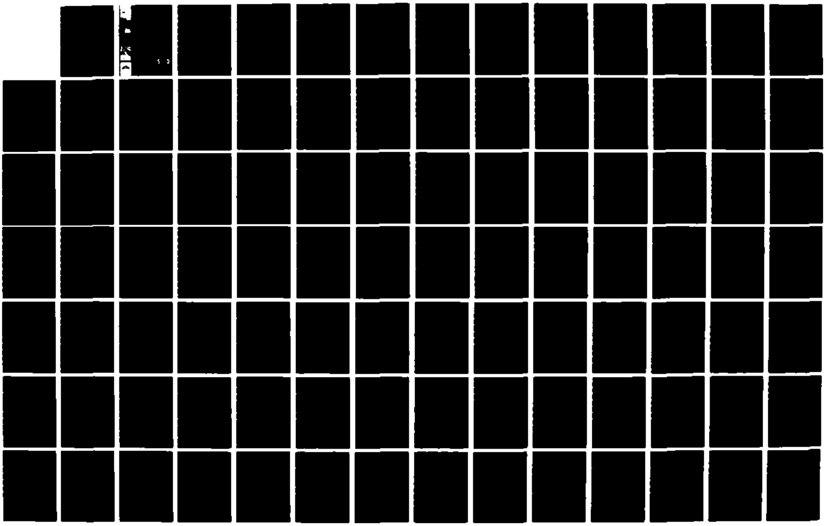

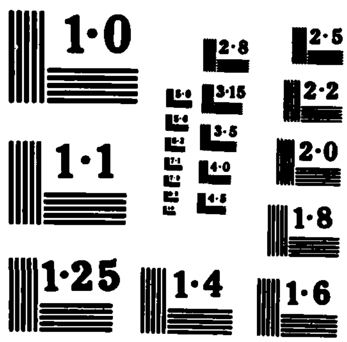

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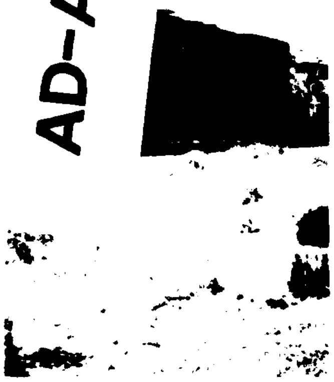

GEOTECHNICAL
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TECHNICAL REPORT GL-85-2

NONDESTRUCTIVE TESTS FOR THE EVALUATION OF RAILROAD TRACK FOUNDATIONS AND LIME SLURRY PRESSURE INJECTION STABILIZATION

by

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This report presents the results of a field evaluation of nondestructive test techniques that could be used for assessing the overall condition of a track support system and for determining the improvement in foundation support derived from lime slurry pressure injection.

A number of test techniques were evaluated on an artificial railroad embankment and level area at the US Army Engineer Waterways Experiment

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20. ABSTRACT (Continued).

Station, Vicksburg, Mississippi, and on two test sections on the Rock Island Railroad west of West Memphis, Arkansas. Twelve different geophysical tools and six penetration and strength tests were employed.

The test sections were injected with lime slurry. However, only the granular materials present at the test sites were injectable and showed significant strength increases.

Based on test sensitivity and repeatability, on the need for geotechnical engineering data, and most importantly on the length of testing time (in order to minimize the impact of testing on normal railroad operations), the following six techniques were selected as best suited for evaluating lime slurry stabilization and overall track support conditions: (a) surface vibratory, (b) surface refraction, (c) mechanical impedance, (d) cone penetration, (e) small aperture CBR, and (f) static or semistatic loading.

PREFACE

This report documents a field evaluation of nondestructive tests for use in the evaluation of lime slurry pressure injection stabilization and of railroad track foundations. The study was conducted by the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, and this report was prepared during the period July 1976 to December 1979 for the Transportation Systems Center (TSC), US Department of Transportation, under Reimbursable Agreement No. 76-41. Technical Monitor for TSC was Mr. Philip Mattson.

The study was conducted under the general supervision of Mr. James P. Sale, former Chief, Geotechnical Laboratory (GL); present Chief of GL is Dr. W. F. Marcuson III. The study was under the direct supervision of Mr. Richard H. Ledbetter, Earthquake Engineering and Geophysics Division (EEGD). Persons actively participating in the study were Messrs. Joseph R. Curro, Jr., Wayne A. Bieganousky, Stafford S. Cooper, and Robert F. Ballard, Jr., EEGD. This report was written by Mr. Ledbetter. Present Chief of EEGD is Dr. Arley G. Franklin.

The Commander and Director of WES at the time of publication of this report was COL Robert C. Lee, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
gallons (US liquid)	3.785412	cubic decimetres
inches	2.54	centimetres
kips (force)	4448.222	newtons
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6894.757	pascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches	6.4516	square centimetres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

1. SUMMARY

1.1 GENERAL

Failure and degradation of railroad track support systems result in operating speed reductions, derailments, and the associated costs of maintenance to prevent such occurrences. New methods are being sought to improve the safety and reliability of railroad operations, among them techniques for assessing the load-bearing capability of track foundations. Particular emphasis has been placed on techniques that are nondestructive to the track structure and that minimize the impact of test activities on day-to-day railroad operations.

In recent years, lime-slurry pressure injection (LSPI) stabilization has become a popular method for upgrading track support systems. It has been used with varying degrees of success; however, the life of the upgraded system under traffic and environmental conditions has not been determined. Therefore, techniques are also needed for assessing the potential for improvement of track foundations by LSPI stabilization and for assessing the life of the upgraded system.

1.2 OBJECTIVE

The objective of this study was to evaluate a number of nondestructive test techniques to determine whether they could be efficiently used to detect foundation material property changes caused by LSPI stabilization or could be used for assessing the overall conditions of a track support system.

1.3 RESULTS

The following geophysical and field penetration and strength test techniques were evaluated:

a. Seismic:
   1. Crosshole shear-wave velocity.
   2. Downhole shear- and compression-wave velocities.
   3. Surface vibratory Rayleigh-wave velocity.

4. Surface refraction shear-wave velocity.
5. Surface refraction compression-wave velocity.

b. Surface resistivity.

c. Radar.

d. Sonar.

e. Nuclear moisture and density.

f. Downhole geophysical logging.

g. Infrared temperature.

h. Dutch cone penetration.

i. Standard penetration.

j. Vane shear.

k. Pressuremeter.

l. California Bearing Ratio.

m. Dynamic stiffness.

n. Mechanical impedance.

o. Static and cyclicly loaded plate.

These test techniques were employed at two controlled lime-injected test sites at the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, and at two lime-injected test sections on the Rock Island Railroad (RIR), west of West Memphis, Arkansas. Laboratory tests on soil samples were also conducted.

Results of this study indicate that the fine-grained clays and silts of the WES and RIR test sites/sections were not injectable in the sense that the lime slurry did not disperse into these soils. However, the more permeable granular materials present at the test sites/sections were injectable and showed significant strength increases. Results also indicate that the injected lime slurry will remain uncured and wet unless the soil is dry enough and of sufficient permeability to absorb the water from the slurry.

1.4 CONCLUSIONS

In using LSPI stabilization, the techniques for injecting lime slurry need to be modified for certain soils, and the soils need to be dry and permeable enough to absorb the slurry water. For nondestructively evaluating LSPI stabilization and the overall conditions of a track support system, the following test techniques were selected based on test sensitivity and repeatability, on the need for geotechnical engineering data, and most importantly on the length of testing time and amount of effort:

a. Surface vibratory Rayleigh wave.

b. Surface refraction compression wave.

c. Mechanical impedance.

d. Dutch cone penetration.

e. Small aperture California Bearing Ratio.

f. Static or semistatic loading.

2. INTRODUCTION

2.1 GENERAL

Failure and degradation of railroad track support systems result in operating speed reductions, derailments, and the associated costs of maintenance to prevent such occurrences. Under the Federal Railroad Administration (FRA) Track Research Program, the Transportation Systems Center (TSC) of the U. S. Department of Transportation (DOT) is pursuing the development of new methods to improve the safety and reliability of railroad operations. One of the key elements of this effort is the development of field evaluation tests and techniques for assessing the adequacy of track substructural support. Development of a data base around the field tests and techniques will lead to the establishment of criteria for assessing roadbed performance and the need for remedial actions.

In recent years, LSPI stabilization has become a popular method for upgrading track support systems. In many cases, the lime injection process has proven to be a valuable remedial measure which can be undertaken with a minimum of downtime. However, there have been instances in which improvements have not been noticeable. Furthermore, the life of the upgraded system under traffic and environmental conditions is not known. Hence, nondestructive field tests and criteria are needed in order to measure changes in subgrade strength and stiffness and life of the upgraded system.

2.2 BACKGROUND

Original plans were to lime slurry inject an actual railroad embankment in the vicinity of Vicksburg, Mississippi, as a test section and to inject a level control test area at WES. In March 1977, a railroad inspection for possible test sections was made. The railroad inspection was conducted with the support of the Illinois Central Gulf Railroad and included approximately 100 miles of track in west central Mississippi.

Soon after this inspection trip, data from tests conducted on the RIR (which was lime slurry injected in 1976) were initially analyzed. The results indicated that changes occurred primarily in the more permeable clay-gravel mixture above the clay subgrade in the two test sections. This behavior implied that nonhigh swelling clays may not be improved by lime slurry injection. Because of this implication, the determination was made by WES and agreed to by TSC that a constructed (controlled) test embankment of lean clay would be better than an actual railroad test section because highly accurate measurements of the preinjection condition could be made on the controlled test embankment. Such a controlled embankment was constructed at WES to provide known conditions and allow careful reevaluation of the potential problem noted in the RIR tests.

A major problem area of this study was the LSPI technique. To fully accomplish the objectives of the study, at least some of the test soils would have to be improved by LSPI. Failure to successfully disperse the lime slurry in injections of the WES test sites in August 1977 led to modification of the technique. Tests were conducted to determine factors which would improve distribution of the lime slurry, and a second attempt at injection was made in January 1978. Injection experience at the WES test sites was very similar to and gave insight to the injection behavior of the RIR test sections. Based on the lime slurry injection results at WES and at the RIR test sections, the FRA lime slurry injection handbook (FRA/ORD-77/30)[1] was extensively revised. The major revisions resulting from this investigation concerned injection techniques and field and laboratory testing to evaluate subgrade soil potential for dispersal of the injected slurry and for improvement of stability.

2.3 PURPOSE

The overall long-range objective of this research effort is to develop tests, data, and guidelines for evaluating the structural suitability of railroad subgrades to perform adequately and safely under specific sets of loading and environmental conditions. In keeping with the above-stated railroad evaluation needs and the overall long-range objective, the immediate objective of this study was to evaluate a number of nondestructive tests to find those that could be efficiently performed to detect subgrade material property changes caused by LSPI stabilization and to assess the overall conditions of railroad embankments.

2.4 SCOPE

Tests were conducted on two controlled test sites at WES and on two test sections on the RIR west of West Memphis, Arkansas. The test sites at WES were a constructed railroad embankment and a level area. Both were injected with lime slurry in accordance with standard commercial practice in August 1977 and were reinjected in accordance with the modified technique in January 1978, as discussed above under Background. The RIR test sections were lime slurry injected in August 1976 as part of an FRA project conducted by the University of Arkansas.[2] Nondestructive preinjection tests and two postinjection tests (1 and 2) were conducted on the WES test sites. For the RIR test sections, preinjection and postinjection tests were conducted in 1976. The RIR was tested again in 1979 to observe the effects of time, traffic, and environmental condition.

3. TESTING TECHNIQUES

3.1 GEOPHYSICAL TECHNIQUES

Determination of in situ mass areal characteristics, as opposed to discrete point characteristics from laboratory testing and field penetration testing, is an important advantage of geophysical techniques. The techniques evaluated in this study are described in detail in Appendix B and were the following:

a. Seismic:

1. Crosshole shear wave (S-wave) velocity (V_s).
2. Downhole vertical shear and compression wave velocities.
3. Surface vibratory Rayleigh wave (R-wave) velocity (V_R).
4. Surface refraction shear wave velocity.
5. Surface refraction compression wave (P-wave) velocity (V_p).

b. Surface resistivity.

c. Radar.

d. Sonar.

e. Nuclear moisture and density.

f. Downhole geophysical logging.

g. Infrared temperature.

In this study, the crosshole shear wave velocity (V_s) results were assumed to be the standard against which to compare results from the other test techniques, because experience has shown this test to be very sensitive and repeatable to less than ± 10 percent.[3-5]

3.2 PENETRATION AND STRENGTH TESTS IN THE FIELD

Various field penetration and loading tests were conducted in order to supplement, verify, and evaluate any trends seen in the results of the geophysical tests and to obtain direct indices of stability. The penetration tests determine indices of strength at discrete points. Appendix B describes these techniques in detail:

a. Dutch cone penetration.

b. Standard penetration.

c. Vane shear.

d. Pressuremeter.

e. California Bearing Ratio (CBR).

f. Dynamic stiffness.

g. Mechanical impedance.

h. Static and cyclicly loaded plate.

3.3 LABORATORY SOIL TESTS

Laboratory soil tests were conducted to verify the findings of tests using the geophysical techniques and the field penetration and strength tests. The following soil parameters were tested for (procedures are described in Appendix B):

a. Atterberg limits.

b. Shear strength parameters.

c. Volumetric change characteristics.

d. Moisture content and density.

e. Elastic (recoverable) and inelastic (nonrecoverable) deformation-load behavior.

f. Chemical reactivity.

g. Lime content (chemical analysis).

4. TEST SITES

4.1 GENERAL

Tests were conducted on two controlled test sites at WES and on two test sections of the RIR west of West Memphis, Arkansas. The test sites at WES were a controlled construction railroad embankment and a level area, both of which were injected with lime slurry in accordance with standard commercial practice in August 1977, and reinjected in accordance with a modified technique in January 1978. The RIR test sections were lime slurry injected in August 1976 under a project conducted by the University of Arkansas.[2] A series of preinjection and two postinjection nondestructive tests (NDT's) was conducted on the WES test sites. For the RIR test sections, preinjection and postinjection NDT's were conducted by WES in 1976, and additional NDT's were undertaken in 1979 after allowing for the cumulative effects of time, traffic, and environmental conditions.

4.2 WES TEST SITES

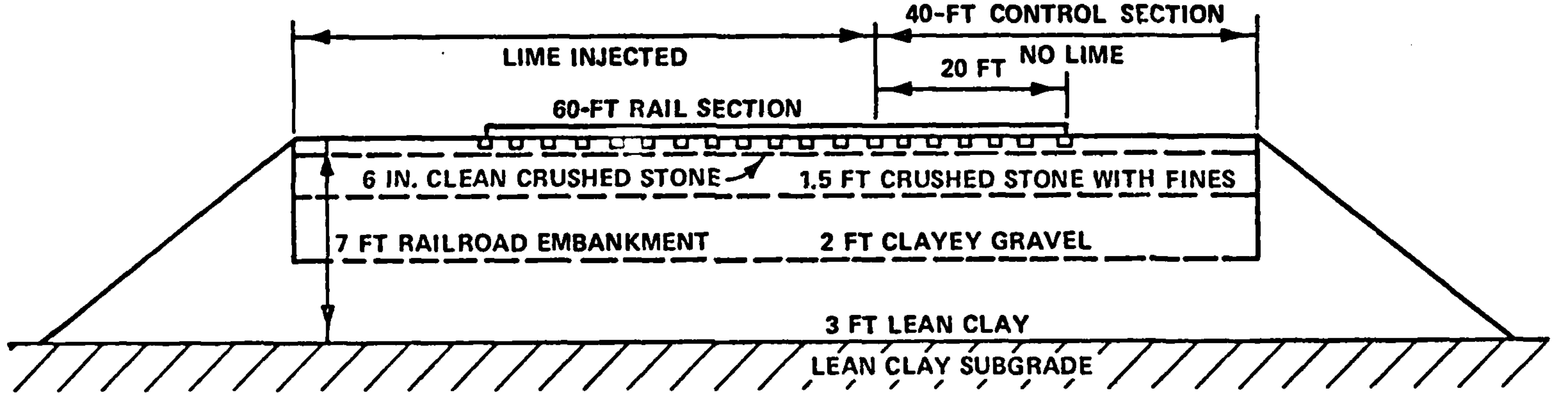
4.2.1 Test Embankment

The test embankment at WES was 100 ft long with a 60-ft section that was lime slurry injected to a depth of 10 ft and a 40-ft noninjected control section (Figure 4-1). The control section was built and tested similarly to the injected section to determine any changes occurring which were not related to the injection of lime slurry. A rail panel was placed for the purpose of conducting tests in a manner and under conditions similar to what would be encountered on a real railroad embankment. In the control section, each material layer was laterally extended to provide an exposed surface layer for control testing and comparisons.

To provide as-constructed conditions for later comparisons, the following tests were conducted on the embankment layers in conjunction with the construction:

a. Seismic refraction.

b. Rayleigh wave.



RAMPS–LEAN CLAY

EMBANKMENT– 6 IN. CLEAN CRUSHED STONE (LAYER 1)
1.5 FT CRUSHED STONE WITH FINES (LAYER 2)
2 FT CLAYEY GRAVEL (LAYER 3)
3 FT LEAN CLAY (LAYER 4)

FIGURE 4-1. LONGITUDINAL CROSS SECTION PARALLEL TO THE RAILS OF THE WES RAILROAD TEST EMBANKMENT

c. Surface shear.

d. Resistivity.

e. Dynamic stiffness.

f. Mechanical impedance.

g. Plate load.

h. CBR.

i. Moisture and density.

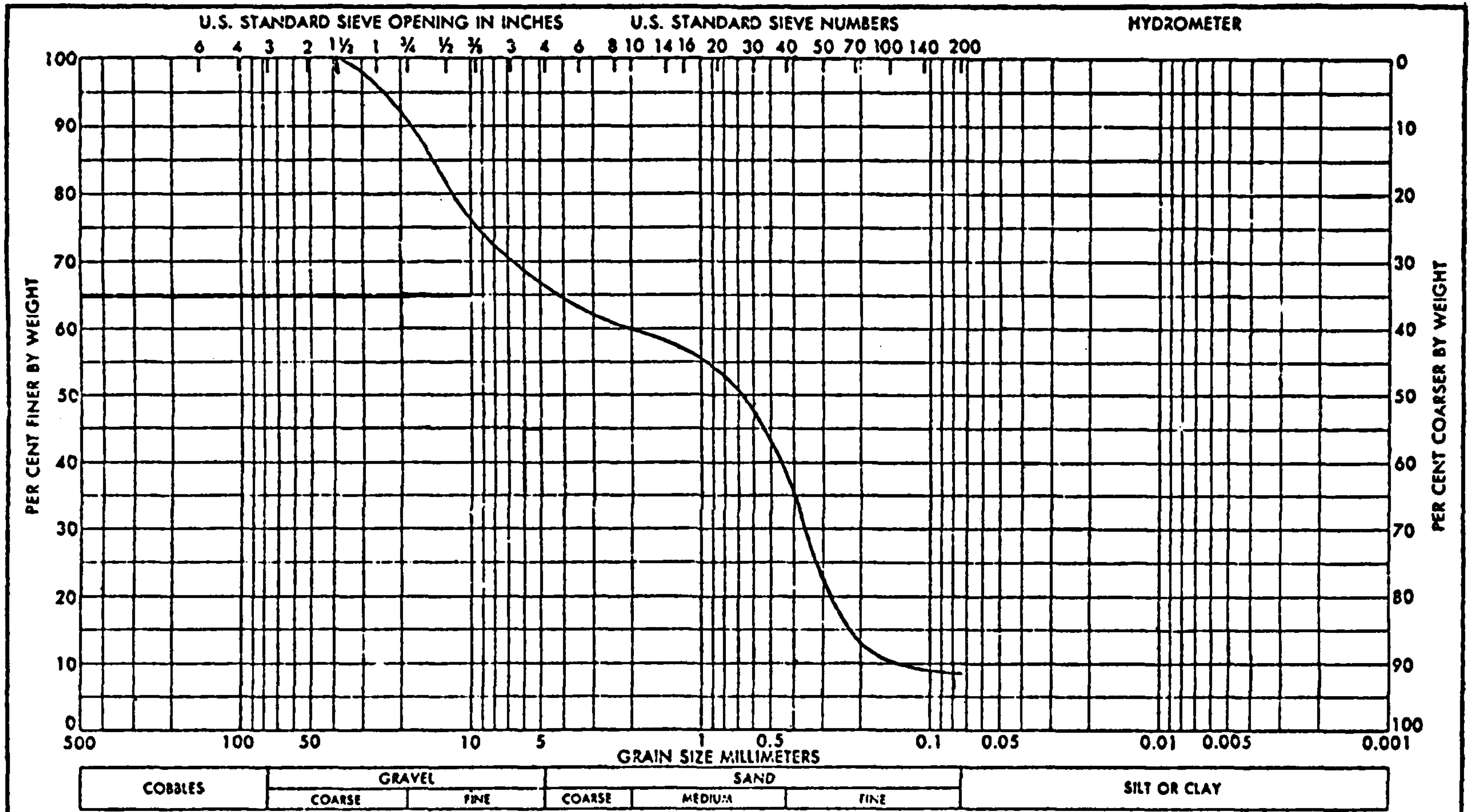
Each material used to construct the embankment and the natural subgrade was chemically reactive to small percentages of lime (approximate pH of 12.4 at 2 percent lime). Design of the test embankment was to simulate an old railroad embankment on a soft clay subgrade as follows:

4.2.1.1 Subgrade - The natural subgrade was a dredge spoil fill area that was placed in 1966 and consisted of a very uniform fine-grained lean silty clay classified CL (according to the Unified Soil Classification System, which is used throughout this report and is detailed in Appendix A). The material has a plasticity index (PI) of 10 to 12, an average water content (WC) of 30 percent, and an average dry density (γ_d) of 94 pcf.

4.2.1.2 Layer 4 - A 3-ft-thick layer of clayey silt (ML-MH, average PI = 19, average WC = 21 percent, average γ_d = 104.2 pcf, average CBR = 6.2) was placed on the subgrade.

4.2.1.3 Layer 3 - A 2-ft-thick layer of clayey gravel material (GC, average PI = 18, average WC = 8 percent, average γ_d = 115 pcf, average CBR = 34) was placed on the clayey silt. A gradation curve for the clayey gravel is shown in Figure 4-2. This layer was intended to simulate the ballast and clayey subgrade-type material mixtures that occur in railroad foundations after years of service and adding ballast.

4.2.1.4 Layer 2 - A 1.5-ft-thick layer of crushed limestone with fines (GM, average WC = 3 percent, average γ_d = 125 pcf) was placed on the clayey gravel in order to simulate degraded ballast that results from repeated traffic



SAMPLE NO	MATERIAL	CLASSIFICATION	NAT W%	LL	PL	PI
		CLAYEY GRAVELLY SAND (SP-SC)		23	12	11

CLASSIFICATION DATA

FIGURE 4-2. GRADATION OF THE CLAYEY GRAVEL FOR THE WES TEST EMBANKMENT loadings and cyclic shear reversals. A gradation curve for the crushed stone with fines is shown in Figure 4-3.

4.2.1.5 Layer 1 - a 0.5-ft-thick limestone No. 4 ballast layer was placed on top of the crushed stone layer in the cribs of the 90-lb rail and tie panel which was sitting on the crushed stone layer.

4.2.2 Level Test Area

In order to provide an LSPI-stabilized test area solely of clayey material where rigorous evaluation tests could be investigated, an area adjacent to the test embankment was chosen. The level test area was in the same dredge spoil fill material as the test embankment subgrade. A 100-ft-long by 30-ft-wide area was designated for tests with a 60-ft portion to be LSPI stabilized (injected to a depth of 10 ft) and a 40-ft noninjected control portion. As mentioned previously for the test embankment subgrade, the silty clay was chemically reactive (pH of 12.4) at 2 percent lime.

4.3 ROCK ISLAND RAILROAD

The RIR test sections[2] were located on the main east-west line between Memphis, Tennessee, and Little Rock, Arkansas. Annual gross tonnage moved over the railroad is approximately 16 million tons. This main line was constructed shortly after the Civil War and crosses the flat alluvial valley region of eastern Arkansas. Much of the railroad was constructed on earth embankments 10 to 15 ft in height. More than 10 miles of track between Briark (approximately 10 miles west of Memphis) and Brinkley, Arkansas, were intermittently lime slurry injected to a depth of 10 ft. The area contained two designated test sections (sites A and B shown in Figure 4-4). Site A started at Mile Post (MP) 21+13 and extended west for 300 ft, with the first 150 ft as a noninjected control area. Site B started at MP 30+22 and extended west for 150 ft. In the 1979 tests, a 150-ft length of the noninjected area west of site B was used as a control area.

A typical profile of the RIR test sections contained the following soil types:

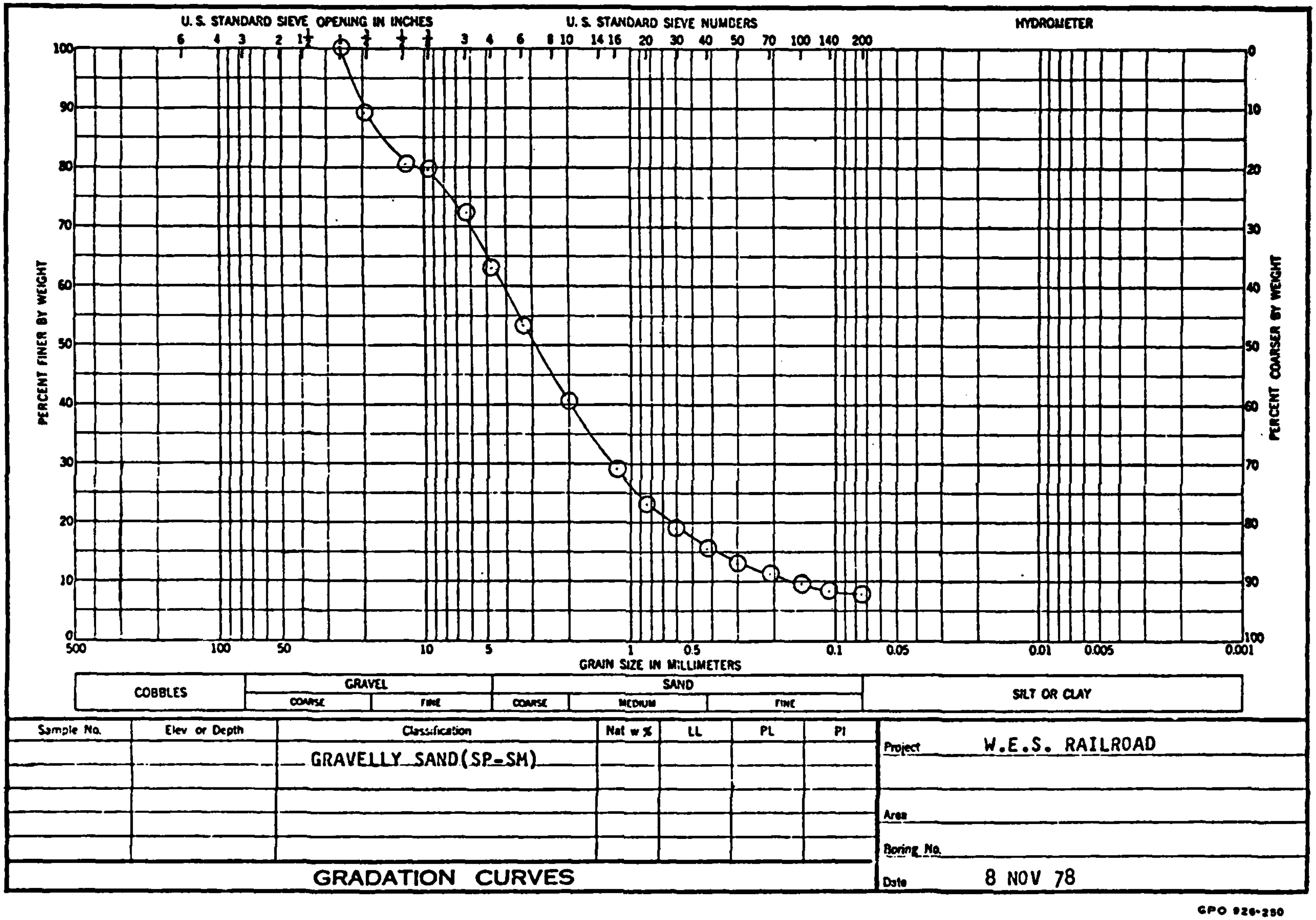


FIGURE 4-3. GRADATION OF THE CRUSHED STONE FOR THE WES TEST EMBANKMENT

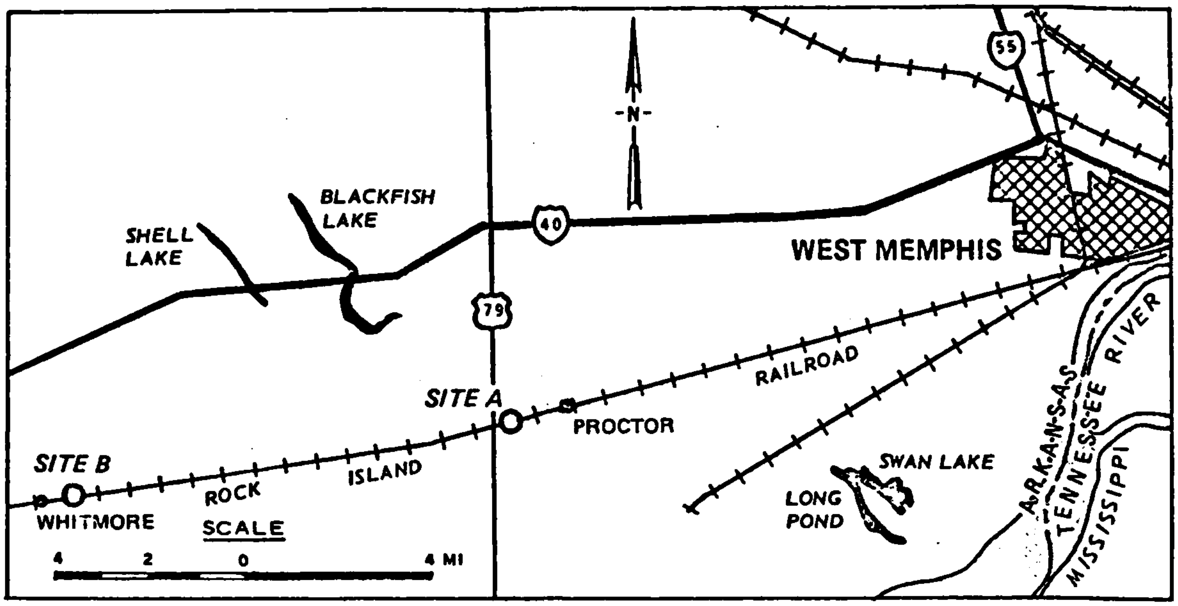


FIGURE 4-4. LOCATIONS OF RIR TEST SITES

a. 0 to 3 ft-GM (old ballast and subgrade mixture).

b. 3 to 7 ft-CL and ML (old ballast and subgrade mixture).

c. 7 to 17 ft-ML, CH, and CL (subgrade).

These materials were chemically reactive to small percentages of lime.[2]

Prior to 1979, sections of the track were experiencing stability problems characterized by the track being out of cross level and alignment due to subsidence. This deterioration was resulting in high maintenance costs and necessitated a reduction in speed limits to 10 to 20 mph. Due to these conditions, the RIR began a program of rehabilitation and upgrading in 1974. Shortly after lime slurry injection (August 1976), the line between Memphis and Little Rock was raised, and the ballast reshaped and tamped. In April 1978, a "resurfacing" program was initiated which was still continuing in June 1979. The track was raised approximately 4 in. with the addition of new ballast, crossties, and 115-lb continuous welded rails. Due to the new track conditions, speed limits were raised to 50 mph in 1979. Several of the injected sections, including the test areas, had not required maintenance since lime slurry injection and resurfacing in 1976.

5. RESULTS OF THE STUDY

5.1 LIME SLURRY INJECTION

5.1.1 WES Test Sites

The WES test sites were injected with lime slurry by Woodbine Corporation, Fort Worth, Texas, in August 1977 and reinjected in January 1978. In both lime slurry injections, every crosstie crib on the embankment and a 2.5-ft grid of points on the flat test area were injected. Injection was performed from the surface in approximate 1-ft intervals to a depth of 10 ft for both test sites.

For the embankment, 2860 and 3000 gal of lime slurry were used in the first and second injections, respectively. In the level test area, 9930 and 4000 gal of lime slurry were used in the first and second injections, respectively. The lime slurry was a 1-to-3 mix with local tap water and was mixed on site at WES. Standard injection techniques were employed, which are continuous, high slurry pressure (pressures between 35 and 50 psi were maintained in this study) and flow even while pushing the injectors to each depth, at which time they are briefly halted.

Fifty days after the first lime slurry injection, dynamic stiffness and crosshole S-wave velocity tests were initially conducted at the sites. The test results showed no change from the preinjection condition. Due to the test results, continuous undisturbed 5-in.-diam soil samples were obtained from the surface to the 12-ft depth in five holes in the level area and in three holes in the embankment. All of the soil samples except for those from two holes in the level area were cut open to visually inspect for lime and photographed. Inspection of the soil showed no significant lime concentrations in the level area and only traces of lime in the crushed stone with fines layer (layer 2) of the embankment. Shallow excavations dug into the side of the embankment showed no significant evidence of lime in the soil materials.

It should be pointed out that prior to soil sampling in the level area, the surface had to be cleared of lime. The surface lime deposit was approximately 3 to 6 in. in depth. The calculated volume of the surface lime deposit was 750 to 1500 cu ft. Lime slurry injection in the level area was approximately 1328 cu ft. Therefore, it appears that almost no lime slurry remained in the ground.

One soil sample in the level area taken at the 8.5- to 11-ft depth had a pocket (about 2 in. in diameter) of wet lime slurry. The presence of the lime slurry pocket indicates that the soil was not dry enough to absorb the water from the slurry, thereby preventing the lime from curing and chemically reacting with the soil.

One possible explanation for the injection results at the WES sites is as follows: For the fine-grained, relatively impermeable, nonfree-draining materials of the sites, continuous high slurry pressures (even while pushing the injectors deeper) forced most of the lime slurry back up the injection holes around the outside of the probes. The technique used was one commonly used in LSPI practice, and it essentially jetted vertical holes of larger diameter than the probes at each injection location.

An alternate injection technique is to turn the slurry flow off before advancing to each depth interval; this tends to allow the soil to seal better around the injection rods than if a continuous flow is maintained. Also, controlling the slurry pressure in a manner that increases it slowly at each depth tends to increase hydraulic fracture and the opening of seams or bedding planes in the soil. When continuous high pressure is maintained, the slurry may only jet a hole along the path of least resistance, which permits the slurry to escape to the surface or to more permeable material such as a ballast layer. Considering that in situ vertical total pressure at a depth of 5 ft is about 4 psi, injection pressures need to be only slightly greater in order to open seams or bedding planes and to fracture the soil. The test sites were reinjected using this above alternate technique with injection pressures no greater than 25 psi. After a curing period of 60 days, 5-in.-diam continuous undisturbed soil samples were taken to approximately 10 ft in depth in two holes in the level test area and one hole on the embankment. The samples were cut open, photographed, sprayed with phenolphthalein, inspected for lime content, and photographed a second time.

Inspection of the embankment samples showed that the complete thickness of the crushed stone with fines layer (layer 2) contained lime slurry; the clayey gravel layer (layer 3) had lime seams at the top and bottom; the clayey silt layer (layer 4) had lime seams and pockets in the top, middle, and bottom; and the subgrade (layer 5) had no lime. The level test area samples showed two lime seams between the 5- and 7.5-ft depths and a large concentration of wet slurry in a 1.5-ft-thick zone between the 7.5- and 10-ft depths. One possible explanation for this concentration is that the soil fractured vertically allowing the slurry to travel to and concentrate in this zone. The uncured lime slurry concentration at the 7.5-ft depth appeared to have churned the soil and created a very weak soil slurry. After the soil sample inspections, the complete suite of field tests and soil sampling was conducted on both test sites.

5.1.2 Rock Island Railroad

The RIR test sections were lime slurry injected to a depth of 10 ft in August 1976 by Roadway Stabilization, located in Eighty-four, Pennsylvania, under contract to the RIR. The University of Arkansas took preinjection and postinjection soil samples,[2] and WES was able to acquire them. All were from the clayey subgrade and none were from the ballast mixture zone. Bag samples of the granular ballast mixture material could not be located. The preinjection and postinjection geophysical tests conducted by WES in 1976 showed that changes occurred in the ballast-subgrade mixture material zone but none occurred in the subgrade material. (These test results will be discussed in the following sections.)

Due to the previously discussed lime slurry injection results at WES and the results of geophysical tests from the RIR test sections, the RIR postinjection subgrade soil samples were cut open and inspected in November 1977. A total of 39 soil samples (26 from site A and 13 from site B) from the subgrade material above the 10-ft depth were cut open, visually inspected, photographed, sprayed with phenolphthalein indicator, and photographed again. The soil samples were representative of the test sites. Only two soil samples had traces of lime slurry (verified by the phenolphthalein): one from site A near the 7-ft depth and one from site B near the 9-ft depth.

The fact that almost no indications of lime slurry were found in the clayey subgrade adds a degree of verification to the postinjection geophysical test results which showed no changes in the clayey subgrade materials (this is discussed later). This fact also supports the lime slurry injection behaviors noted in the injection of the WES test sites.

Injected lime still in slurry form and not solidified or reacting with the surrounding soil was found in the June 1979 tests, approximately 3 years after injection. A lime slurry paste was continuously appearing on the Dutch cone tip and rods in both test sites (A and B). This further emphasizes the finding that the soil must be dry enough to absorb the water from lime slurry before the lime can cure and react with the soil.

In 1979 at sites A and B, lime was visually observed in the ballast-subgrade mixture zone, but none was found in the subgrade materials. The lime content visually observed varied between different locations, which was consistent with the Dutch cone and impedance test result variations within each site. A perched water table was encountered at about the 4-ft depth in site A. The embankment at site A was at least 20 ft above the surrounding area, and the water appeared to be trapped in the zone between the stabilized ballast mixture and nonstabilized top of subgrade.

5.2 LABORATORY AND FIELD TEST RESULTS

5.2.1 General

5.2.1.1 Test Program - For the WES test sites, the complete series of field tests, excluding radar and sonar, was conducted prior to the first lime slurry injection and after the second lime slurry injection. As discussed previously, preliminary dynamic stiffness and crosshole S-wave velocity tests were conducted after the first lime slurry injection. Sonar and radar tests were also conducted after the first lime slurry injection, and radar tests were conducted again after the second lime slurry injection. (As discussed in Appendix B, use of the radar and sonar test equipment was restricted due to its use in conjunction with other projects at WES.)

For the RIR test sections, preinjection and postinjection tests were conducted by WES in the summer of 1976. The postinjection tests were conducted approximately 30 days after injection in August. Geophysical investigations involved use of the following test techniques:

a. Crosshole S-wave and P-wave velocities.

b. Downhole S- and P-wave velocities.

c. Surface refraction P-wave velocities.

d. Surface S-wave velocities.

e. Surface R-wave velocities.

Preinjection and postinjection 3-in.-diam soil samples were obtained in the fine-grained materials below the granular ballast-subgrade mixture by the University of Arkansas. These researchers determined moisture contents and also conducted preinjection and postinjection surface level grids and Dutch cone penetration tests. The Dutch cone penetration test data were acquired only in the fine-grained materials below the ballast-subgrade mixture in site B.[2]

In June 1979, 2 years and 10 months after lime slurry injection of the RIR, WES conducted the following field tests:

a. Mechanical impedance tests through the railroad track railheads.

b. Mechanical impedance tests through a 30-in.-diam plate on the crossties along the track center line.

c. Dutch cone penetration tests.

d. R-wave propagation velocities associated with the impedance tests and additional high-frequency tests (up to 300 Hz for some sites) conducted with a 50-lb vibrator.

e. Surface refraction P-wave tests.

f. Drilling and sampling.

Five-in.-diam undisturbed samples were obtained to a depth of 12 ft. Good undisturbed samples were not acquired in the ballast-subgrade mixture zone because the material was composed primarily of flat-shaped particles that would not allow easy penetration of the sampling tube or movement of the material into the tube. No crosshole S-wave tests were conducted in 1979, because the cased holes (placed in 1976) had been broken and crushed internally by the track rehabilitation work.

5.2.1.2 Analysis of Data - Evaluation of the test techniques was based on method sensitivity to material changes and on repeatability of test results. Sensitivity of a test technique was judged in comparison to material changes shown in the laboratory test results and velocity changes shown by the crosshole S-wave test. (S-wave velocity in soil is indicative of dynamic shear modulus;[3] therefore, since undrained strength and modulus generally change together in soils, changes in S-wave velocity are indicative of dynamic strength changes.) In this study, the crosshole S-wave velocity results were assumed to be the standard against which to compare the other test techniques, because experience has shown the test to be very sensitive and repeatable to less than ±10 percent.[3-5] Repeatability of a technique was judged from results obtained in tests of close proximity, within 1 to 2 ft. Even though the WES embankment was of controlled construction, variability of material conditions occurred along the embankment. In addition to comparing individual test results at a location for sensitivity and repeatability analysis, mean values and standard deviations for an entire test site were compared.

5.2.1.3 WES Level Test Area Site - The full suite of field test techniques detected no significant changes between preinjection and postinjection conditions in the level test area. Also, as discussed previously under Section 5.1, field inspection of undisturbed soil samples showed essentially no lime in the materials. Therefore, figures showing NDT results from the level test area are not believed necessary in this report and are not presented.

Continuous undisturbed soil samples for laboratory use were obtained from three holes 12 ft deep in the injected portion of the test area. Lime contents, moisture contents, Atterberg limits, and classifications were determined in the laboratory from each soil sample. These data also showed that no significant lime content or changes occurred in the soils sampled. The results from the above laboratory tests are presented in Section 5.2.2. Based on the field test results and the above laboratory test results, it was decided that no further laboratory investigations would be conducted.

5.2.2 Laboratory Test Results

5.2.2.1 WES Test Sites - Laboratory test results showed that no significant amounts of lime dispersed into the fine-grained soils of the level test area or into the soils below the crushed stone with fines layer of the test embankment. For the fine-grained soils of both test sites, the material taken from the 5-in.-diam undisturbed samples trimmed to 3 in. in diameter was used for determinations of lime contents, moisture contents, Atterberg limits, and classifications. The soil samples were trimmed to the 3-in.-diam size so they could be preserved for possible triaxial testing. Soil sample lengths ranged from 12 to 18 in., because the sampler tube acquired a sample 30 in. long which was cut and canned in two containers. Trimmings from a soil sample were thoroughly mixed and tested; therefore, the test results were average values representative of the 12- to 18-in.-thick zone from which the sample was obtained. For the crushed stone with fines and clayey gravel materials from the test embankment, tests were conducted on complete 5-in.-diam samples.

Figures 5-1 to 5-3 present for the treated portion of the level test area, the preinjection and postinjection 2 lime content, water content, and Atterberg limits, respectively. The results in Figure 5-1 are presented with connected symbols representative of the thickness of the zone (or height of sample) from which the tested material was obtained. Figure 5-1 shows that negligible amounts (0 to 0.6 percent) of lime were injected, a result qualitatively verified by the field sample inspections discussed previously. The chemical reactivity test indicated that a lime content increase of 1.5 to 2 percent was necessary for reaction in the soil. As seen in Figure 5-2, the water contents had negligible variations. Figure 5-3 presents the Atterberg limits and plasticity index values at the midpoint of each zone indicated on Figure 5-1. No significant changes occurred in the limits or index values. Normally, lime-treated reactive soils have significant decrease in plasticity index.

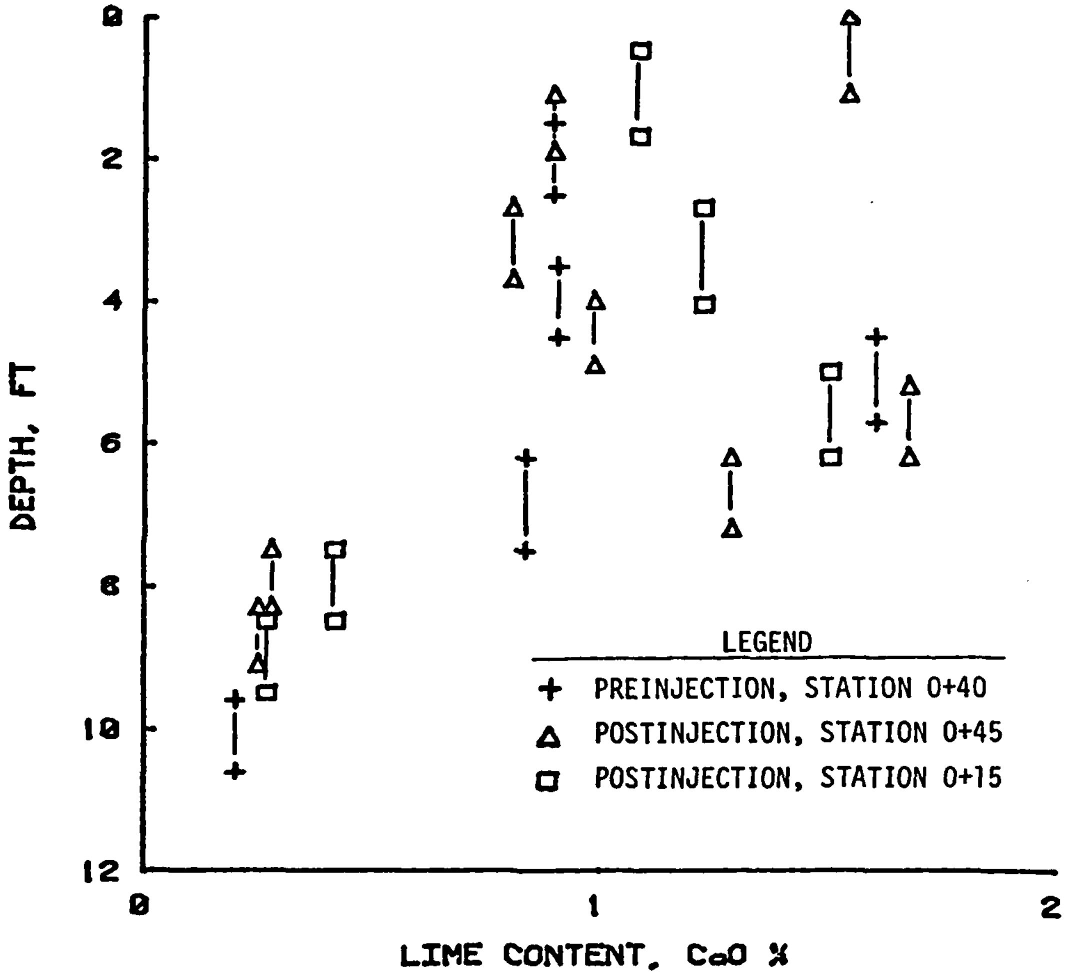


FIGURE 5-1. PREINJECTION AND POSTINJECTION LIME CONTENT FOR THE LEVEL TEST AREA AT WES

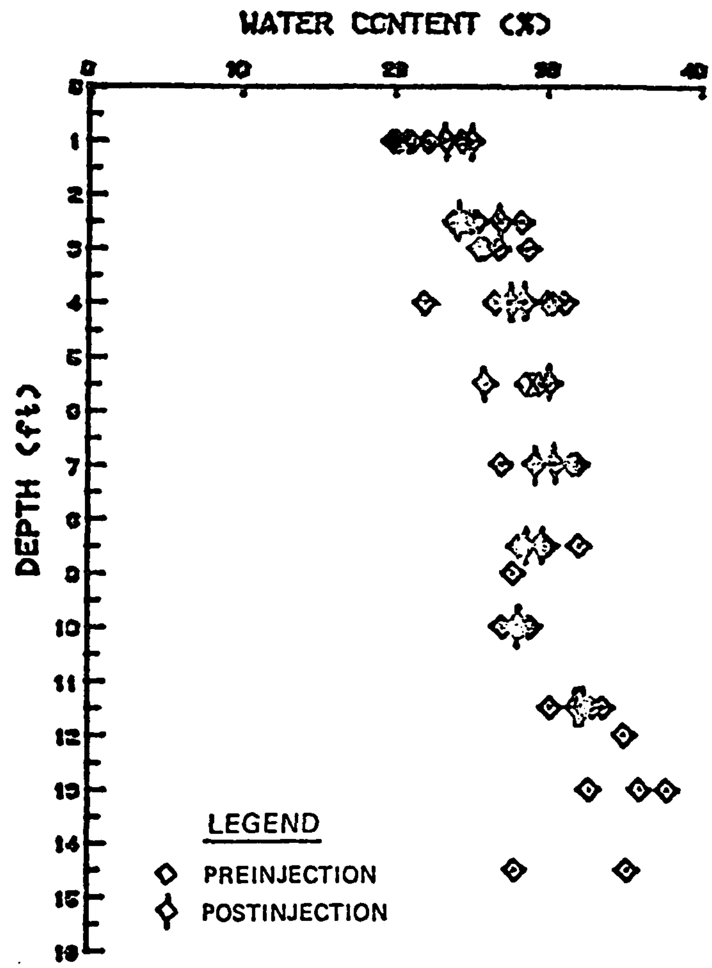


FIGURE 5-2. PREINJECTION AND POSTINJECTION WATER CONTENT FROM STANDARD PENETRATION TEST SAMPLES IN THE INJECTION PORTION OF THE LEVEL TEST AREA AT WES

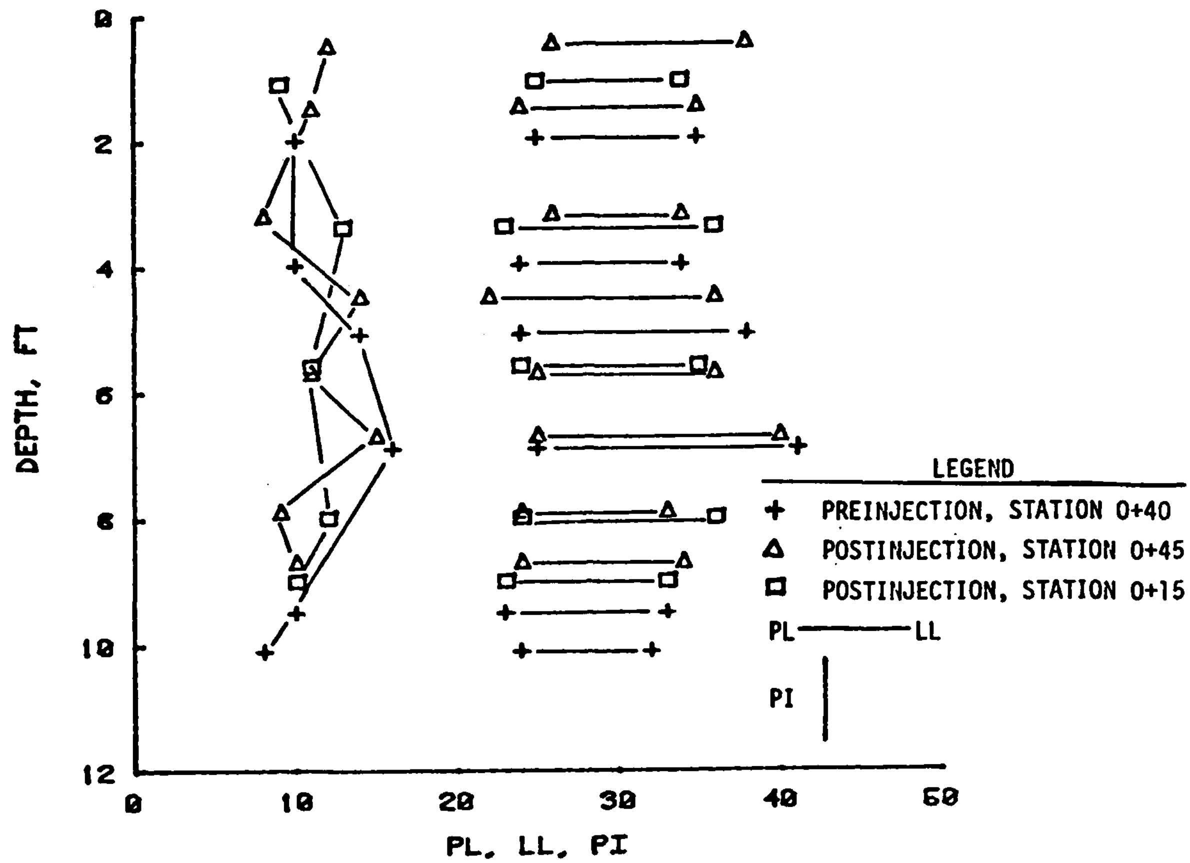


FIGURE 5-3. PREINJECTION AND POSTINJECTION ATTERBERG LIMITS AND PLASTICITY INDEX VALUES IN THE LEVEL TEST AREA AT WES

Figures 5-4 to 5-6 present the test embankment preinjection and post-injection 2 lime content, water content, and Atterberg limits, respectively. The results in Figure 5-4 are presented as in Figure 5-1. The clayey gravel, clayey silt, and lean clay materials all required a 1.5 to 2.0 percent increase in lime content for chemical reactivity to occur. Only a small percentage of lime and water was required for chemical reaction in the crushed limestone with fines layer. As shown in Figure 5-4, only the crushed stone with fines and the constructed clayey silt layers had a sufficient average increase in lime for reaction (up to 4.5 percent increase in the clayey silt in the vicinity of station 0+45). The data in Figure 5-4 are consistent with those obtained in the field sample inspections discussed previously, which showed that the clayey silt layer had approximately three lime seams (top, middle, and bottom) but did not show good lime slurry impregnation around the seams. Lime seams were found at the interfaces of the different material layers, but there was

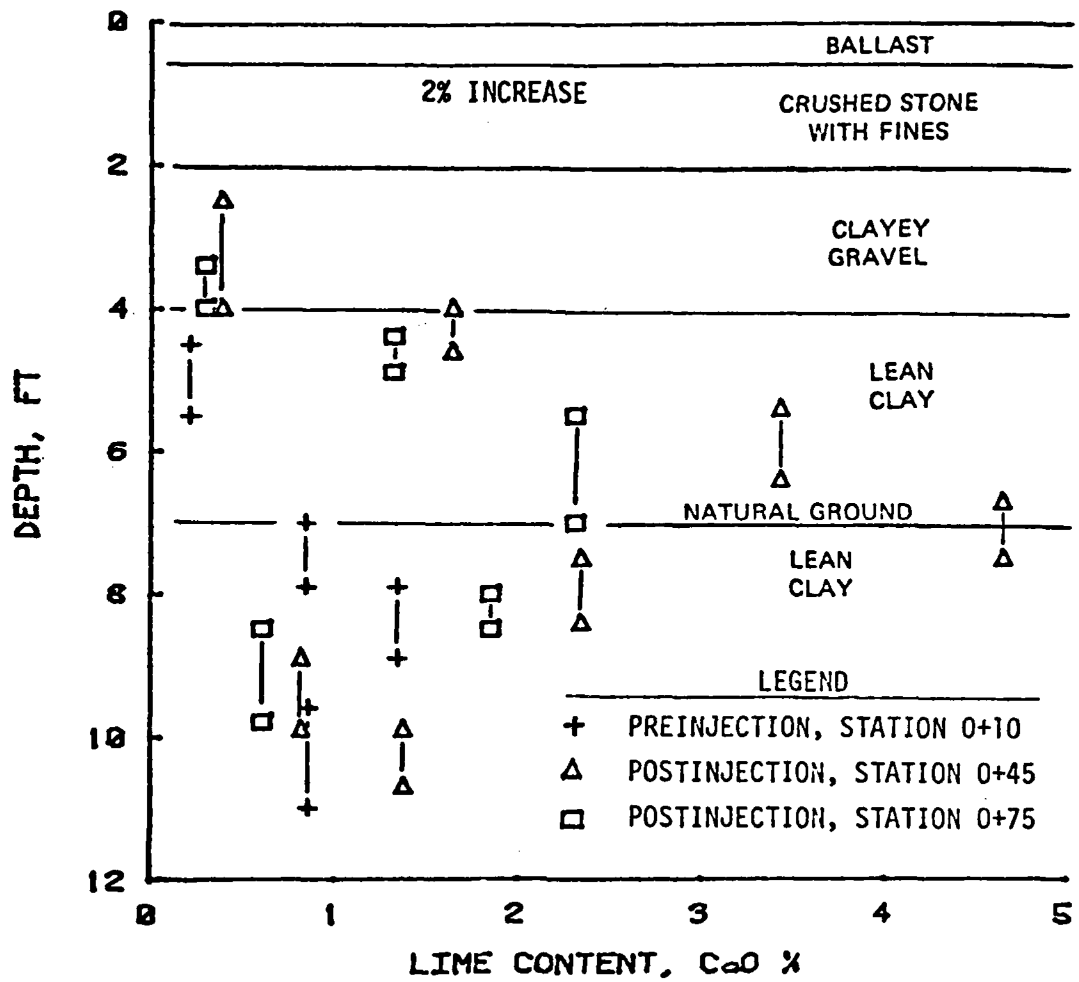


FIGURE 5-4. PREINJECTION AND POSTINJECTION LIME CONTENT FOR THE WES TEST EMBANKMENT

negligible impregnation of the surrounding materials except in the crushed stone with fines which was saturated with lime slurry.

Figure 5-5 shows a large increase in water content in the constructed clayey silt layer and only slight differences in the other materials. The Atterberg limits increased in the clayey silt layer, as shown in Figure 5-6, but the plasticity index did not change.

The Dutch cone penetration test results for the test embankment (presented in the following sections) showed that the clayey silt layer actually lost strength in the vicinity of station 0+45. This strength loss is attributed to

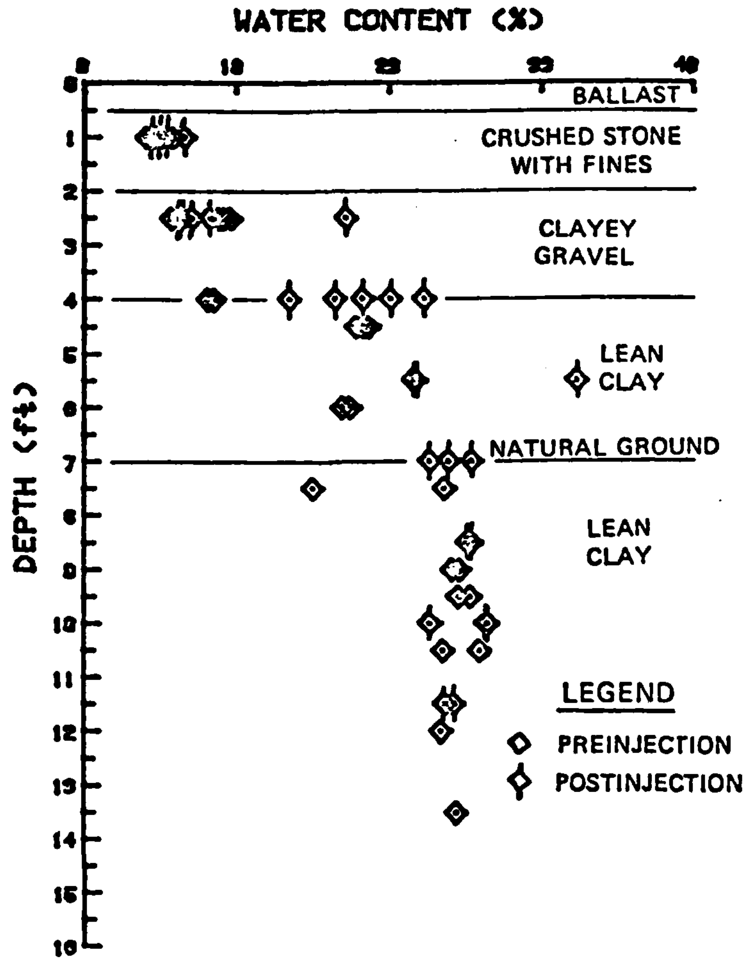


FIGURE 5-5. PREINJECTION AND POSTINJECTION WATER CONTENT FROM STANDARD PENETRATION TEST SAMPLES IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

the large increase in water content. Based on the laboratory test results for the above embankment material and on the NDT results in the following section, no further laboratory tests were deemed necessary on material below the crushed stone with fines layer.

For the crushed stone with fines layer of the embankment, the pertinent laboratory findings in comparing preinjection and postinjection results were as follows:

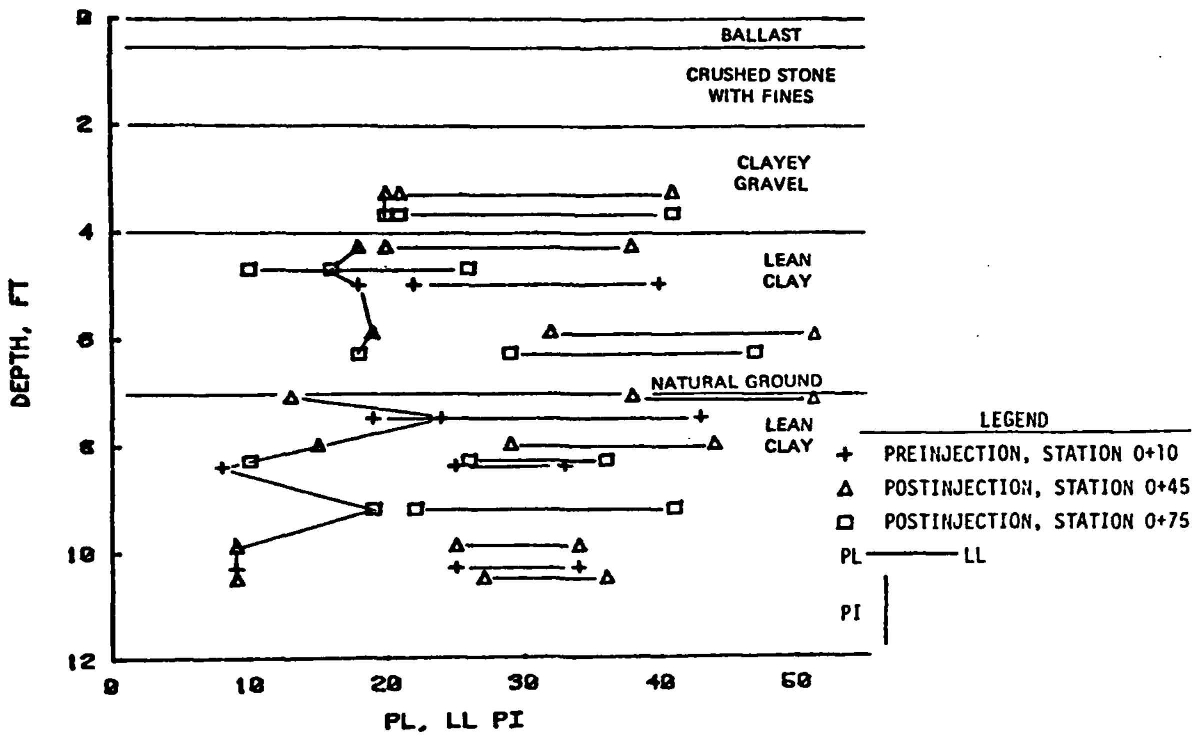


FIGURE 5-6. PREINJECTION AND POSTINJECTION ATTERBERG LIMITS AND PLASTICITY INDEX VALUES FOR THE WES TEST EMBANKMENT

a. No density change occurred.

b. Shear strength increased as evidenced by the average internal friction angle increasing from 29.5 to 37.3 degrees (26 percent increase).

c. Laboratory drained triaxial test compression characteristics[6] revealed the following:

1. Total deformation response (elastic plus inelastic volumetric strain) under load reduced an average of 42 percent as mean principal stress increased from 10 to 60 psi.

2. The unload deformation response (elastic rebound volumetric strain) reduced an average of 37 percent as mean principal stress decreased from 60 to 10 psi. The portions of the unload deformation versus load response preinjection and postinjection curves were parallel in the load range of the large field vibrator with the postinjection curve showing less deformation response.

3. Total deformation response under reloading reduced an average of 52 percent as mean principal stress increased from 10 to 30 psi.

4. The inelastic deformation (load minus unload volumetric strain) reduced an average of 43 percent in the mean principal stress range between 10 and 60 psi.

5.2.2.2 RIR Test Sections - As discussed previously in Section 5.1.2, inspection of samples from the fine-grained soils below the ballast-subgrade mixture showed no significant amounts of lime. Preinjection and postinjection water contents were determined from undisturbed soil samples by the University of Arkansas in 1976[2] and by the WES in 1979. Figure 5-7 shows typical water content results. As can be seen, no significant changes occurred. Undisturbed

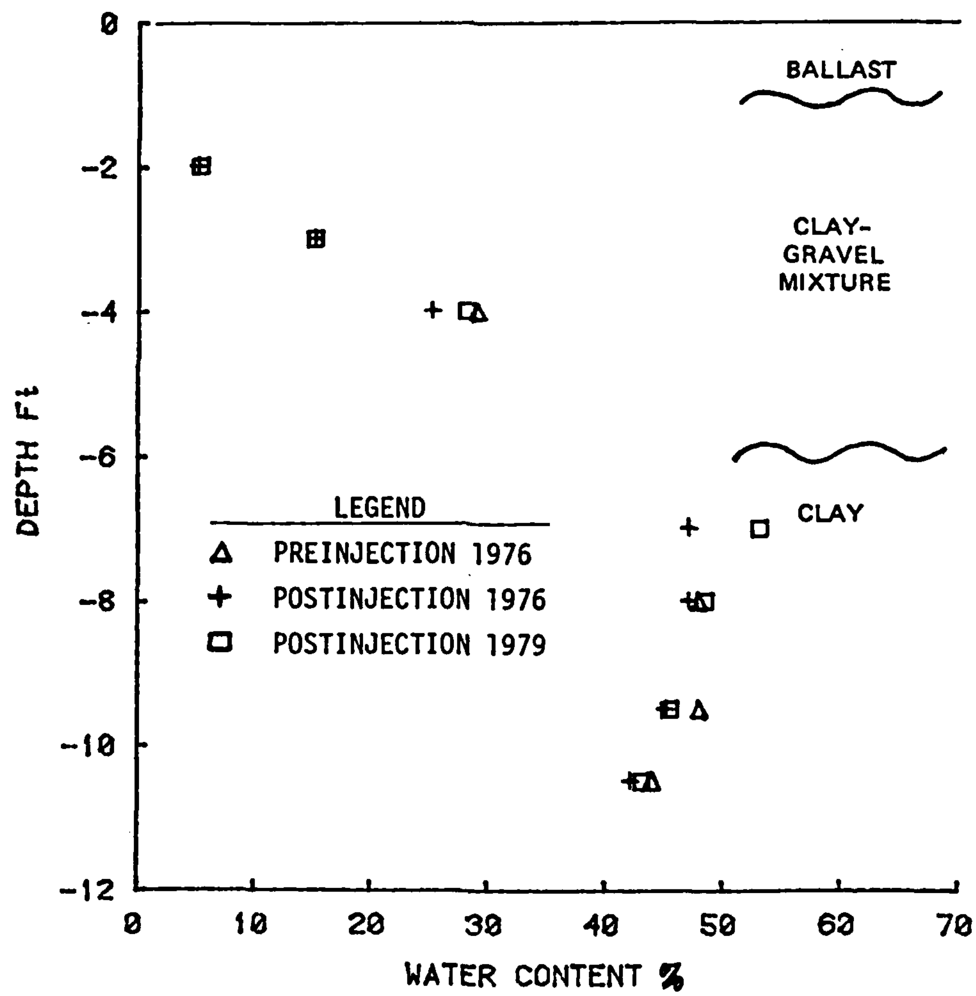


FIGURE 5-7. TYPICAL PREINJECTION AND POSTINJECTION WATER CONTENT FOR THE RIR TEST SECTIONS

soil samples could not be obtained in the granular materials, but visual inspection of boreholes in 1979 showed significant amounts of lime in these materials. Remolded samples of the granular materials would not have been representative; therefore, further laboratory tests were not conducted.

5.2.3 Field Test Results

With the exception of the constructed lean clay layer (layer 4) in the vicinity of station 0+45, laboratory test results showed that no changes occurred and essentially no lime dispersed into the WES test embankment materials below the crushed stone with fines layer. Therefore, the field test results should show no changes below the crushed stone with fines, except in the vicinity of station 0+45, and any indicated change would reflect on the repeatability of the particular test technique. For the crushed stone with fines layer (layer 2), the laboratory findings showed increases in lime, moisture, and apparent angle of internal friction, and significant reduced deformation (elastic and inelastic) behavior under load. Therefore, the field test results should show significant changes in the crushed stone with fines layer. The field tests should also show a change in layer 4 near station 0+45.

Soil sample inspections and laboratory tests showed that no lime dispersed into the fine-grained materials of the RIR test sections. Undisturbed samples could not be obtained in the granular materials, but visual inspection showed significant lime concentrations in these materials. Therefore, the field test results should show changes in the granular materials and no changes below them.

Table 5-1 summarizes the field test results. The following paragraphs present typical test results from the WES test embankment and from the RIR test sections.

5.2.3.1 Crosshole Wave Velocity - Figure 5-8 shows typical results for the WES embankment. The S-wave velocity increased an average of 34 percent in the crushed stone with fines material. However, the control portion of the embankment had an average S-wave velocity increase of 24 percent in this layer, which is attributed to the natural cementing action at the surface of

TABLE 5-1. SUMMARY OF FIELD TEST RESULTS

TEST TECHNIQUE	TEST RESULTS: WES TEST EMBANKMENT	TEST RESULTS: RIR TEST SECTION	EVALUATION COMMENTS
Crosshole S-wave	V_s increase = 34% in layer 2. Repeatability ± 10%.	V_s increase in upper granular material in 1976. No data in 1979; cased holes crushed	Good test technique. Repeatability good. Considered as the standard for other test comparisons
Surface vibrator R-wave	V_R increased. Test data more variable and masked small changes or differences	V_R increased. S-wave velocities approximately equal in 1976 and 1979, which implies the stabilized zone had not deteriorated	Test data more variable than the crosshole S-wave results
Surface refraction P-wave	V_p in bottom 1 ft of layer 2 increased an average of 59%	V_p in upper granular material increased an average of 31%	When velocity decreases with depth, test is only good for top material
Surface electrical resistivity	Resistivity decreased in layer 2, which indicates increase in moisture or improved electrical conductivity	Not conducted	Only shows electrical changes. Does not indicate change in strength or stability
Dutch cone penetration	Point bearing increased in layer 2	Point bearing increased in top granular material	Test sensitivity and repeatability are good
Standard penetration test	Did not indicate the degree of change shown by other test techniques	Not conducted	Test not as sensitive to changes or differences as Dutch cone
Plate cyclic load	Static load carrying capacity increased	Not conducted	Test data were reduced to total, elastic, and inelastic vertical deformations. Sensitivity and repeatability are good

(Continued)

(Sheet 1 of 3)

TABLE 5-1. (CONTINUED)

TEST TECHNIQUE	TEST RESULTS: WES TEST EMBANKMENT	TEST RESULTS: RIR TEST SECTION	EVALUATION COMMENTS
Dynamic stiffness	No significant difference occurred in preinjection and post 2 injection data because the load deformation response curves were parallel in the vibratory load range, as shown by laboratory tests	Dynamic strength increase occurred	Test data were derived at 20 Hz from the mechanical impedance results. Sensitivity and repeatability are good
Mechanical impedance	Because mechanical impedance testing is new to geotechnical investigation and the possible wealth of information contained in the test results is, at the present state of the art, not fully developed, only limited use of the results was made in this study. Test results and a hypothesis for interpreting them are presented in Appendix C. Tests were conducted on both the WES and the RIR test sites		
Vane shear	No data in stabilized zone	Not conducted	The device starts testing at a 6-ft depth which was below the stabilized zone. Repeatability good in fine-grained materials
Infrared survey	No thermal radiation differences due to lime slurry injection	Not conducted	
Downhole seismic	Data not good	Data not good	Good quality signals could not be generated in granular ballast
CBR	Postinjection test not conducted in the stabilized granular material	Not conducted	Not highly accurate in granular materials
Surface S-wave	Data not good	Data not good	Sufficient coupling for seismic source and receivers is difficult to achieve in granular ballast

(Continued)

(Sheet 2 of 3)

TABLE 5-1. (CONTINUED)

TEST TECHNIQUE	TEST RESULTS: WES TEST EMBANKMENT	TEST RESULTS: RIR TEST SECTION	EVALUATION COMMENTS
Ground penetrating radar	The privately developed unit could not penetrate due to moisture. The commercial unit showed no difference between stabilized and nonstabilized areas	Not conducted	Without the known profile, interpretation of layers was questionable
Sonar	The test results showed no major difference between stabilized and nonstabilized areas	Not conducted	Without the known profile, interpretation of layers was questionable
Downhole geophysical logging	Data not good	Not conducted	Impractical for shallow boreholes and thin layers due to tool length and speed of operation
Pressuremeter	Data not good	Not conducted	Augering of holes and sloughing of granular material made the holes impractical for testing. The soft fine-grained materials did not provide enough lateral resistance for testing

(Sheet 3 of 3)

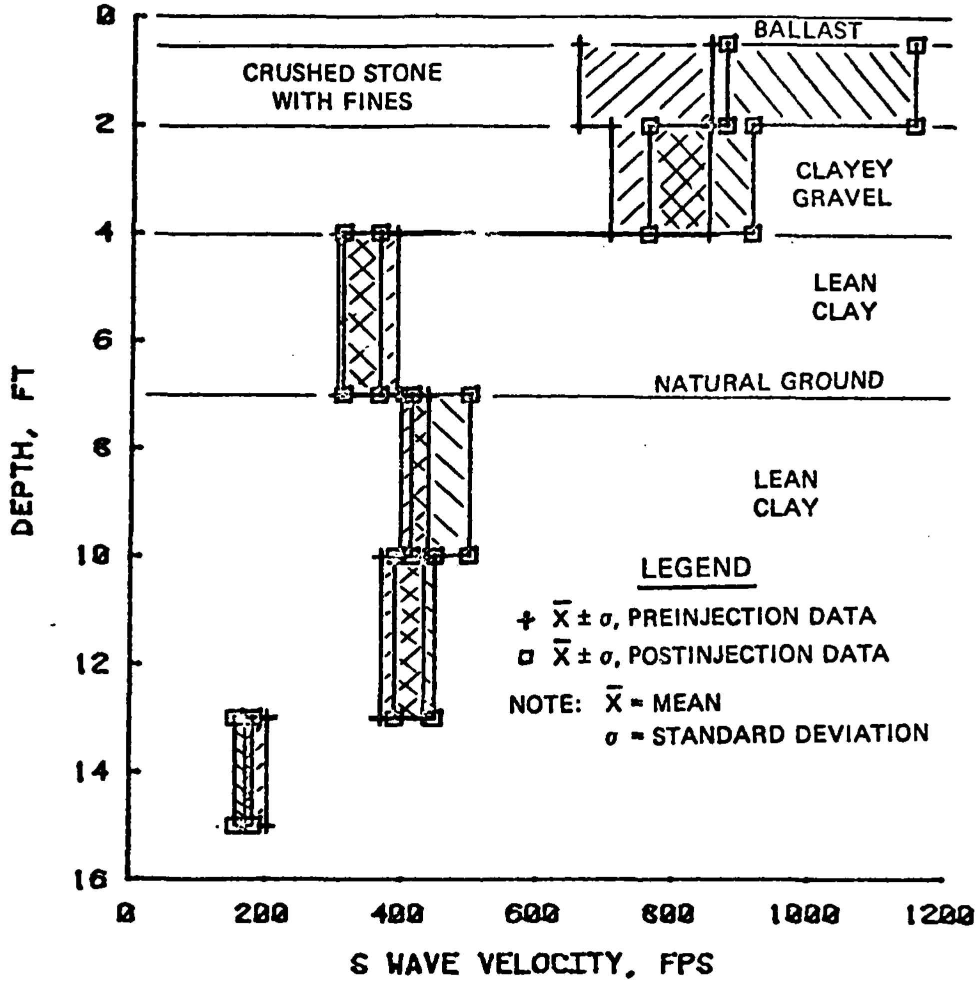


FIGURE 5-8. TYPICAL PREINJECTION AND POSTINJECTION CROSSHOLE SHEAR WAVE VELOCITIES IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

limestone particles. This cementing of the limestone particles was induced by the environment over the elapsed time between test periods (approximately 1 year). (Cementing of limestone particles also occurs in degraded ballast in actual railroad embankments.) No density change was measured in the laboratory samples which would indicate density increase as the source of the velocity increase. The range of S-wave velocities in the crushed stone with fines material shown in Figure 5-8 is significantly greater than that in the control portion of the embankment, which indicates significant strength increase due to the injected lime slurry.

Figure 5-9 shows typical results for the RIR test sections in 1976. As can be seen, the S-wave velocity increase is in the upper portion of the

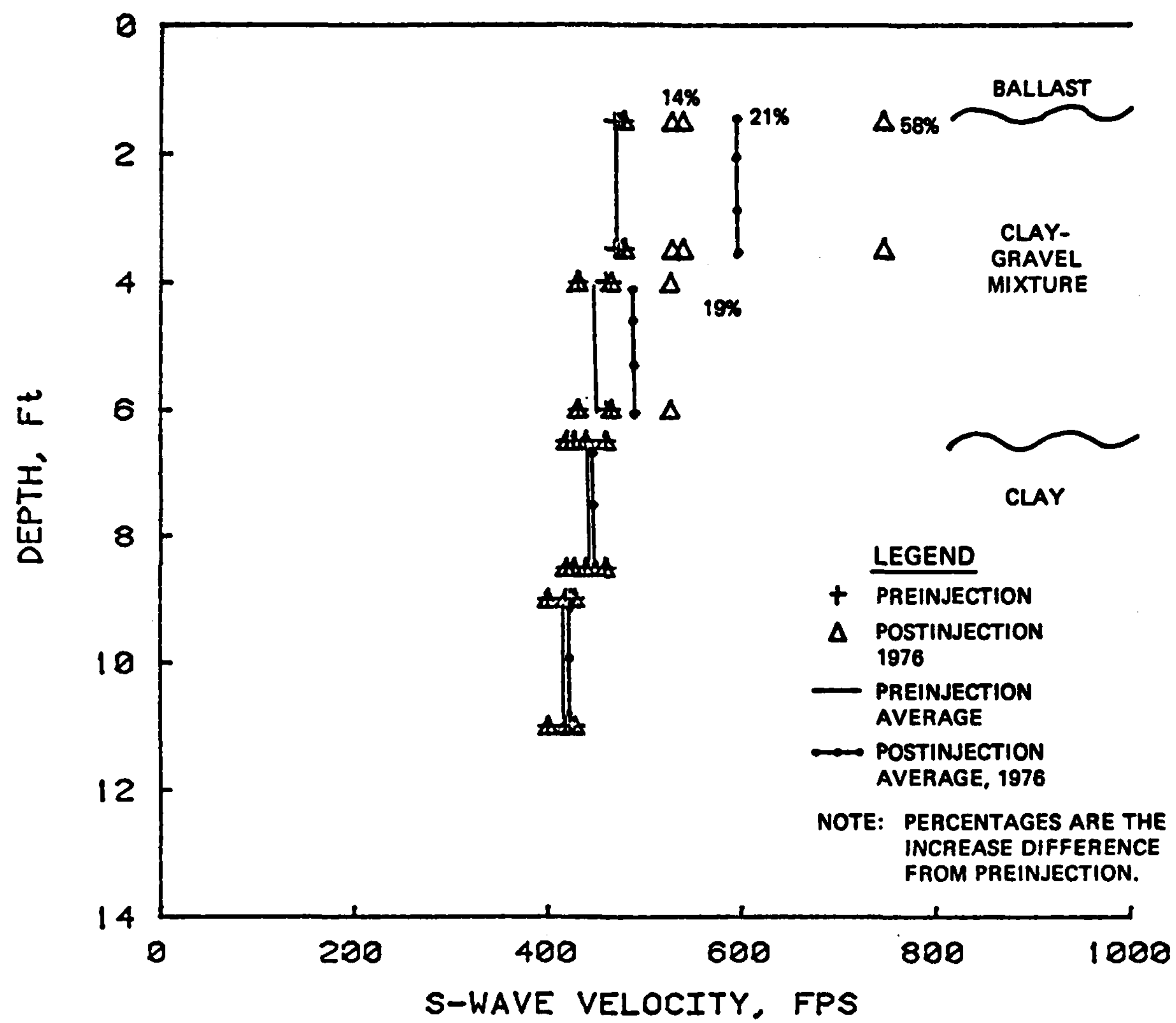


FIGURE 5-9. TYPICAL PREINJECTION AND POSTINJECTION CROSSHOLE SHEAR WAVE VELOCITIES IN THE RIR TEST SECTIONS, 1976

clay-gravel mixture. No crosshole S-wave tests were conducted on the RIR in 1979 because the cased holes had been broken and crushed internally by track rehabilitation work. The average increase in S-wave velocity indicates a doubling of the small stress-strain dynamic shear modulus, assuming density remained the same.

5.2.3.2 Surface Vibratory R-Wave Velocity - Figure 5-10 shows typical results for the WES embankment. A comparison of the R-wave and S-wave velocities of Figure 5-8 shows that the R-wave velocities exhibit greater variability and consequently mask small changes that occur in the material properties.

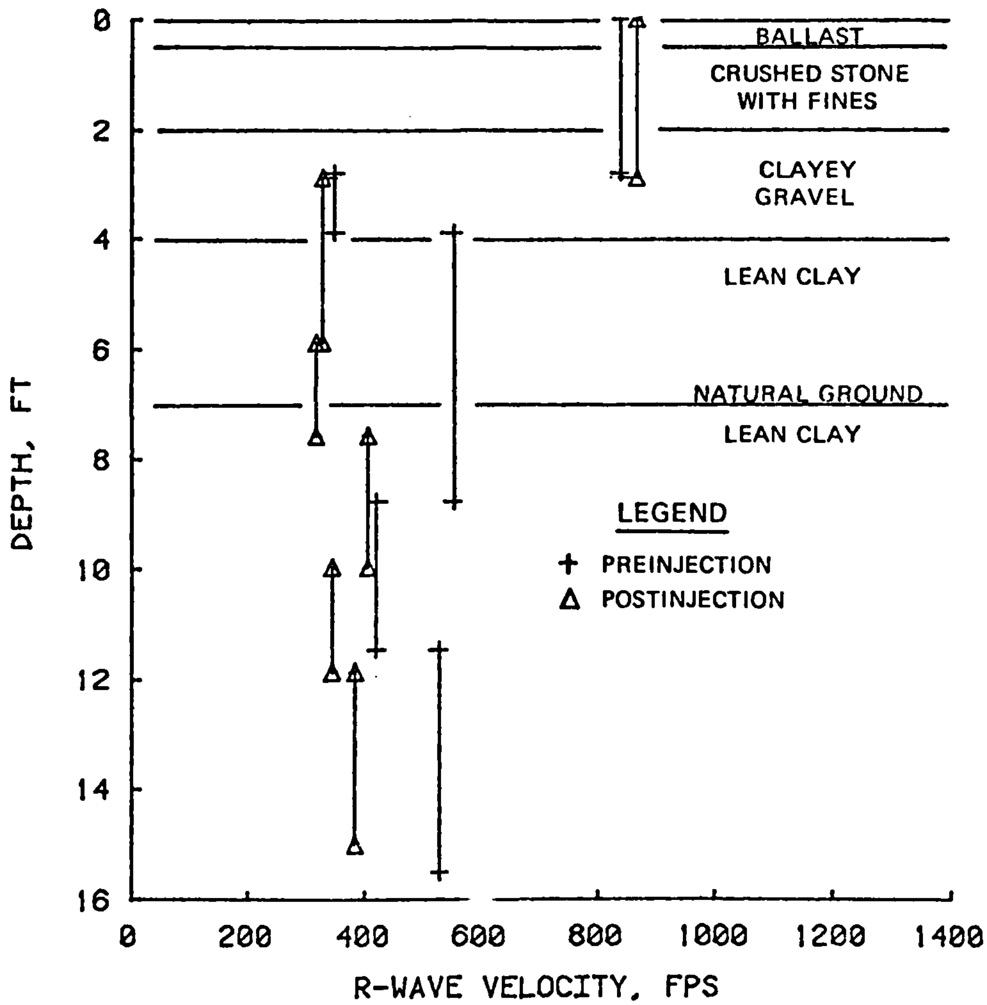


FIGURE 5-10. TYPICAL PREINJECTION AND POSTINJECTION RAYLEIGH-WAVE VELOCITIES IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

Figure 5-11 shows typical results for the RIR test sections. A comparison of Figure 5-11 and 5-9 shows that the S-wave velocities were essentially the same in 1979 as in 1976, which implies that the lime-stabilized zone had not deteriorated due to the 3 years of elapsed time, traffic, and environmental change. Due to the layering in the embankments, the R-wave velocity data in Figures 5-10 and 5-11 have been analyzed by a procedure developed in Appendix C. The velocity data are presented with connected symbols representing the average velocities in depth zones as assumed to be determined by the change in R-wave length.

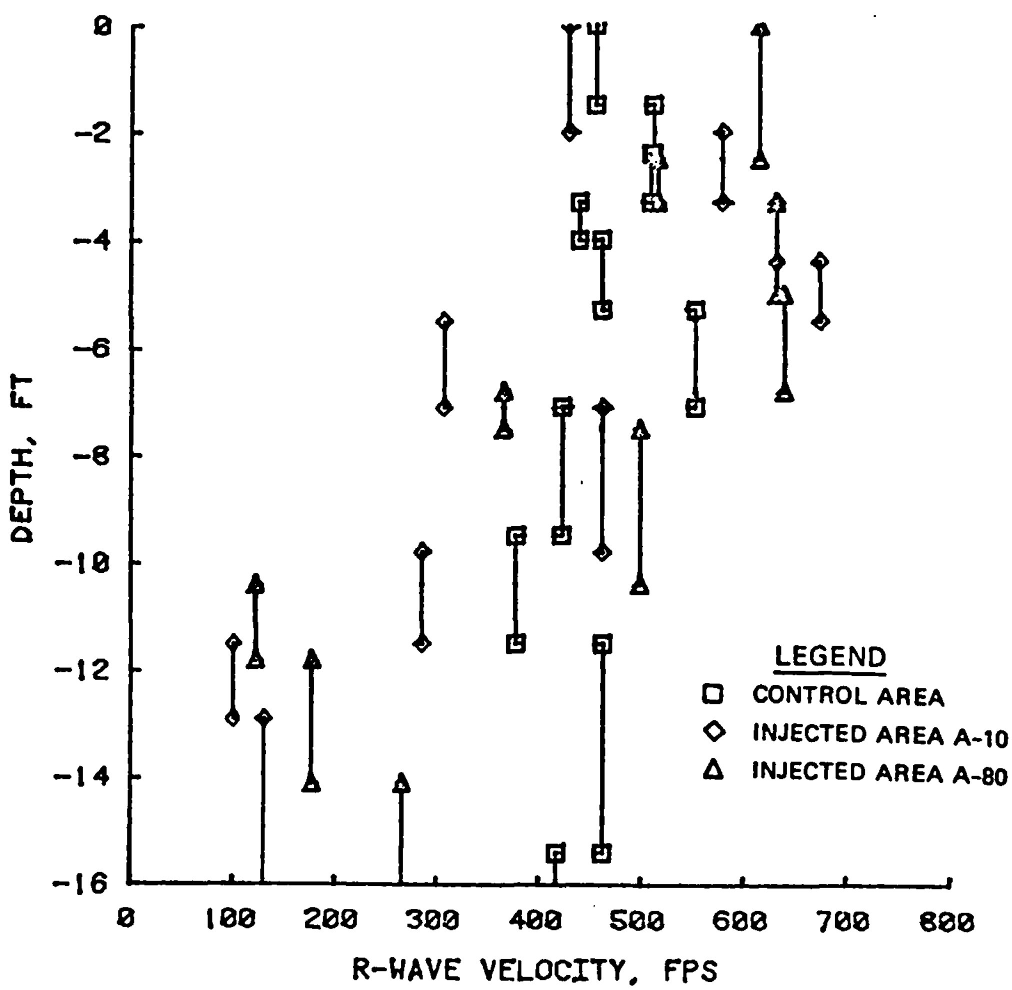


FIGURE 5-11. TYPICAL POSTINJECTION RAYLEIGH-WAVE VELOCITIES IN THE RIR TEST SECTIONS, 1979

5.2.3.3 Surface Refraction P-Wave Velocity - Tables 5-2 and 5-3 present typical results of the surface P-wave tests: The top materials of the embankments had significant P-wave velocity increases.

TABLE 5-2. AVERAGE P-WAVE VELOCITIES, WES TEST EMBANKMENT

LAYER(S)	PREINJECTION VELOCITY, FPS	POSTINJECTION VELOCITY, FPS	
		CONTROL	TREATED
Ballast and crushed stone	1585	1579	1510 (top 1 ft)
			2519 (bottom 1 ft)
Clayey gravel and silt	1060	1077	--
Subgrade lean clay	1100	--	--

TABLE 5-3. AVERAGE P-WAVE VELOCITIES, RIR TEST SITE A, 1979

LOCATION	VELOCITY, FPS
Untreated area top mixture	833
Treated area top mixture	1111 (station 0+10)
	1074 (station 0+80)

5.2.3.4 Surface Electrical Resistivity - Figure 5-12 shows typical electrical resistivity sounding results for the WES embankment. A significant resistivity decrease is shown in the top 1 to 1.5 ft (depth equals approximately one half the electrode spacing[3]), which indicates improved electrical conductivity in this zone. No electrical resistivity surveys were conducted on the RIR.

5.2.3.5 Dutch Cone Penetration - Figure 5-13 shows typical Dutch cone penetration tests results for the WES embankment. The data are point resistance (q_c)

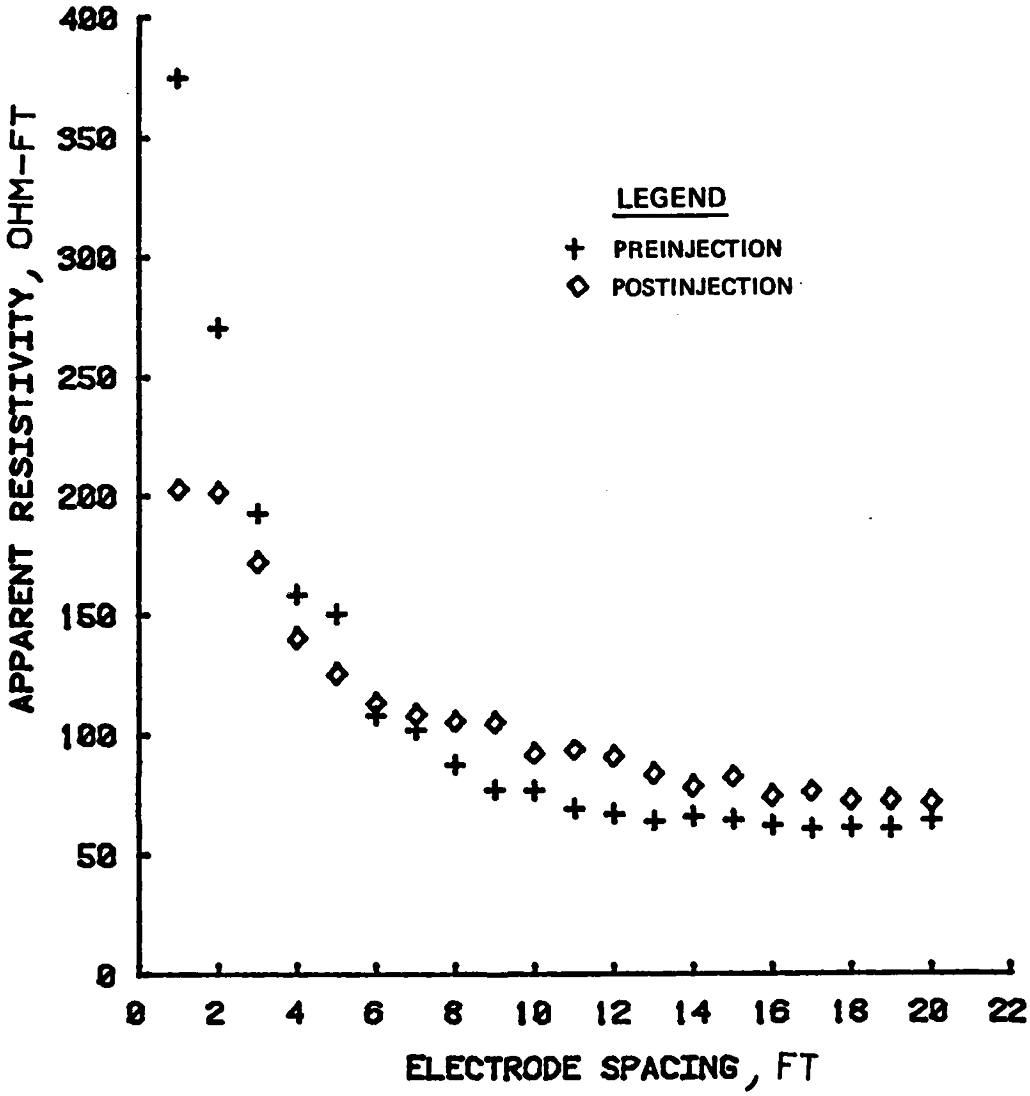


FIGURE 5-12. TYPICAL PREINJECTION AND POSTINJECTION ELECTRICAL RESISTIVITY SOUNDING RESULTS IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

and sleeve friction resistance (f_s) values for 3 in. out of every 6 in. going from the surface of the crushed stone with fines layer to the 15-ft depth. As can be seen, the strength increase in the crushed stone with fines material layer is evident. Also, no high-strength seams or zones were detected in the materials below the crushed stone with fines layer. Figure 5-14 presents typical Dutch cone penetration point resistance for the clayey silt layer in the vicinity of station 0+45 where the high water contents discussed in

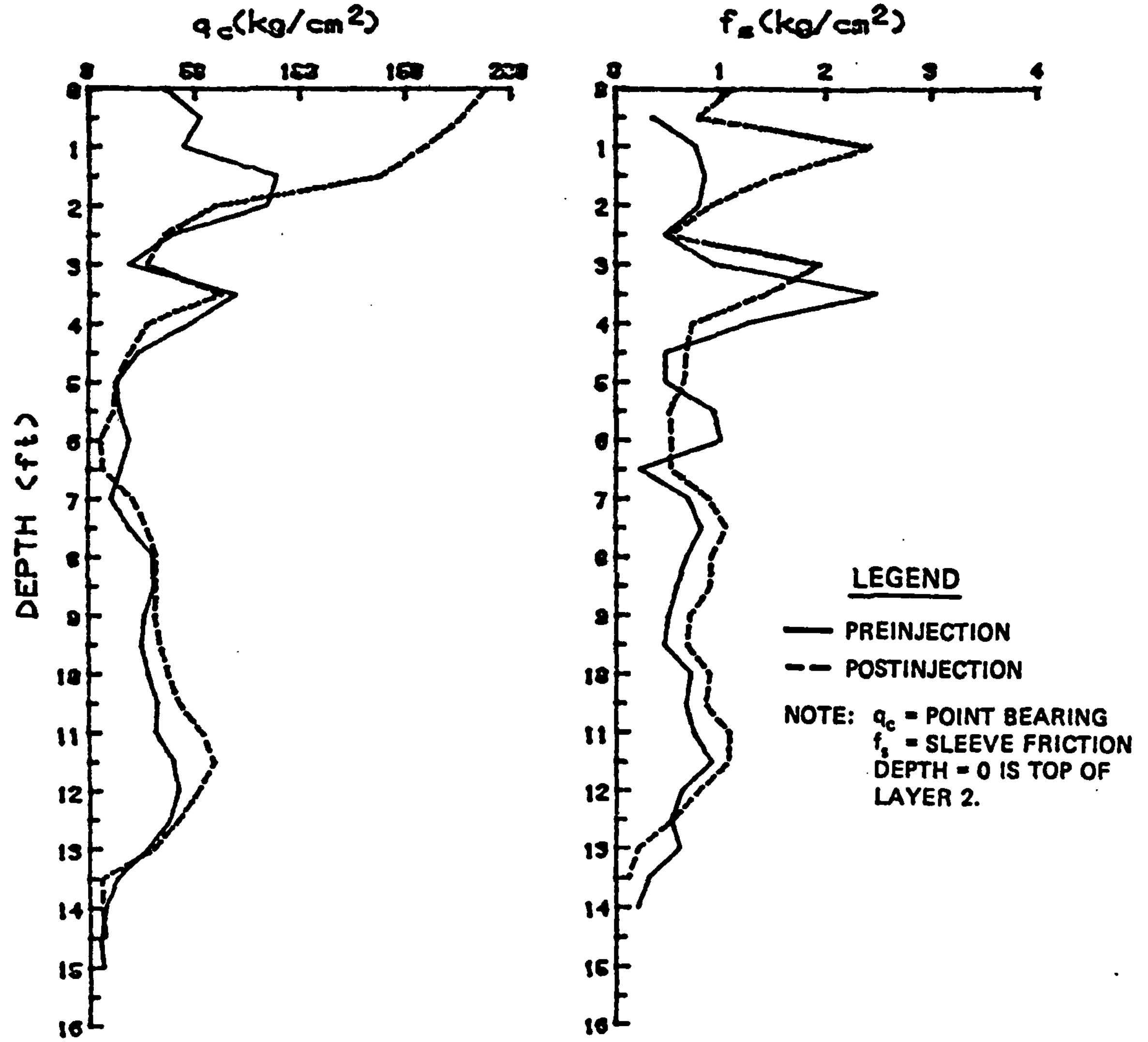


FIGURE 5-13. PREINJECTION AND POSTINJECTION DUTCH CONE MEAN VALUES OF ALL TESTS IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

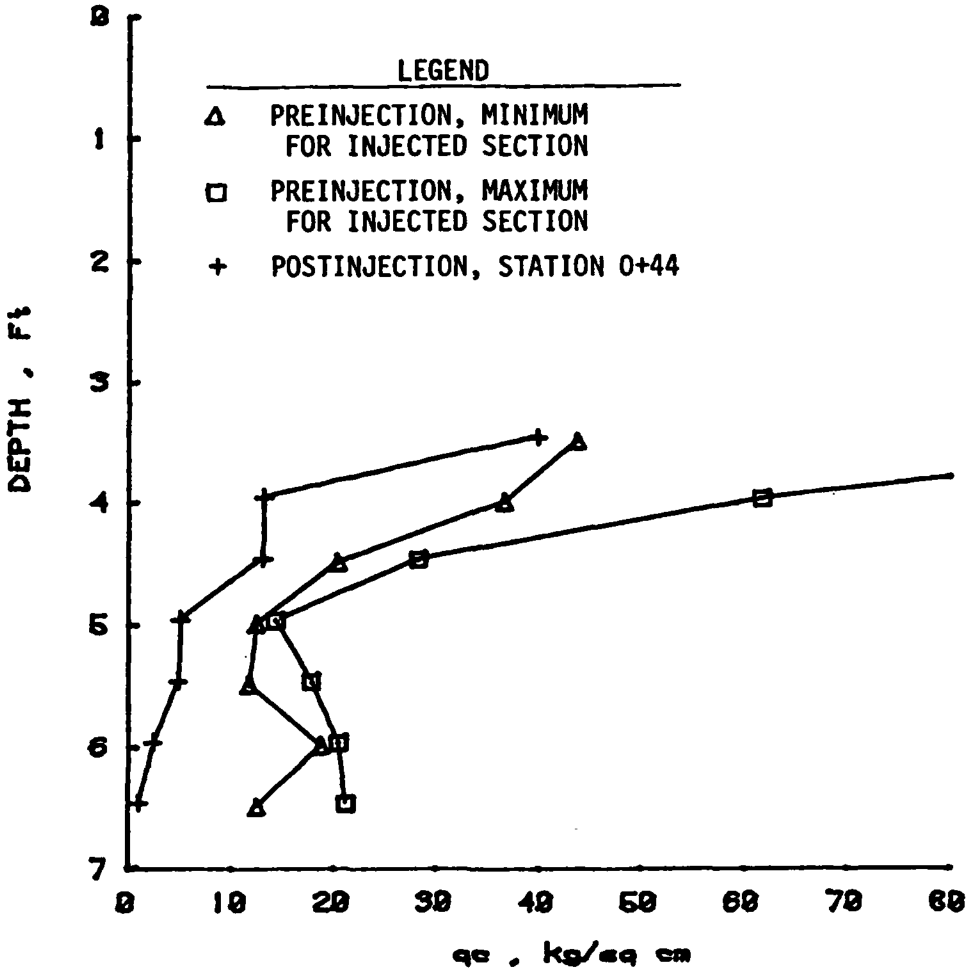


FIGURE 5-14. TYPICAL DUTCH CONE TEST RESULTS IN THE CLAYEY SILT LAYER OF THE INJECTED PORTION OF THE WES TEST EMBANKMENT, STATION 0+43.65

Section 5.2.2.1 were obtained. The significant decrease in point resistance between preinjection and postinjection results in Figure 5-14 is obvious and can be attributed to the large increase in water content. Figures 5-15 and 5-16 show typical Dutch cone penetration test results for the RIR. Comparison of Figures 5-15 and 5-16 shows the zone that has been stabilized by lime slurry.

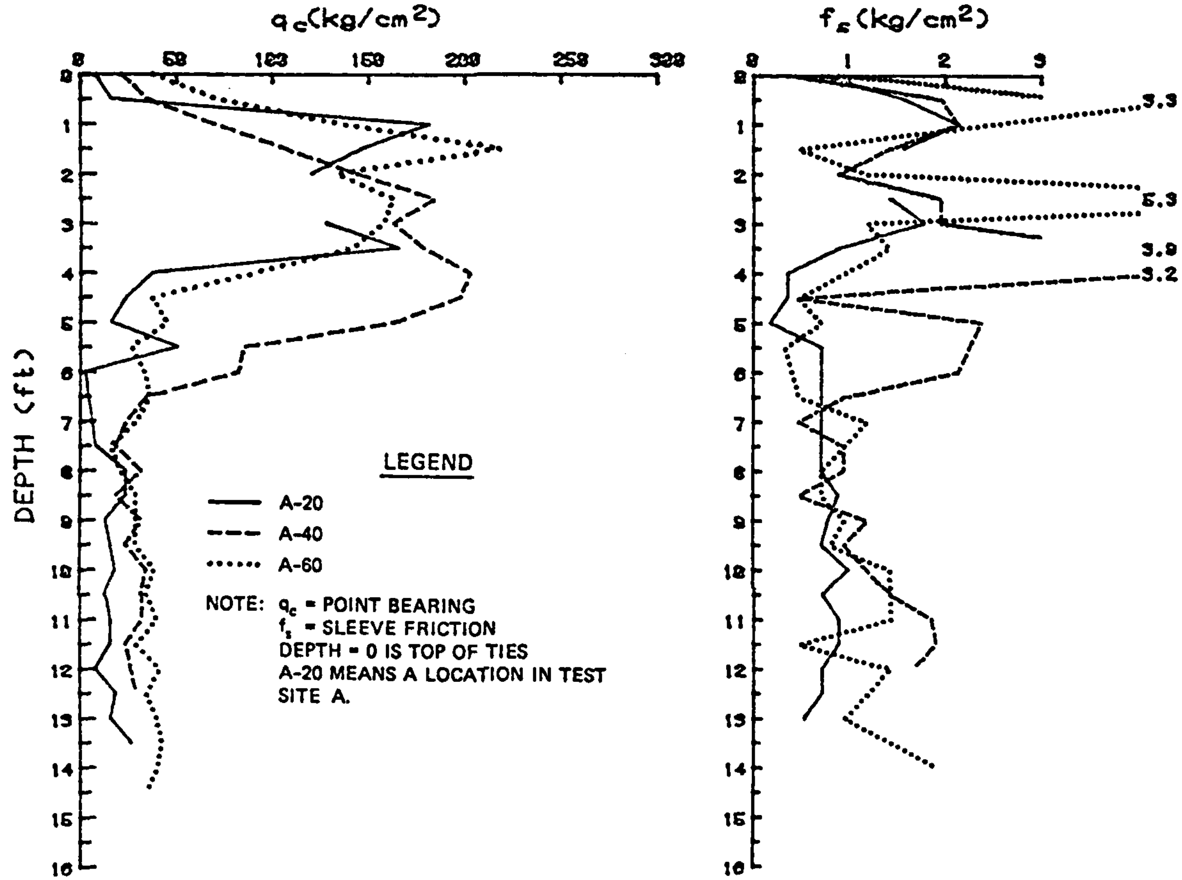


FIGURE 5-15. TYPICAL DUTCH CONE TEST RESULTS IN THE UNTREATED AREAS OF THE RIR TEST SECTIONS, 1979

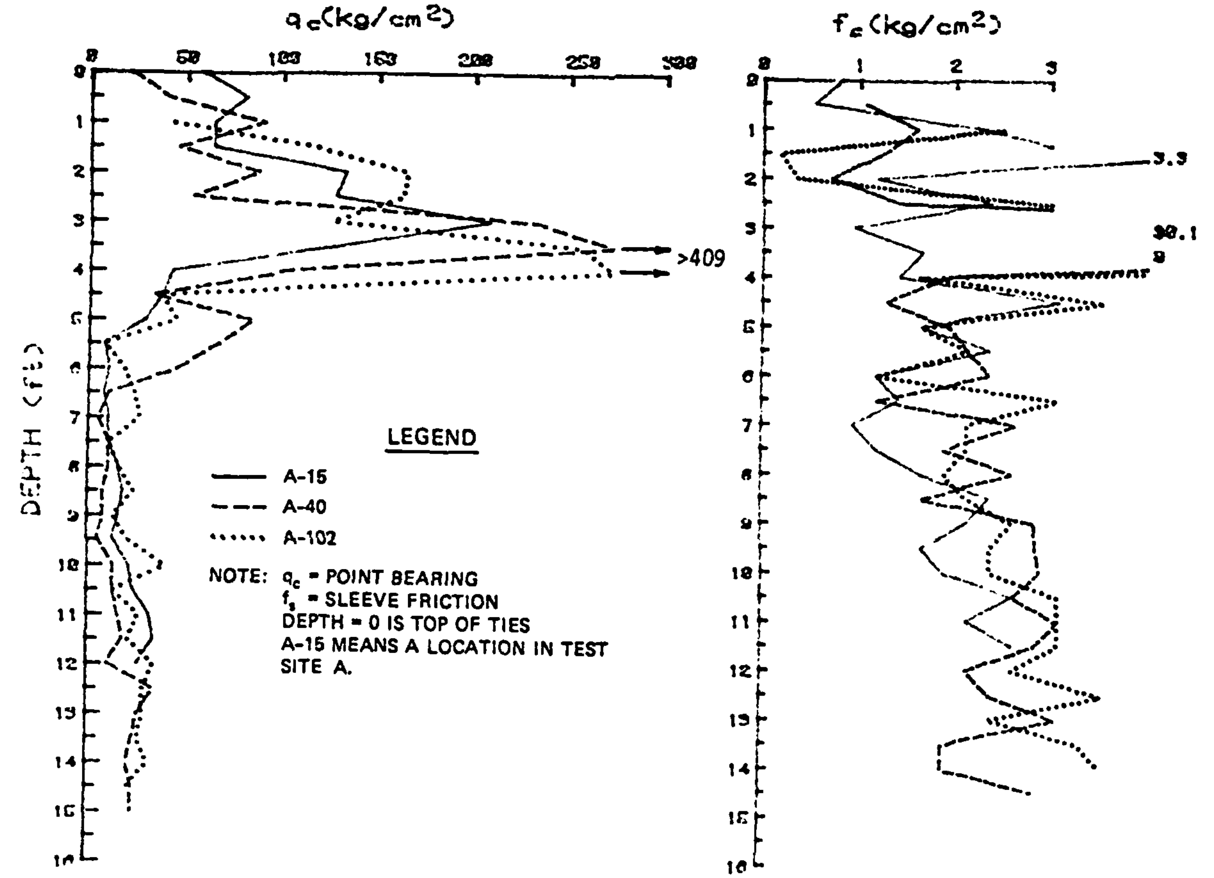


FIGURE 5-16. TYPICAL DUTCH CONE TEST RESULTS IN THE INJECTED AREAS OF THE RIR TEST SECTIONS, 1979

5.2.3.6 Standard Penetration Test - Figure 5-17 shows typical results of the standard penetration test (SPT) as blows per 6 in. versus depth for the WES embankment. As is obvious from a comparison of Figures 5-17 and 5-13, the SPT is not as sensitive as the Dutch cone penetration test. The SPT was not conducted on the RIR.

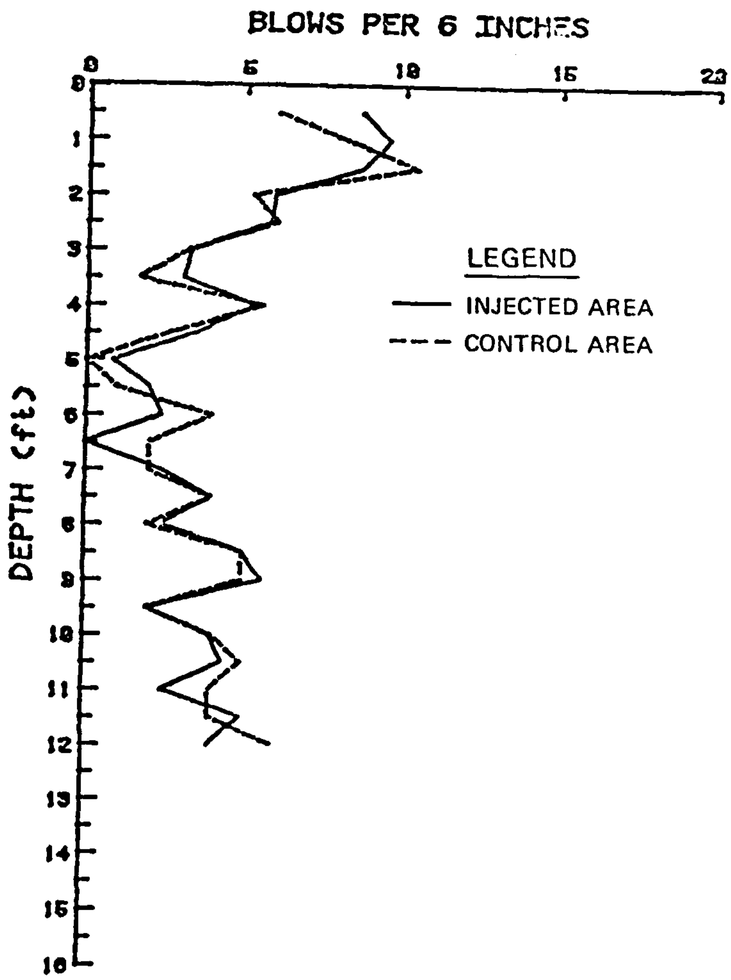


FIGURE 5-17. POSTINJECTION MEAN VALUES OF ALL STANDARD PENETRATION TEST RESULTS IN THE WES TEST EMBANKMENT

5.2.3.7 Plate Load Test - Plate cyclic load tests were conducted on the WES embankment by loading (a) on ballast portions without the rail panel, (b) on crossties for which the plate spanned two ties, and (c) through the rail heads by means of a bridge. For each load increment in the cyclic tests, the data were reduced to responses of total, elastic rebound, and inelastic vertical deformations. (Inelastic deformation is the difference between the total and rebound deformations.)

Figures 5-18 and 5-19 show typical results for cyclic load tests through the rail heads. The significant reduction in vertical deformation is also typical of test results from loadings on the crossties and ballast portions.

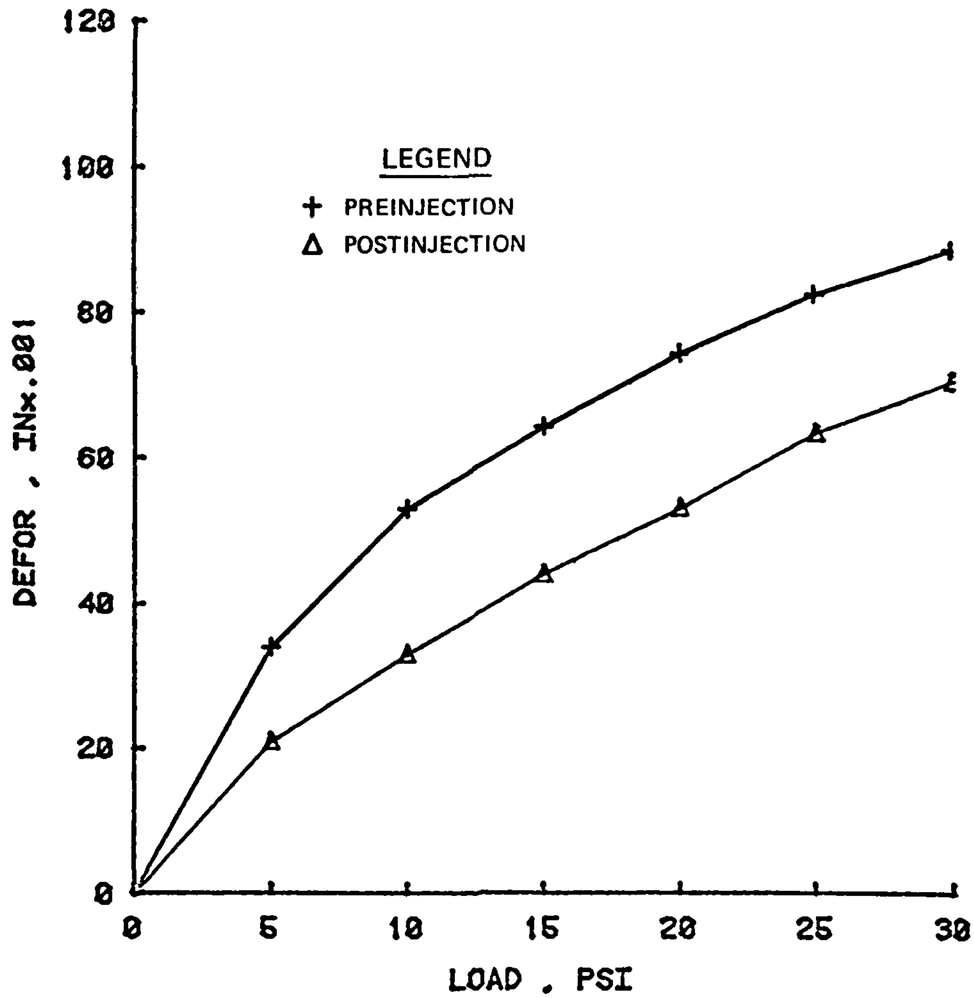


FIGURE 5-18. TYPICAL PREINJECTION AND POSTINJECTION VERTICAL TOTAL DEFORMATION FROM PLATE CYCLIC LOAD TESTS THROUGH THE RAIL HEADS OF THE INJECTED PORTION OF THE WES TEST EMBANKMENT

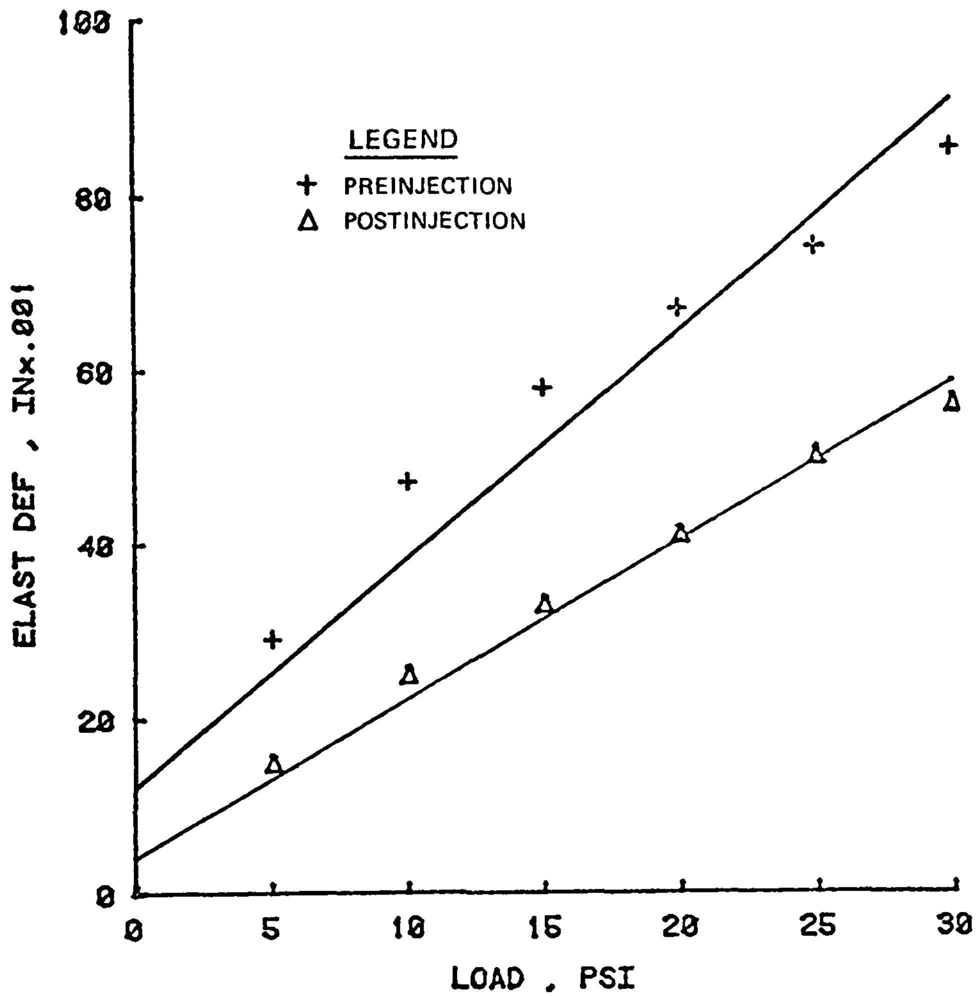


FIGURE 5-19. TYPICAL PREINJECTION AND POSTINJECTION VERTICAL ELASTIC DEFORMATION FROM PLATE CYCLIC LOAD TESTS THROUGH THE RAIL HEADS OF THE INJECTED PORTION OF THE WES TEST EMBANKMENT

Plate load tests were not conducted on the RIR; however, total vertical deformations measured when the 16-kip mass vibrator was lowered onto test locations on the WES embankment indicated that the lime-treated areas deformed in the order of 0.050 to 0.060 in. less than the untreated areas.

5.2.3.8 Dynamic Stiffness - Dynamic stiffness data were derived from the mechanical impedance test results at the 20-Hz frequency point. These were conducted on the WES embankment ballast, crossties, and rail heads. The RIR sites were tested on the crossties and rail heads.

Figure 5-20 shows typical vertical dynamic stiffness results for the WES embankment. (Dynamic load in the figure is the single peak value.) As can be seen, the dynamic vertical stiffness results for the WES embankment did not show the strength increase indicated by the cyclic load plate tests. As previously mentioned in Section 5.2.2.1, the laboratory determined rebound elastic deformation versus load curves were parallel in the vibratory load range. The cyclic load plate tests operate in the initial portions of these response curves which are not parallel. A plate load test starts at zero load condition, whereas the vibratory load test is operating around the 16-kip mass static load. Therefore, the cyclic plate load test results and the dynamic stiffness results would not show similar behavior.

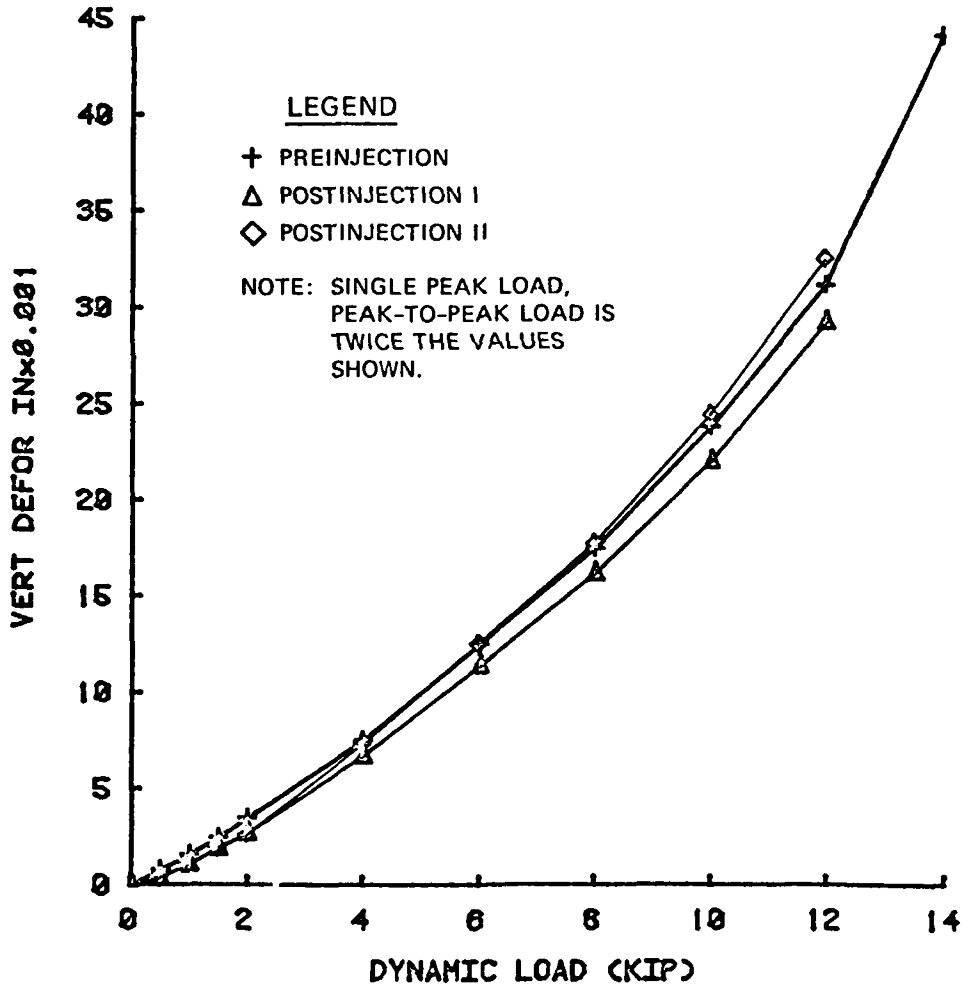


FIGURE 5-20. TYPICAL PREINJECTION AND POSTINJECTION VERTICAL DYNAMIC DEFORMATION AT 20 HZ IN THE BALLAST AREA OF THE INJECTED PORTION OF THE WES TEST EMBANKMENT

Figure 5-21 shows typical results for the RIR. The treated area of site A averaged 32 percent stronger (comparing dynamic loads at the 0.020-in. vertical deformation point) than the untreated area.

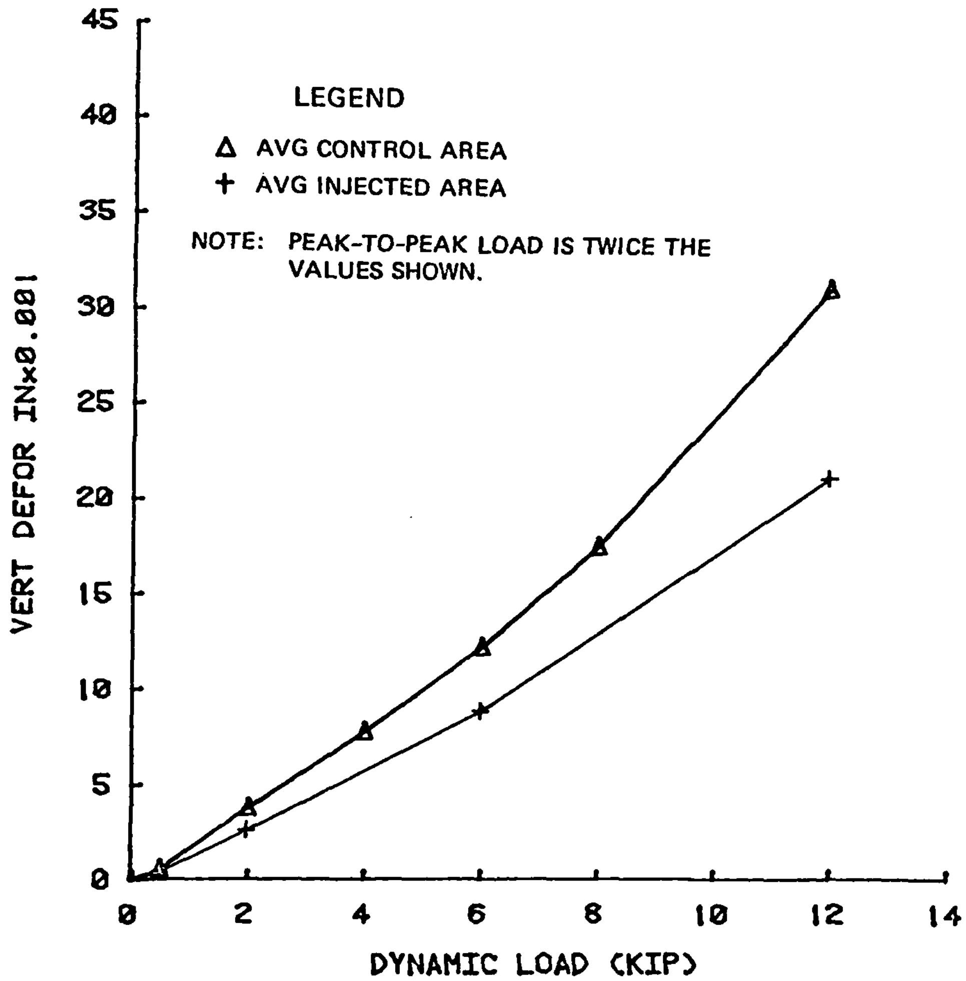


FIGURE 5-21. TYPICAL AVERAGE VERTICAL DYNAMIC DEFORMATION AT 20 HZ THROUGH THE RAIL HEADS OF UNTREATED AND TREATED AREAS OF THE RIR, 1979

5.2.3.9 Mechanical Impedance - The mechanical impedance test in this study was an adaptation to geotechnical investigation from the studies of mechanical and electrical engineering. In the past, mechanical impedance testing as applied to soils has not been extensively employed or studied, and no wealth of literature exists. (Cooper[7] reported results for the Kansas Test Track.)

The attraction of such testing for geotechnical investigations is that the test offers, for a soil system, in situ determination of the dynamic characteristics (elastic stiffness (k), damping (c), participating mass (m), and resonant or modes of resonant frequency) necessary for geotechnical dynamic analysis. Because mechanical impedance testing is new to geotechnical investigations and the wealth of information contained in the test results is, at the present state of the art, not fully developed, only limited use of the test results (comparisons of dynamic stiffness at a selected frequency) was made in the study. A hypothesis for interpreting the mechanical impedance test results in terms of nonlinear response and distinguishing different layer contributions is presented in Appendix D along with some typical test results.

The mechanical impedance test was conducted on both the WES and RIR test sites. As discussed previously in Section 5.2.3.8, no major changes in dynamic stiffness in the test load range occurred in the WES embankment. Therefore, no significant changes or shifts occurred in the impedance curves from preinjection to postinjection. The RIR test results did show a shift in the mechanical impedance curves to higher stiffness in the lime slurry injected test sites.

5.2.3.10 Vane Shear - The vane shear device used in this study starts testing at a 6-ft depth due to its mechanical construction. On the WES embankment, the 6-ft depth was below the stabilized crushed stone with fines layer; therefore, no data were acquired in the stabilized material. The test was not conducted on the RIR.

5.2.3.11 Infrared Survey - Temperatures were monitored on the WES test sites with an infrared thermometer for several days after lime slurry injection. No temperature differences within the $\pm 1^{\circ}$C accuracy of the instrument were detected.

5.2.3.12 Downhole Seismic Tests - Good quality signals were not generated at the WES and RIR test sites due to the loose granular surface ballast. Seismic signal propagation has always been a problem in loose granular materials due to such things as poor seating of the seismic source and receivers and generally poor transmission of signals through such open-spaced materials.

5.2.3.13 CBR Tests - Postinjection small aperture CBR tests were not conducted after other tests showed that the lime and major strength changes occurred only in the crushed stone with fines material on the WES embankment. In granular materials, the CBR test results are erratic due to the influence and bearing on individual particles. No tests were conducted on the RIR.

5.2.3.14 Surface S-Wave Tests - Surface S-wave testing both preinjection and postinjection did not yield good results at either the WES or the RIR test sites. The problem associated with this test was that a sufficient coupling for the seismic source and receivers could not be achieved on the ballast surface.

5.2.3.15 Ground Penetrating Radar - The noncommercially available 4300-MHz CW-FM radar was not used on the WES embankment after tests showed it only penetrated a few inches in the Vicksburg loess of the level test area. The commercially built ground penetrating pulsed radar (at WES in June 1978) was tested on the embankment, and no distinction between injected and noninjected portions could be made. Also, there was no definitive evidence of the significant depth of penetration. No tests were conducted on the RIR.

5.2.3.16 Sonar - A noncommercially available 24-kHz pulsed sonar system was used for ground penetration tests on the WES embankment in August 1977. The test results showed no major difference between the injected and noninjected embankment portions. Many reflections occurred in the test data, and by having the known profile and using the known P-wave velocities, layer interfaces could be identified in the multiple reflections. However, without the known profile and P-wave velocities, interpretation of layers from the multiple reflections is questionable. No tests were conducted on the RIR.

5.2.3.17 Downhole Geophysical Logging - The borehole geophysical logging equipment was found to not be practical for use in the WES embankment because of the shallow boreholes and the thin embankment layers. For shallow boreholes such as the 15-ft-deep holes in the test sites and for expected material variations at depth to be about 2 ft apart, the tool length of 5.5 ft and the speed of operation made the logging tools relatively insensitive for the investigations required in this study. The borehole logger was not used at the RIR test sites.

5.2.3.18 Pressuremeter Tests - A Menard pressuremeter was investigated and was found not feasible for use in the WES embankment. The NX long probe with a metal strip sheath was used in the embankment granular materials. However, augering of holes and sloughing of the granular materials made the holes impractical for testing. In the fine-grained materials, the metal strip sheath provided more deformation resistance than the soil, and the materials did not provide enough deformation resistance for the practical use of the rubber sheath. No tests were conducted on the RIR test sites.

6. CONCLUSIONS

6.1 LIME SLURRY INJECTION

Results of the study reported herein strongly indicate problems with lime slurry injection of fine-grained, non-high swelling soils. The fine-grained clays and silts of the WES test sites and the RIR test sections were not injectable. Lime slurry did not disperse into the soils. However, the more permeable granular materials (mixtures of the deteriorated ballast and subgrade) were injectable and showed significant strength increases at both test locations. The improvements in performance under train loading noted at the RIR are due to the stabilization of a 1.5- to 6-ft-thick zone of deteriorated ballast and subgrade mixture. Stabilization and strength increase on the RIR sections were variable, with areas of low and high strength increase. Table 6-1 summarizes the injectability of the soil types investigated at WES and on the RIR and shows that gaps exist in direct knowledge of injectability for some types of soils.

The injection problem is believed to be due to three basic factors: (a) fine-grained soils with grains finer than the lime particles which will not allow dispersal of the lime, (b) low permeability of fine-grained soils which will not allow penetration and dispersion of the slurry liquid, and (c) the standard injection technique of continuous high pressure and flow which will hydraulically jet a hole in fine-grained soils and wash the slurry to the surface or into a more permeable material such as the ballast mixture zone. Dry and highly fissured clay (not investigated in this study) above the groundwater table in the dry season of the year may be an exception to the above factors. A better technique is believed to be stopping flow and pressure before advancing to each depth interval and bringing pressure and flow up slowly to less than 25 psi.

In many cases, stabilization of only the old ballast-subgrade mixture will be enough to improve the support conditions because: (a) this tends to waterproof the top of the subgrade, and (b) the strength increase in this zone which is often fairly thick is very beneficial. The top few feet of a railroad structure carries the majority of the train load, and strengthening

TABLE 6-1. INJECTABILITY OF SOIL TYPES
INVESTIGATED AT WES AND ON RIR

Symbol	Injectability
GW	
GP	
GM	Good injectability[a] WC = 4% γ_d = 123 pcf
GC	Noninjectable[a] WC = 7 to 20% γ_d = 109 to 125 pcf
SW	
SP	
SM	
SC	
ML	Noninjectable[a,b] WC = 22 to 30%[a] γ_d = 90 to 100 pcf[a] WC = 25 to 45%[b]
CL	Noninjectable silty clay to slightly injectable lean clay with poor impregnation[a,b] WC = 17 to 33%[a,b] γ_d = 85 to 104 pcf
OL	Organic soils do not have good soil-lime chemical reactivity
MH	
CH	Noninjectable[b] WC = 43 to 53%[b]
OH	Organic soils do not have good soil-lime chemical reactivity
Pt	Organic soils do not have good soil-lime chemical reactivity

NOTES: WC = in situ water content; γ_d = in situ dry density

[a] From the WES test sites in Vicksburg, Mississippi.

[b] From the RIR test section, Forrest City, Arkansas.

this zone would have the greatest benefit for improving performance. Strengthening the old ballast mixture zone will also decrease the train-induced stress on the subgrade and allow it to perform better. In other words, a stabilized ballast mixture zone will bridge a soft troublesome subgrade.

Another potential problem was indicated by this study. Uncured wet lime slurry was found in the WES and RIR test sites, which at the RIR sites represented material after an elapsed time of approximately 3 years after injection. For successful LSPI stabilization, the soil must be dry enough and of sufficient permeability to absorb the water from a slurry and allow the lime to cure and react. For fine-grained, relatively impermeable, non-high swelling soils which usually have high moisture content, the results of this study indicate that the curing and reaction of lime will not occur because these soils do not rapidly change moisture content or drain. Drainage of clayey soils takes many years (up to hundreds of years depending on load and drainage path conditions). In relatively impermeable fine-grained soils, the addition of water which cannot be rapidly drained will prevent lime from curing and will cause strength loss instead of a strength gain. Strength loss occurred at locations in the fine-grained soils of both the WES test embankment and the level test area.

6.2 CHANGES CAUSED BY LIME INJECTION

Lime slurry was not successfully injected into the fine-grained lean clays of the WES level test area. Consequently, no strength increases occurred. However, a change was effected in a localized zone at about 7 to 9 ft where a concentration of injected slurry occurred. The result was that an almost complete strength loss occurred due to the increased moisture content in this zone.

On the WES test embankment, the only material that successfully accepted the lime slurry was the crushed stone with fines layer which simulated broken and deteriorated ballast. A localized zone of the clayey silt layer had significant increases in both moisture and lime concentration. However, the increased moisture content did not allow the lime to cure and caused a significant strength loss in the material. The lime in the crushed stone with fines material did cure and resulted in a significant strength increase.

For the RIR test sections, the only material that successfully accepted the lime slurry was the old ballast and subgrade mixture zone which was thicker than the similar layer in the WES test embankment. The S-wave velocity tests in 1976 indicated a dynamic shear modulus increase. In 1979, the R-wave velocities indicated that the S-wave velocities had not significantly changed since the 1976 values. Results of the vibratory load and Dutch cone tests conducted in 1979 showed that the treated areas were significantly stronger than the untreated adjacent areas. The test results imply that the lime-stabilized zone had not deteriorated after approximately 3 years elapsed time and approximately 45.3 million gross tons of traffic. The RIR maintenance department stated that several of the lime-injected sections, including the designated test areas, had not required maintenance since lime injection and track resurfacing (including new crossties and continuous welded rails). However, other sections of this line have required continuous maintenance.

6.3 NONDESTRUCTIVE TESTING

Based on NDT technique sensitivity and repeatability and on the need for geotechnical engineering data, and most importantly on the test effort and time (considering the time period a section of railroad track would be closed to traffic), the following techniques are believed best suited for evaluating the changes caused by lime slurry injection and the overall conditions of a track support system (further discussion of these and their use is given in Section 7):

a. Surface vibratory generated R-wave velocities.

b. Surface refraction yielding P-wave velocities.

c. Mechanical impedance yielding definition of dynamic nonlinearity response with respect to frequency and load magnitude and dynamic stiffness data.

d. Dutch cone penetration yielding relative strength comparisons between material layers and strength variations throughout a site or between different sites.

e. Small aperture CBR yielding CBR values of the material layers and describing the variability throughout a site or between different sites.

f. Static and/or semistatic loading, which from a test time and effort consideration, can be efficiently conducted with a large mass in a railroad track vehicle.

7. RECOMMENDATIONS

7.1 LIME SLURRY INJECTION

The only way to know if lime slurry injection has a possibility of stabilizing or improving a particular railroad site is through certain laboratory and field tests. Soil samples should be obtained and laboratory tests conducted in order to determine if the soil is chemically reactive (pH $\approx$ 12.4) with small amounts of lime, such as 1 to 2 percent. If the soil is not chemically reactive, no pozzolanic reaction or strength increase will occur. If the soil is chemically reactive, either laboratory or field permeability tests should be conducted to determine if the soil will allow dispersion of the lime slurry during the field pressure injection process. If the soil is relatively impermeable and has few fractures, slurry liquid will not travel into the soil in sufficient amounts to cause changes or chemical reactions. The soil should be classified and the grain-size distribution should be determined (except for clays), because the soil can filter out lime particles and prevent dispersion of the lime slurry. At the present time, the only way to judge if soils are injectable is to conduct field pumping tests with an injection rod and lime slurry. Samples* must be obtained and inspected or test pits dug and inspected to determine if the soil is being impregnated, if lime seams are forming, and if lateral dispersion of the slurry is occurring to cover the space between proposed injection points.

If roadbed subsoils prove to be nonreactive and/or noninjectable, consideration can be given to lime slurry injection stabilization of the ballast layers and pockets and the old ballast-subgrade mixture zones. When field pumping tests are conducted, they should also include the ballast materials. Chemical reactivity of the ballast materials should be checked. The present study showed that stabilized ballast layers are the chief source of increased roadbed stability and strength in the cases of the lean clay subgrades investigated. Stabilized ballast will help prevent water infiltration into the roadbed subsoils and pockets and will reduce load-induced stress in the subsoil. Lime stabilization of degraded ballast and subballast materials should be

* Laboratory test sample quality is not necessary; however, the soil stratifications and structure must be preserved and no mixing should have occurred.

considered as economically and technically sound at sites where subsoils do not permit good dispersion of the slurry and where stabilization of these materials provides the largest gain in stability and strength.

Based on the results of the present study, injection techniques in certain fine-grained soils need to be modified. If the slurry flow is turned off before advancing to each depth interval, the soil will tend to seal around the injection rods better than if a continuous slurry flow is maintained. Field pumping tests using an injection rod and lime slurry will indicate the best injection technique to be used in a particular soil.

7.2 NONDESTRUCTIVE TESTING AND EVALUATION

Track roadbeds can be tested with the selected nondestructive techniques presented in Section 6. Evaluation can only come either from a relative comparison with track sections of known geotechnical conditions and performance or, more meaningfully, from a general data base of track performance. However, at the present time, there is no quantitative link between calculated or measured track-soil-structure response indices and performance or behavior in pertinent terms such as useful life. A data base is needed for relating field test results, laboratory test results, and analytical methodology to performance The relationship must include factors such as age, traffic history, maintenance history, soil type, drainage and track conditions, and environment. The non-destructive techniques, coupled with proper laboratory tests and analytical analysis, should be used to gather data in order to build a data base.

A possible subgrade condition evaluation methodology which would develop both specific and general data elements should include the following:

a. Profiling the subsurface layers, and detecting anomalies such as ballast pockets with a cone penetrometer and/or radar (when a method is proven) which would determine thicknesses and moisture content data.

b. Acquiring cone penetration data for relative strength comparisons within a site and for extrapolation to other sites.

c. Acquiring small aperture CBR data for each subsurface material for correlation with the data base of pavement behavior.

d. Conducting surface refraction seismic tests yielding compression wave velocities.

e. Using a high-railed large dynamic load vibrator (30-kip peak-to-peak load or larger) to generate and acquire response data from static load tests, impedance tests, and R-wave propagation.

The static load tests would provide data concerning the track substructure inelastic and total deformation response characteristics. As for pavements, the inelastic deformation response characteristics for track substructures are directly related to performance and remaining service life. The effects and importance of inelastic behavior are evidenced by the rapid deterioration of track in slow speed and stopping areas such as dual track, switches, inclines, and curves. Soil materials respond to loading with elastic and inelastic deformation phases which act simultaneously to give a total deformation under load. Soils subjected to moving loads, which induce rotation of principal planes, have an inelastic deformation response that in most cases is equal to the elastic deformation. Steady state dynamic responses are indicative of only the elastic response deformations within the load range employed. Definition of the inelastic deformation response is important and changes in it should be determined due to its correlation with surfacing activity; therefore, static or semistatic loading response is necessary for determining magnitudes and changes in the inelastic and total deformation responses of a track-soil-support system.

The impedance tests would provide track structure nonlinear response data concerning dynamic elastic deformation behavior, resonant frequencies, and frequency response characteristics that relate to moving load behavior. Measurement of velocities of the R-waves propagated by the surface vibrator in the impedance test will closely approximate S-wave velocities for the layers of a structure from which elastic shear modulus (G) values at low strain levels are obtained. In addition to G, combining the surface refraction test P-wave velocities with the S-wave velocities for the top materials and using the theory of elasticity will result in in situ small strain values of Young's modulus (E) and Poisson's ratio (ν). The CBR data will provide a link with the existing wealth of pavement response information that could provide benefits in determining track substructure behavior and life.

Study of the mechanical impedance test data should continue as well as the track-soil system behavior that it is portraying in the possible hope that the contribution of each distinct material layer to the composite behavior can be defined. Further work along this line in impedance data analysis could be important from the standpoint of remotely locating soft, low strength subsurface problem layers in a track-soil structure, and remotely determining the dynamic response or contribution of each distinct layer relative to the response of the composite system.

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APPENDIX A
SOIL CLASSIFICATION

TABLE A-1. UNIFIED SOIL CLASSIFICATION SYSTEM

Group Symbol	Description
GW	Well graded gravels, gravel-sand mixtures, little or no fines
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
GM	Silty gravels, poorly graded gravel-sand-silt mixtures
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
SW	Well graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, poorly graded sand-silt mixtures
SC	Clayey sands, poorly graded sand-clay mixtures
ML	Inorganic silts and very fine sands, rock floor, silty or clayey fine sands with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silt-clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays of medium to high plasticity
Pt	Peat or other highly organic soils

APPENDIX B
TESTING TECHNIQUES

B.1 GEOPHYSICAL TECHNIQUES

B.1.1 Seismic Tests

The following seismic test techniques were employed in this study:

B.1.1.1 Crosshole Shear Wave (S-Wave) Velocity - The test involves the determination of the S-wave velocity of each layer of earth material within the depth of interest through the measurement of the arrival time of the vertical S-wave component of a seismic signal that has traveled from a source in one borehole to a detector in another.[3,4] In this study a vertically polarized shear wave source was used as depicted in Figure B-1.

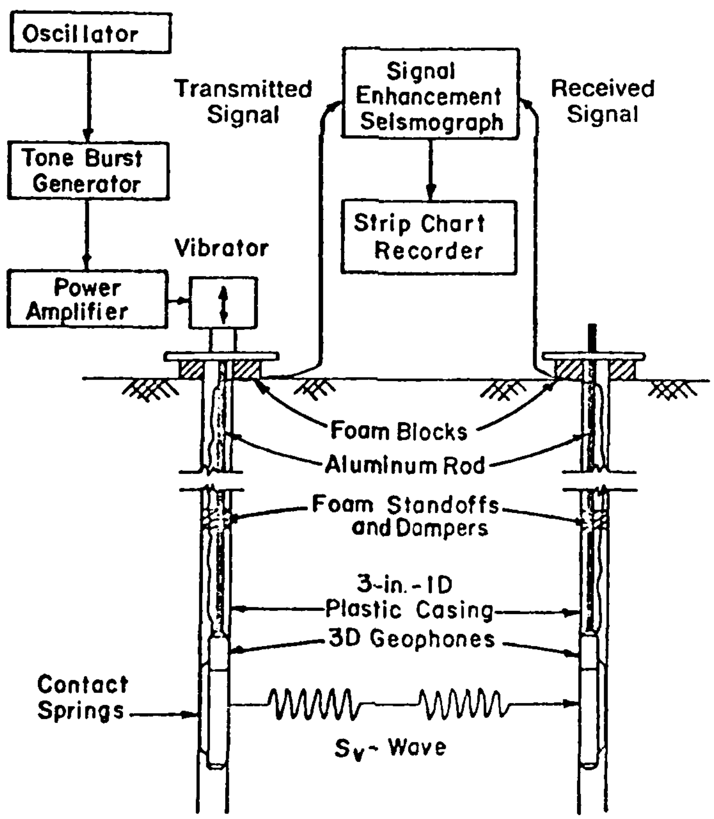


FIGURE B-1. EQUIPMENT AND PROCEDURE USED IN THE CONDUCT OF CROSSHOLE S-WAVE TESTS WITH VIBRATORY SEISMIC SOURCE[4]

B.1.1.2 Downhole Vertical S-Wave and Compression Wave (P-Wave) Velocities - The test involves the determination of the P- and S-wave component travel times and average velocities from an energy source on the ground surface to detectors in a borehole.[3,5] For this type survey, the travel paths of the seismic signal are forced to traverse all of the material layers between the source and detector. The P-wave source used is the striking of a metal plate with a sledgehammer (a very commonly used source), and the S-wave source is the striking of the end of a large wooden plank on the ground surface with a sledgehammer (most commonly used source). The P-wave test is depicted in Figure B-2.

B.1.1.3 Surface Vibratory Yielding Rayleigh Wave (R-Wave) Velocity - The R-wave is a near-surface wave whose ground motion amplitude decays exponentially with depth; consequently, most of the energy is concentrated in a zone about 1-wavelength deep.[3] It is held, in common practice, that the product of frequency and wavelength approximates the S-wave velocity. For homogeneous linear elastic media and for Poisson's ratios ranging between 0.2 and 0.5, the R-wave is less than 9 percent slower than the S-wave. Therefore, S-wave velocities and shear moduli can be approximately determined with the R-wave test. By varying frequency, wavelengths are changeu and consequently different depths of material are sampled. In this test, an energy source on the surface generates R-waves and a line of geophones detects the waves and travel times. This study used as its energy sources an electromagnetic vibratory source operating in the frequency range of 30 to 300 Hz and an electrohydraulic vibratory source operating in the frequency range of 5 to 80 Hz.

B.1.1.4 Surface Refraction S-Wave Velocity - Refraction means the deflection of seismic waves from a straight path as they pass from one medium to another with different material properties. The method[3] consists of measuring the travel times of S-wave components generated by an energy source on the surface to other points at various distances along the surface of the ground as depicted in Figure B-3. The propagation of seismic energy through elastic media and across boundaries between layers is described by essentially the same laws that govern the propagation of light rays through transparent media. In this study, the energy source was striking the end of a large wooden plank with a sledgehammer.

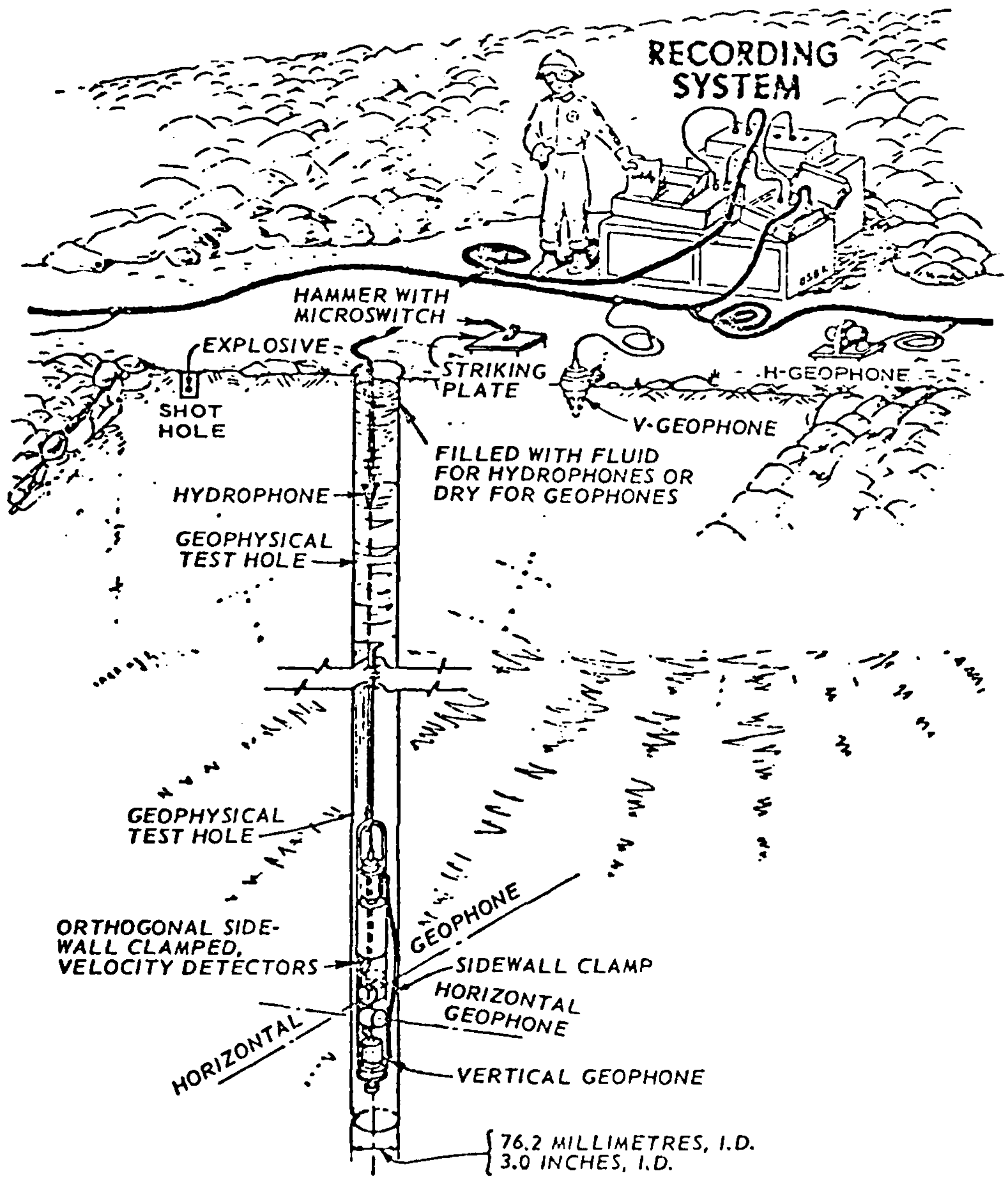


FIGURE B-2. DOWNHOLE SURVEY TECHNIQUES FOR P-WAVE DATA[5]

B.1.1.5 Surface Refraction P-Wave Velocity - The above discussion is also applicable for P-wave refraction tests. P-wave refraction test results are easier to interpret because the P-wave is the first arrival in the seismic wave train. Velocities and interface depths of layers below the surface can be interpreted by conventional methods[3] if the layers are increasing in velocity with depth. Conventional interpretation fails in the case of underlying lower velocity layers.

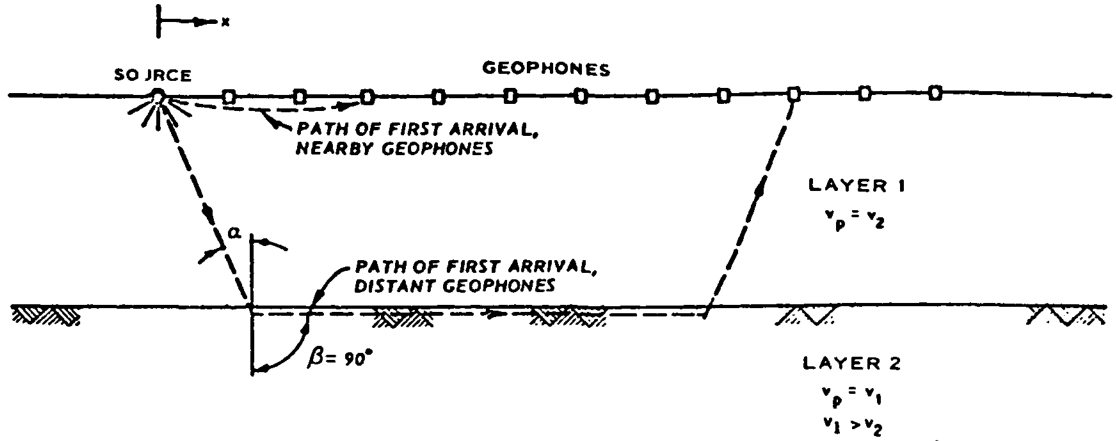


FIGURE B-3. SCHEMATIC OF SEISMIC REFRACTION SURVEY[4]

B.1.2 Surface Resistivity Test

Surface electrical resistivity surveying is based on the principle that the distribution of electrical potential in the ground around a current-carrying electrode depends on the electrical resistivities of the surrounding soils.[3] The usual technique consists of passing a current through the earth and measuring the voltage difference or potential at selected points, as illustrated in Figure B-4. A change in electrical resistivity does not indicate a strength or

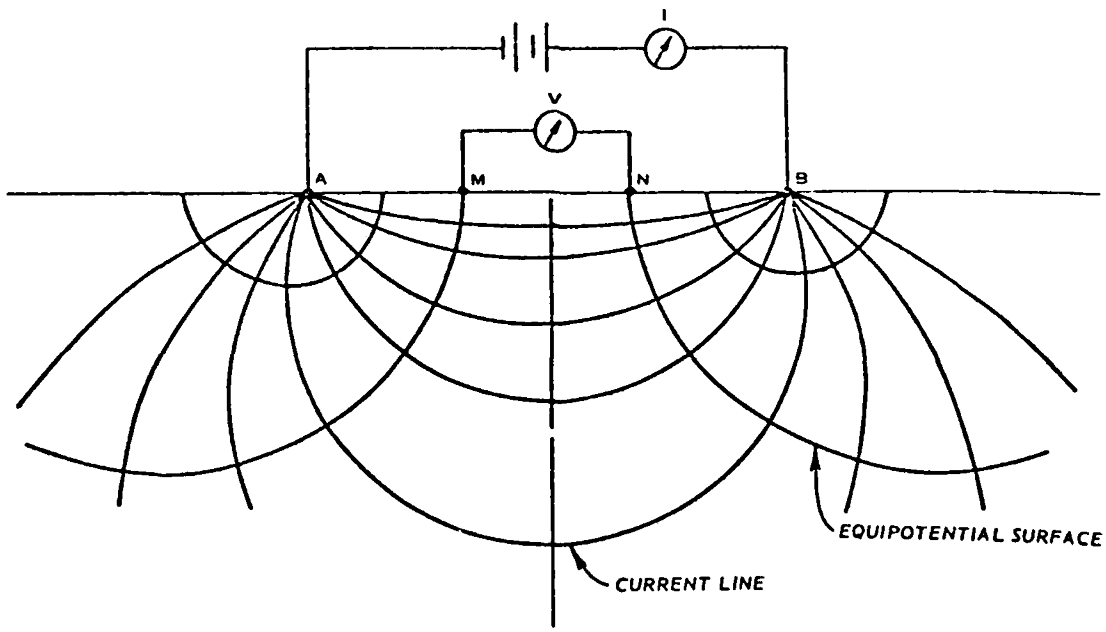


FIGURE B-4. ELECTRICAL FIELD PRODUCED BY A PAIR OF CURRENT ELECTRODES A AND B, AND ITS MEASUREMENT BY POTENTIAL ELECTRODES M AND N[5] stability change. The resistivity technique distinguishes materials only through whatever contrast exists in their electrical properties. Properties that affect the resistivity of a soil include porosity, water content, composition (clay mineral and metal content), salinity of pore water, and grain size distribution. An increase in conductivity or decrease in resistivity may indicate an increase in moisture content, increased density, or improved electrolytic conditions.

This study used four electrodes, two current and two potential electrodes, placed in the ground surface, in line, and evenly spaced. A configuration of this type is referred to as a Wenner array. Two surveying techniques were used, sounding and profiling. Soundings were accomplished by ever expanding the electrode configuration while keeping equal spacing between electrodes. As the electrodes were moved farther apart the depth of investigation was increased and the spacing correlated with the depth. For profiling, the spacing between electrodes was maintained constant, and the array was moved over the area. This allows a survey of the uniformity of a site within a depth regime which is related to the spacing chosen.

B.1.3 Radar Test

Radar subsurface profiling is penetrating the ground with microwaves and receiving the reflections from interfaces or anomalies of materials with different dielectric constants. Depth of penetration and attenuation rates are functions of microwave frequency and wavelength and of the soil type, density, moisture content, and dielectric constant.[8] Two privately owned radar systems were at WES in conjunction with a cavity detection project, one in August 1977 and the other in June 1978. One system was a commercially built (Geophysical Survey System, Inc., Model GSS1-4700P) pulsed radar system operating in the 0.05- to 1.0-GHz frequency region using 100- and 300-MHz antennas. The other system which is not commercially available was a 4300-MHz, continuous-wave, frequency-modulated (CW-FM) radar.[9] The radar systems were evaluated for subsurface profiling of the railroad embankments at the WES railroad test embankment and the level test area.

B.1.4 Sonar Test

Sonar subsurface investigation is accomplished by using a sonic source and reflected return signals. Testing and interpretation of results are similar to those for radar. A noncommercial privately developed sonar system used for probing rock salt[9] was at WES in August 1977 in conjunction with the previously mentioned cavity detection project. The sonar system was evaluated on the WES test embankment and level area for possible application to railroad embankment surveying. The system uses 24-kHz sonic pulses.

B.1.5 Nuclear Moisture and Density Tests

Soil moisture and wet unit weight can be determined in place by nuclear methods. Wet unit weight is determined by the attenuation of gamma rays and requires a gamma source and detector. The moisture content of a material is determined by the detected intensity of slow or moderated neutrons and requires a neutron source and detector. This study investigated both surface testing (shallow depth of approximately 1 ft) and borehole logging.

B.1.6 Downhole Geophysical Logging

The WES borehole geophysical logging equipment investigated in this study included the following:[10]

a. Caliper tool for mapping a borehole diameter.

b. Natural gamma tool for determination of wet unit weights and clay contents.

c. Neutron tool for determination of water contents.

d. Resistance tool for resistivity measurements.

e. Self-potential tool for determining naturally occurring electrical potentials.

f. A 3-D acoustic velocity tool for determining P- and S-wave velocities.

g. TV camera for visual observance of the inside of a borehole. The characteristics that are measured by borehole logging have been discussed previously. The tool length is 5.5 ft.

B.1.7 Infrared Survey

Curing of lime slurry emits heat, and lime slurry in soil may alter the thermal emissivity or radiation properties of the soil. Accordingly, temperature measurements were made over the WES sites immediately after injection at various times of the day using a calibrated Wahl-infrared thermometer (accurate to $\pm 1^{\circ}C$) to investigate the feasibility for mapping the lateral extent of injected lime slurry.

B.2 PENETRATION AND STRENGTH TESTS IN THE FIELD

B.2.1 Dutch Cone Penetration

The Dutch cone penetration test[11] is a quasistatic penetration test which is used in subsurface explorations to obtain an estimate of soil strength. Present theories for describing and evaluating measurements are incomplete and complicated due to the many factors affecting the test. Interpretation of the measured cone tip and shaft resistances depends on type of soil failure (shear, compression, grain-crushing, cavity expansion, etc.). These resistances are also influenced by depth, shear strength, compressibility, grain strength, particle orientation, and positive or negative pore pressures. In practice, the test is used via empirical correlations of foundation performance with resistances and as a relative strength measurement which can be used to survey large areas and identify soft materials. The method is fast and sensitivity is improved over that offered by dynamic penetrometers.

In this study, a mechanical penetrometer sometimes referred to as the Discontinuous Dutch Friction Mantle Cone[11] was used to measure end-bearing resistance and side friction. The penetrometer uses a telescoping tip (60-degree cone point angle, base diameter at 1.406 in., and projected area of 1.55 in.^{2}). A friction sleeve is just above the tip and measures the local side friction resistance between the steel and soil. In this study, data were automatically recorded from a transducer measuring the force required to push the tip and friction sleeve. Figure B-5 shows a schematic of the Dutch cone penetrometer.

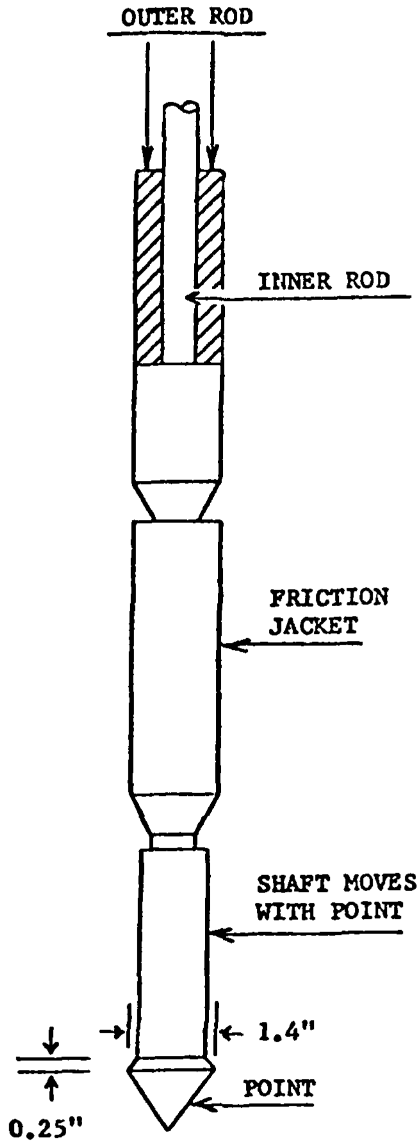


FIGURE B-5. MECHANICAL DUTCH
CONE PENETROMETER

B.2.2 Standard Penetration

The Standard Penetration Test (SPT) is a field test used to obtain disturbed samples of the substrata and provide information regarding the dynamic penetration resistance of the tested materials. In the SPT, a split-spoon sampler is driven 18 in. into the soil with blows from a 140-lb drop weight falling

30 in. Samples thus obtained are disturbed primarily due to the large area ratio of the sampler. The samples are used to provide information regarding the stratification of the soils and for simple laboratory tests which do not require undisturbed samples. Penetration resistance measured in blows per foot is an index which has been used for correlations to strength, density, and compressibility in particular types of soil. The measure of resistance is obtained by counting the blows needed to drive the split-spoon sampler for three consecutive 6-in. increments. Summing the blows from the last two increments gives a number termed the SPT N value. Testing equipment and procedures can significantly affect the results of this test.

In this study, the split-spoon sampler had a 2.0-in. outside diameter and a 1.38-in. inside diameter. It was driven with a 140-lb weight falling 30 in., striking an anvil, and driving a string of rods and sampler. A mechanical trip hammer ensured repeatability of the energy content of every blow. The energy which is input into the system significantly affects the output N values. Drill rods of N-size were consistently used, and all holes were dry-augered with a 6.5-in. hinged auger. The interval of sampling and testing was continuous; i.e., where one drive stopped, the subsequent drive began.

B.2.3 Vane Shear Test

A vane shear test[11] is basically performed by placing a four-bladed vane in undisturbed soil and rotating it from the surface to determine the torsional force required to shear a cylindrical surface of soil that is defined by the diameter of the vane assembly. Two strength tests are usually conducted at each depth: (a) an initial test which measures the peak undisturbed shear strength of the soil, and (b) a test for residual or remolded shear strength that is determined after the soil has been completely sheared.

For this study, the test equipment consisted of inner and outer rod assemblies with the inner rod transmitting the applied torque (measured with a proving ring) to the vane. The vane size is 6 in. high by 3 in. in diameter. With the assembly used in these tests, the interval between test depths is 2.5 ft. Vane insertion is accomplished by predrilling a borehole to a desired depth, placing the inner and outer rod assemblies in the predrilled hole, and then pressing the vane unit to its extended position, which places it 1.5 ft below the bottom of the borehole and outer rod. Due to its mechanical construction, the device used in this study starts testing at a 6-ft depth. Figure B-6 is a schematic of the test equipment.

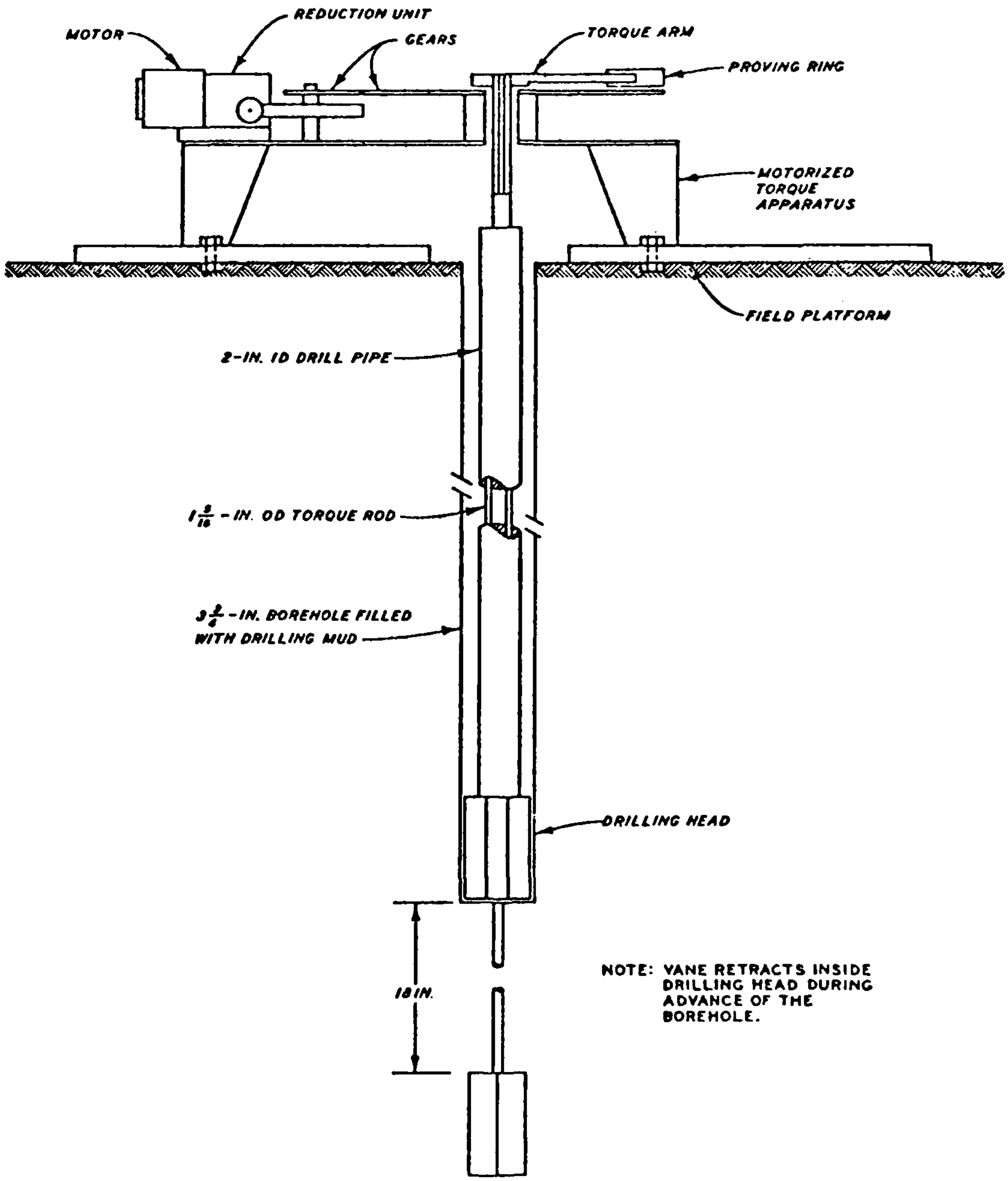


FIGURE B-6. SCHEMATIC DIAGRAM OF FIELD VANE SHEAR IN TESTING POSITION[14]

B.2.4 Pressuremeter Test

A Menard pressuremeter[12] was used in this study and consists of a combination volumeter and manometer connected to a cylindrical device which expands and is the pressuremeter probe. The probe is a steel tube surrounded by two flexible rubber membranes which form two coaxial cells that are inflated. The central cell is designed to apply radial pressure on the wall of a borehole and simultaneously measure the hole diameter increase by the volumeter-manometer registering the liquid flow into the probe.

The pressuremeter probes used in this study have a diameter of 2.64 in. and were used in 3-in.-diam boreholes. Two lengths of probe were used: (a) an NX short probe which is 24 in. long overall with a central measuring cell length of 8.27 in., and (b) an NX long probe which is 36 in. long overall with a central measuring cell length of 18.31 in. Two types of sheaths were used: rubber and metallic. The long probe with a metallic (strips of metal) sheath was used in the granular materials, and the short probe was used in the other soils.

B.2.5 California Bearing Ratio (CBR) Test

The CBR of a soil material is an indication of the strength of the material relative to the strength of a standard material. Essentially, the CBR is the ratio obtained by dividing the unit load required to push a 1.95-in.-diam (3-sq-in.) piston at 0.05 in./min into the material for a depth of 0.1 in., divided by a standard pressure of 1000 psi. The standard push pressure is roughly that required to produce the same piston penetration into a mass of crushed rock. Therefore, the CBR is the strength of the material relative to that of crushed rock. The test is conducted by measuring load in increments up to a penetration of 0.5 in., and the CBR calculated at 0.1 in. penetration.

In this study, CBR tests were conducted on the surface of each embankment layer during construction and on the surface of the level test area. Tests were also conducted at various depths in 3-in.-diam boreholes which are small aperture CBR tests.

B.2.6 Plate Load Test

The plate load tests in this study were cyclic loadings and were conducted using a 30-in.-diam plate on the surfaces of both the embankment and level test area. The 30-in.-diam plate was chosen in order to stress a large mass of material and to acquire a significant depth of influence. On the embankment, tests were on ballast, on crossties, and conducted through the rail heads by loading in the middle of three pieces of 90-lb rail acting as a bridge between the embankment rails. Cyclic tests were conducted by loading, unloading, and reloading to a higher load for several steps. Loads were measured and applied with a calibrated hydraulic loading device jacking against a flat bed semi-trailer loaded with lead weights. Vertical deformations were measured with three dial gages positioned around the periphery of the plate. The loads were held until there was a deformation change of 0.001 in. or less in an elapsed 5-min time period. Cyclic load tests were conducted so that both the elastic (rebound) and inelastic (nonrecoverable) behavior responses could be determined.

B.2.7 Dynamic Stiffness Test

Dynamic stiffness (force/displacement) is determined from the elastic vertical displacements that result from steady-state vibratory loadings. The test is conducted at a selected frequency or a range of selected frequencies. For this study, the tests were conducted with a WES vibrator which has a 16-kip static load plus and minus a 15-kip dynamic load. Peak-to-peak dynamic load can be varied from 0 to 30 kips, and the normal operating frequencies are between 5 to 100 Hz. In this study, dynamic stiffness data were derived from the mechanical impedance tests (discussed below) at the 20-Hz frequency point. The 20-Hz frequency point was chosen because the ground vertical displacements were stable in this range and not near resonance. The vibrator operates electrohydraulically and is housed in a 36-ft semitrailer that contains supporting power supplies and automatic data recording systems. Dynamic measurements are made with load cells for load and with transducers for vertical velocity and acceleration.

B.2.8 Mechanical Impedance Test

Mechanical impedance[13] is a measure of response to a defined input vibratory force. It is defined as the ratio (Z) of an input force (F) to the resulting velocity (V) at the input point and is a measure of the property of a material to resist motion from a given force. The results of an impedance test are spring values or elastic stiffness (force/displacement), participating mass, damping, and resonant frequency. Elastic responses as well as an indicator of inelastic responses (damping) are measured. Figure B-7 shows a plot of impedance versus frequency for a linear elastic damped homogeneous material. Figure B-8 shows the stiffness portion of impedance versus frequency for a system with two linear elastic stiffness elements. A layered system of soil materials could be expected to behave in a manner similar to combinations of Figures B-7 and B-8 except the soil responses would be nonlinear. As an example, stiffness (k) would not follow a constant line but would vary with force and frequency. Therefore, the nonlinear dynamic responses of a soil layered system could be studied and evaluated as functions of varying force and frequency. The objective is to be analogous to the varied load and speed of a moving train.

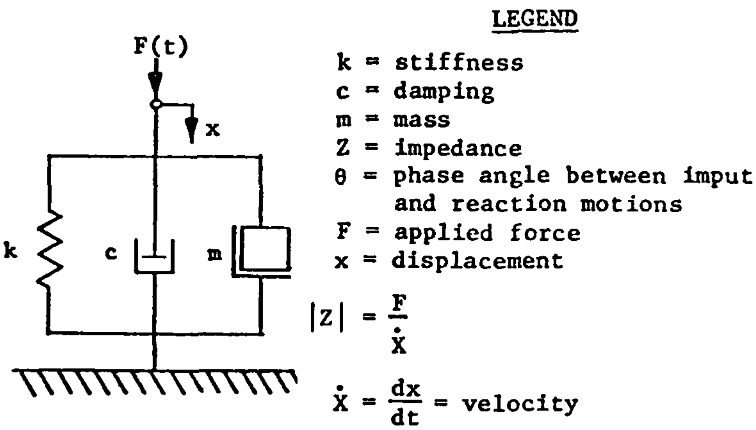
In this study, vertical impedance was determined by using the above-mentioned WES vibrator in varying dynamic load increments and sweeping frequency. The vibratory truck equipment automatically conducts tests, reduces data, and graphically plots the impedance test results in the field. From input measurements of vertical load, velocity, and acceleration, the equipment automatically reduces the data for each constant force level specified and plots the following:

a. Frequency (which ranges between 5 and 100 Hz) versus phase angle between the railroad structure reaction motion and the vibrator input driving motion.

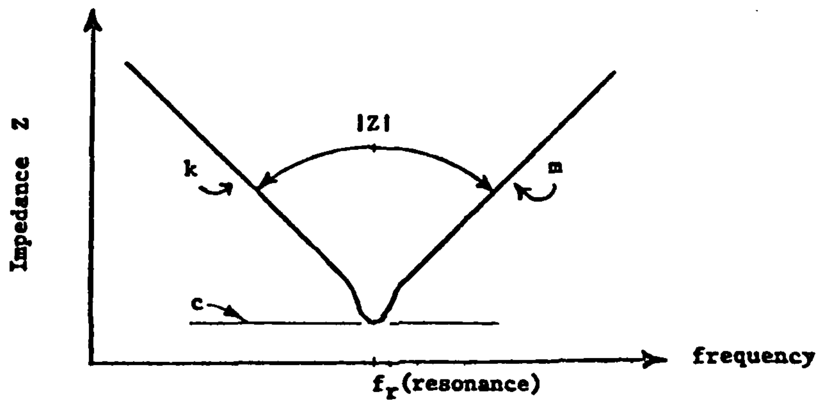
b. Frequency versus vertical deformation of the structure.

c. Frequency versus mechanical impedance.

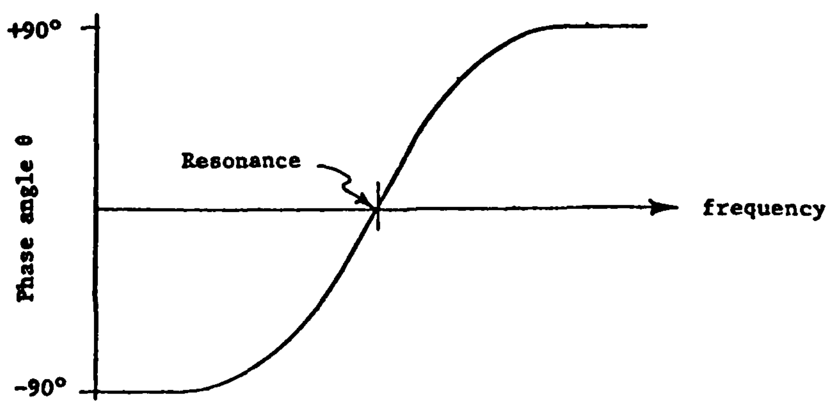
The vibratory truck had high rail wheels installed for conducting the June 1979 RIR tests and for future railroad testing. A special bridge beam (constructed of 2-in. steel plate and having a weight of 1500 lb) with adjustable radiused feet (the radius of a 14-in. rail wheel) for spanning and loading



(a) Parallel three-element linear single degree of freedom system

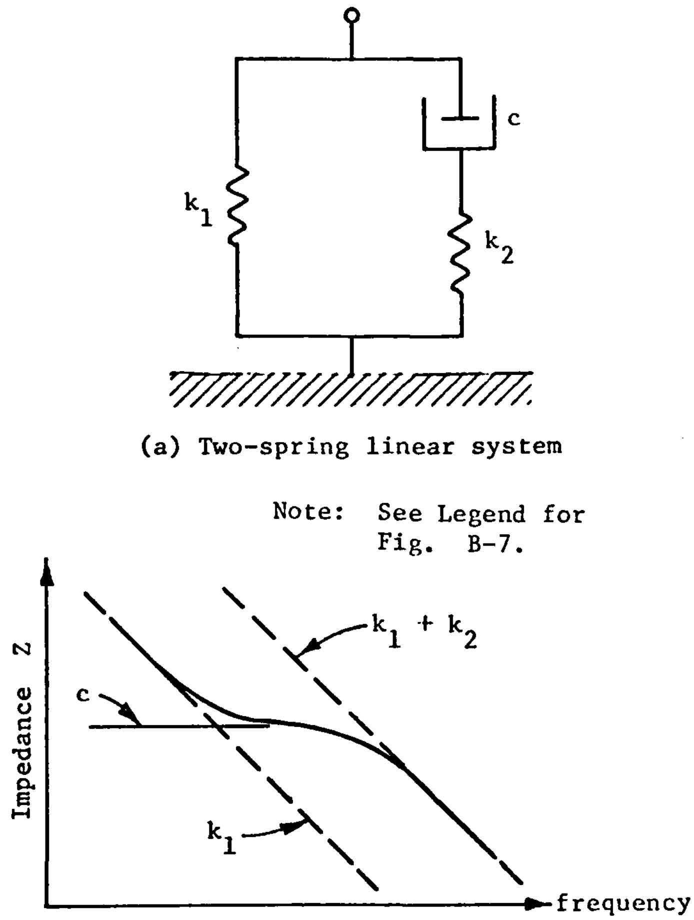


(b) Impedance versus frequency (log-log plot)



(c) Phase angle θ versus frequency

FIGURE B-7. TYPICAL MECHANICAL IMPEDANCE FOR A DAMPED LINEARLY ELASTIC MATERIAL



(a) Two-spring linear system

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

FIGURE B-8. MECHANICAL IMPEDANCE FOR A DAMPED TWO SPRING LINEAR ELASTIC SYSTEM

rail heads was fabricated for the RIR and future railroad impedance tests. Vibratory tests in 1977 and 1978 on the WES embankment were conducted through the rail heads by loading in the center of three pieces of 90-lb rail spanning the embankment rails. Tests were also conducted on ballast and crossties by loading through a 3-in.-diam plate, which was chosen in order to stress a large mass of material and to acquire a significant depth of influence.

B.3 LABORATORY SOIL TESTS

B.3.1 Soil Samples

Soil samples were obtained from the WES test sites prior to and after the first and second lime slurry injections. A 5-in.-ID, thin-wall, fixed-piston sampler was used.[14] The sampler moves with respect to a piston within it only during the actual push; this creates a vacuum that helps suspend the sample in the tube and retains it after the push is made. Samples were extruded in the field, logged, and photographed. The samples were then sealed in cardboard containers following the procedures of EM 1110-2-1907.[14] Preinjection soil samples were stored in the WES soil storage humid room.

Postinjection soil samples were stored in a cool room at 35°F in order to stop the lime-soil chemical reaction (temperatures below 40°F stop the reaction[15] so that laboratory test results could be determined under the same conditions at which the field tests were conducted.

Preinjection and postinjection soil samples from the RIR test sections were acquired in 1976 by the University of Arkansas under a different FRA contract.[2] The samples were taken with 3-in.-ID Shelby tubes, extracted in the field, wrapped in cellophane and aluminum foil, and stored in a laboratory at normal room temperature at the University of Arkansas campus in Little Rock. Only undisturbed soil samples of the subgrade materials were obtained. The quality of these soil samples is doubtful. Moisture changes should be suspected because the soil samples were not waxed. Also, it is doubtful that the postinjection soil samples are representative of conditions that existed when field tests were conducted, because chemical reaction and strength changes will have occurred due to the soil samples being stored at room temperature. Soil sample storage at room temperature will cause the lime chemical reaction to continue at a faster rate and to be quite different from the field chemical reaction which should have been very slow to negligible during the winter months.

B.3.2 Atterberg Limits

Atterberg limits are the index properties of a soil as it changes state from a coherent solid to a plastic solid and to a liquid state. Further description and test procedures are found in EM 1110-2-1906.[16] Lime-modified soils exhibit changes in the Atterberg limits and index properties.[17]

B.3.3 Shear Strength and Parameters

One of the most important strength properties of a soil is its shearing resistance or shear strength which resists stresses that tend to cause shear deformations and influences bearing capacity and stability. The soil shear strength is a function of the cohesion (C) and friction angle (ϕ) parameters,[16] and these parameters are affected in lime-modified soil.[17] The shear strength parameters were determined in this study from consolidated drained laboratory triaxial shear tests.

B.3.4 Volumetric Change Characteristics

Soil volumetric change behavior was determined by laboratory triaxial hydrostatic consolidation and drained stress-path controlled moving load approximation tests.[6] Volumetric behavior influences the deformations and stability of soils and can be affected in lime-modified soils. Triaxial test descriptions and procedures for consolidation are in EM 1110-2-1906.[16]

B.3.5 Moisture Content and Density

Moisture content and density test procedures are in EM 1110-2-1906.[16] The injection of lime slurry will change the moisture content depending on how well the slurry and/or liquid phase disperses into the soil.

B.3.6 Elastic and Inelastic Behavior

Elastic and inelastic deformation-load response characteristics of the soil may be affected by lime. In this study, soil elastic and inelastic deformation-load response characteristics were investigated in laboratory triaxial drained stress-path controlled tests.[6]

B.3.7 Reactivity (Pozzolanic Behavior)

The chemical reactivity of the soils to lime was investigated according to the pH determination test.[1] A pH of about 12.4 must be achieved in lime-soil mixtures in order for the cation exchange and pozzolanic reactions to take place which cause a strength increase to take place.

B.3.8 Chemical Analysis for Lime Content

Samples for chemical analysis were taken from the undisturbed soil samples. The length of samples ranged from 12 to 18 in.; therefore, the lime content determinations are average values representative of the 12- to 18-in.-thick zones from which the samples were obtained. Each sample was thoroughly mixed and dried at 105°C overnight and then ground to pass a No. 50 sieve, and thoroughly mixed portions were then taken for lime content determinations. The portions were digested in a mixture of HCl for 1 hour and filtered. The filtrate was then analyzed for lime for an atomic absorption spectrophotometer. Percent lime was calculated based on dried weight.

APPENDIX C
RAYLEIGH WAVE VELOCITY ANALYSIS

In this study, two sources were used for Rayleigh wave (R-wave) propagation: (a) the 30-kip peak-to-peak semitrailer housed vibrator for frequencies between 5 and 80 Hz, and (b) a 50-lb electromagnetic vibrator operating between 30 and 300 Hz. Common practice is to postulate that the product of frequency and wavelength approximates the shear wave (S-wave) velocity at a depth of one half wavelength. From tests conducted on the surface, R-wave velocities at one half wavelength depths in the WES test embankment did not compare directly to the crosshole S-wave velocities (considered as the true in situ shear wave velocities). The comparison of R-wave and crosshole S-wave velocities was the worst in the relatively low strength clayey silt layer where the R-wave velocity was 60 to 100 percent higher than the crosshole S-wave velocity.

Additional R-wave tests were conducted in order to verify the data. Methods used were steady-state vibrations (commonly used), pulsed vibrations, and locations of wavelengths by moving a geophone along the surface. All test results verified the original R-wave data. An analytical solution for R-wave transmission in layered systems[18] was applied to the data with no success for explaining the results. The analytical solution produced results that were only slightly different from the one half wavelength postulation. It should be pointed out that no detailed comparisons of crosshole S-wave and surface R-wave velocities have previously been made at sites where inverted velocity layering (such as a railroad embankment) exists.

Successful correlations were developed between the R-wave and crosshole S-wave velocities by considering the R-wave velocity that would be measured at the surface by taking the weighted average of the crosshole S-wave velocities in the depth equal to a full wavelength of a given R-wave test (most of the energy is concentrated in a zone about one wavelength deep[19]). (This assumes S-wave velocity is approximately equal to the R-wave propagation velocity.) For increasing R-wave lengths, the weighted averages were computed by considering the velocity effects of the additional depth of material that a surface R-wave would pass through as the wavelengths were increased. The velocity contribution of each layer in a given depth equal to R-wave length was weighted by layer thickness, and the average propagation velocity (which is the value measured at the surface) was calculated for the total depth being considered. In other words, the change in R-wave velocity measured at the surface is assumed to be caused by the wave propagation velocity of the additional material at depth that a wave passes through when the wavelength is increased and vice versa for decreasing wavelengths. Also, the R-wave velocity measured at the surface is assumed to be the average for the combination of materials that the corresponding wavelength propagates through.

These above assumptions are expressed in the following equation:

$$\lambda_1 V_1 + (\lambda_2 - \lambda_1)V_2 = \lambda_2 V_{avg}$$

where

λ_1 = a measured or calculated R-wave wavelength at a given surface vibratory source frequency. Also, the depth of the propagating R-wave

V_1 = the average surface measured or calculated R-wave velocity corresponding to the wavelength λ_1

λ_2 = the measured or calculated R-wave length from the surface vibratory source at a frequency subsequent to that of λ_1. Also, the depth of the propagating R-wave subsequent to that of λ_1

V_2 = the average R-wave propagation velocity within the additional depth $(\lambda_2 - \lambda_1)$ of material passed through by the λ_2 wavelength

V_{avg} = the R-wave velocity measured at the surface corresponding to λ_2 that is a function of the additional depth $(\lambda_2 - \lambda_1)$ and the velocities V_1 and V_2

For the WES embankment, the crosshole S-wave velocities and layer depths were used to solve for V_{avg} in the above equation. The V_{avg} values corresponded closely to the surface measured R-wave velocities at wavelengths equal to the layer depths. Based on this study, the R-wave velocities determined in the surface tests are the velocity weighted averages for the materials within the approximate depths of the full R-wave wavelengths (which are varied by varying frequency of the vibratory source). Given velocity and wavelength values from a normal R-wave survey, one can start at the surface zone, or first velocity (V_1) and iteratively solve for the velocity V_2 of each additional material thickness ($\lambda_2 - \lambda_1$) required to give the weighted average surface velocity (V_{avg}) corresponding to each R-wave wavelength.

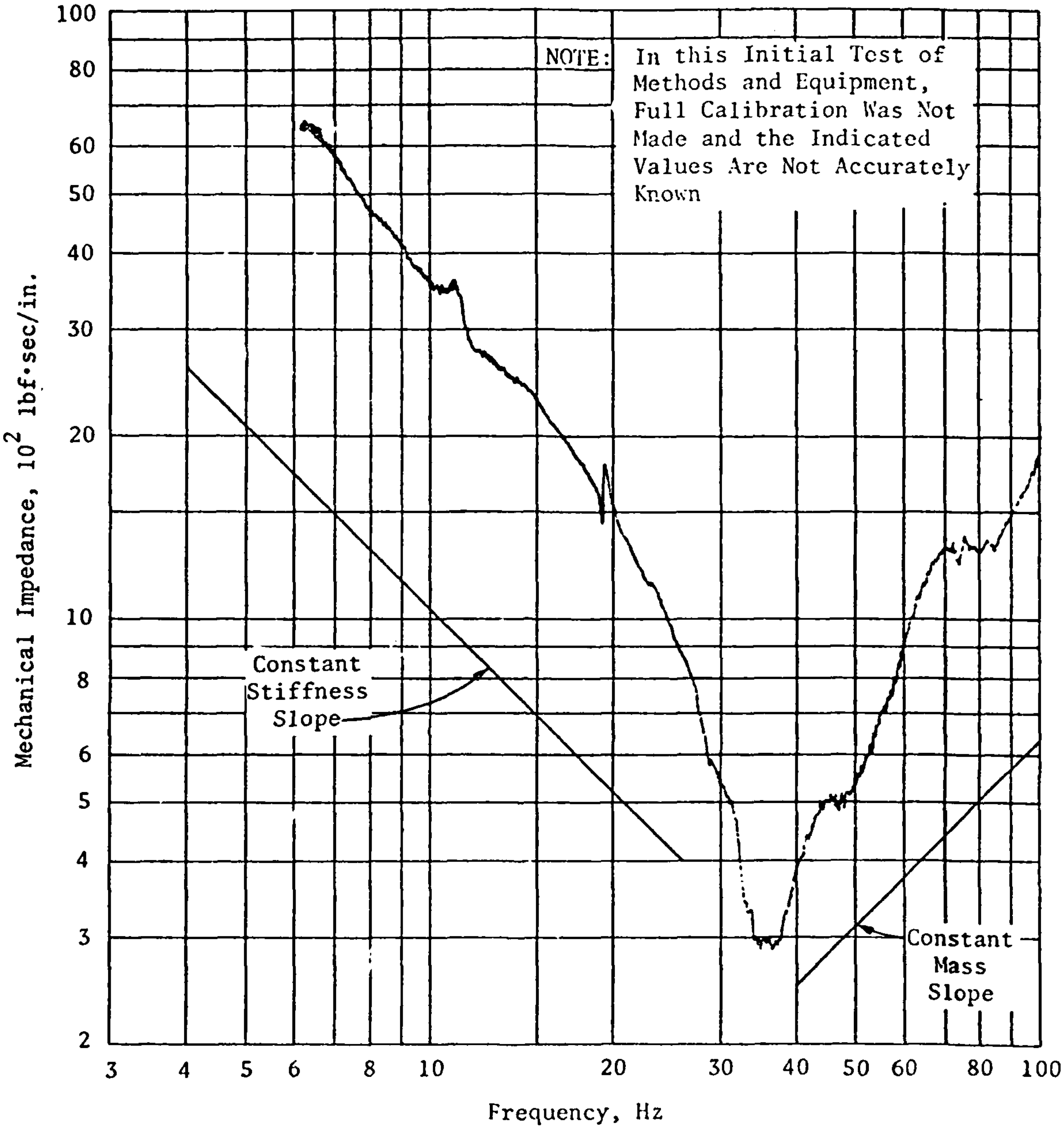
For the WES embankment, using the R-wave velocity and wavelength data in solving the above equation for the V_2 velocities within material thicknesses ($\lambda_2 - \lambda_1$) gave better comparisons to the crosshole S-wave velocity profile, the Dutch cone results profile, and the standard penetration results profile. The above equation was used to analyze and develop R-wave velocity profiles for all of the test sites investigated in this study.

APPENDIX D
MECHANICAL IMPEDANCE IN GEOTECHNICAL APPLICATIONS

The mechanical impedance test as applied to geotechnical investigations offers in situ determination of the dynamic characteristics as compared to those of a fictitious linear single degree of freedom system which has the same response to vertical harmonic excitation as does the combination of ground and superstructure on which the test is run.

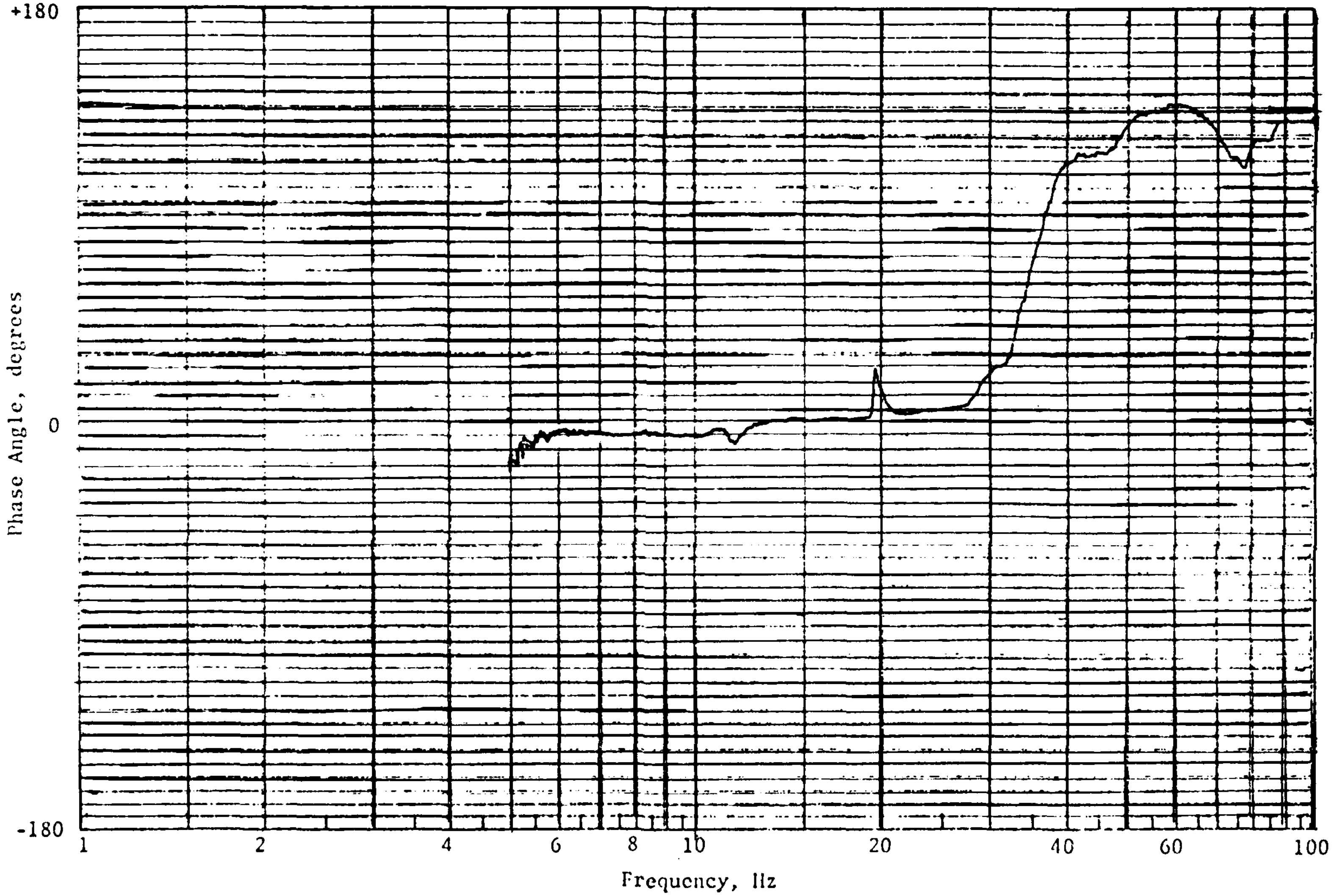
In the forced harmonic vibration of a single degree of freedom system, input motion and reaction motion are in phase until resonance, at which point the input motion shifts 180 degrees out of phase with the reaction motion. For a vibrator which is not rigidly coupled to a body being excited when resonance is reached, the motion of the body becomes out of phase with the vibrator. In the present study, the WES semitrailer housed vibrator was not mechanically attached to the ground or rails. At resonance of the ground or rail-embankment system, the vibrator had to be stopped because the loss of contact and subsequent impact that result from out of phase motions between the vibrator and ground are mechanically detrimental to the vibrator. Therefore, the impedance tests were only conducted up to the point where motions of this type were becoming excessive.

In an initial test on the WES embankment rail heads, conditions of load, frequency, and motions were such to allow passage through resonance without large separations and severe impacts. This only occurred once, all other tests could not be taken completely through resonance. Figure D-1 shows the data from this first impedance test. (In this initial investigation of methods and equipment, full calibration of equipment and components was not made and the indicated values are not accurately known.) This test was chosen for the following discussions because it illustrates the mechanical impedance results with complete passage through resonance. As shown in Figure D-1b, the phase angle passed through 90 degrees at the same frequency that the impedance reached its minimum in Figure D-1a (this phase shift verifies that the impedance low peak is at resonance). Also seen in Figure D-1b, the motions did not



a. Mechanical Impedance

FIGURE D-1. MECHANICAL IMPEDANCE TEST RESULTS AT THE 2-KIP SINGLE PEAK DYNAMIC LOAD THROUGH THE RAIL HEADS FOR PREINJECTION CONDITION EQUIPMENT CHECK ON THE WES TEST EMBANKMENT (SHEET 1 OF 2)



b. Phase Angle Between Input and Reaction Motion

FIGURE D-1. (SHEET 2 OF 2)

become a full 180 degrees out of phase which is indicating that the rail-soil system is not truly simulated by the linear single degree of freedom model.

As seen in Figure D-1a, the impedance curve prior to resonance does not follow a constant stiffness (k) slope nor does it follow a constant mass (m) slope after resonance. This means the rail-soil system does not behave in the same manner as an ideal single degree of freedom system with changing frequency of loading but behaves as a nonlinear function of frequency. The impedance curve describes the nonlinear relation of the system response to changing frequency of loading. The impedance curve also shows that under the indicated 2000-lb peak dynamic load, the rail-soil system has the highest amplitude of vertical motion at a loading frequency about 35 Hz.

By observing the perturbations, shifts, and slope changes along the impedance curve in Figure D-1a and knowing that the test embankment is composed of distinct layers of different strength materials, a hypothesis can be formed that the specific influence of distinct layers is being manifested as frequency is swept in the test. (The phase plot of Figure D-1b shows perturbations corresponding to those of Figure D-1a.)

Each distinct layer has stiffness and damping characteristics that are not only dependent on the strength of the layer but are also dependent on the layer depth which imposes the in situ state of stress caused by overburden and confinement. Also for a surface dynamic loading, the induced stress intensity on any given layer at depth is a function of the rise time, frequency, and magnitude of the surface load application. As depth to a material or location increases, a surface dynamic load must continue significantly acting for longer periods of time in order to significantly stress a given material at depth. (Stress is transmitted from a surface load to a depth location by the shear strains and deformations of the materials above the location and has a time function. In other words, induced stress at depth is not instantaneous with application of surface load but takes a period of elapsed time for build up.) An example of the above is measured decrease of induced pressures at depth as the speed increases of a surface moving load, which is analogous to rise time and frequency, such as in the results of Ledbetter[20]. Therefore, for a

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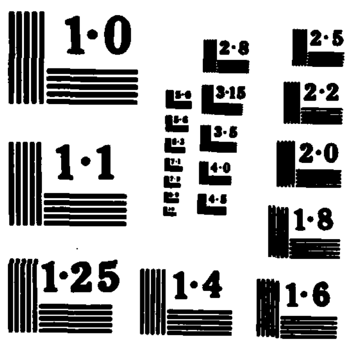


NATIONAL BUREAU OF STANDARDS
MICROCOPY RESOLUTION TEST CHART

given surface dynamic load, the depth of influence can be controlled by the frequency of loading with low frequency causing deep depth stress influence and high frequency causing shallow depth stress influence in the ground.

It now becomes practical to hypothesize that as frequency of dynamic loading is swept and for resulting motion measured at the surface, the contributing influence of distinct material layers or zones causes changes along the impedance curve. In other words, as dynamic loading frequency is swept increasing and the depth of induced stress influence is decreasing, a material layer with distinctly different dynamic chracteristics from a layer above it will eventually become beyond the depth of dynamic load influence and the impedance then changes to that of the system without the particular layer. If dynamic loading frequency is swept decreasing, the depth of induced stress influence is increasing, and as a material layer becomes within the depth of dynamic load influence, the impedance changes to that of the system including the new layer.

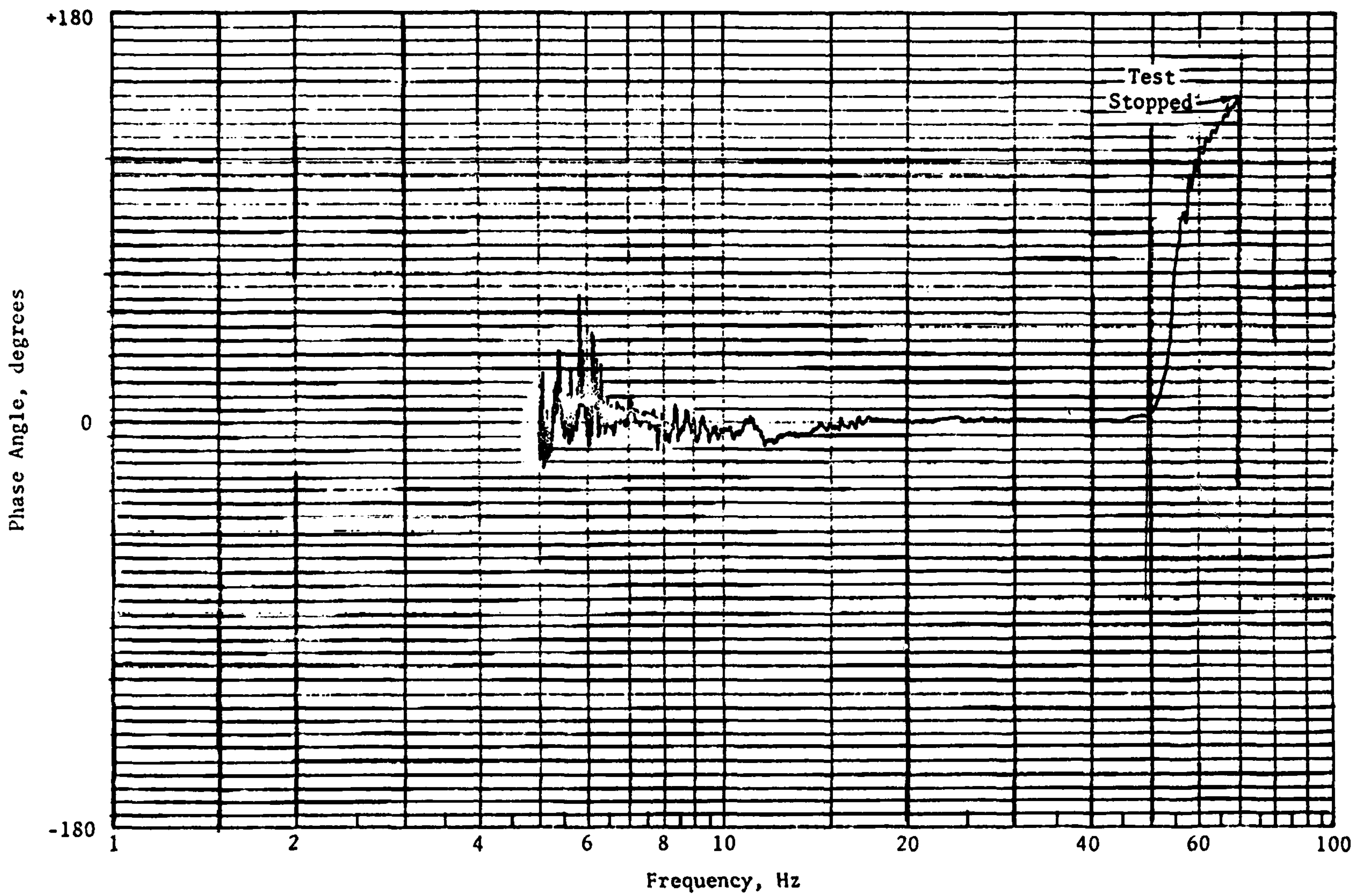
Dynamic load magnitude and effects must be included with the above hypothesis. In this study, frequency sweeps were made at constant dynamic load levels. The single peak dynamic load levels were 500, 1000, 1500, 2000 lb, and increasing in 2000 lb increments to as high as the embankment rail system deformations would allow considering the limits of the test equipment. Dynamic loads did not exceed 15,000 lb, because at 16,000 lb the dynamic force exceeds the static mass of the vibrator. Increasing the dynamic load increases the stress intensity at every depth. As dynamic load levels were increased, the resonance frequency of the embankment-rail system decreased and occurred each time near the previous perturbations (in decreasing order). This means that, for each dynamic load level as frequency was swept increasing and depth of induced stress influence decreased, a depth of stress influence was reached within which the dynamic load intensity and motions were sufficient for that participating mass (composed of different layers) to resonate. Each increase in dynamic load level had an associated lower resonance frequency and, according to the above hypothesis, a deeper induced stress influence which consequently involved a larger participating mass of layers. This is consistent with dynamic theory in which resonance frequency decreases with increasing participating mass.[19]

The phase plots in Figure D-2 verify (phase passes through 90 degrees) the resonances of the WES embankment tests and the fact that they decrease with increasing dynamic load. Figures D-1b is the 2000-lb peak dynamic load test phase plot for the series in Figure D-2. In Figures D-2d, e, and f, the phase plot is broken close to resonance because the tests were stopped to avoid damage to the apparatus and then continued with frequencies above resonance. Figure D-3 presents the downward stroke peak vertical deformations versus frequency corresponding to the same tests as in Figures D-1 and D-2.

The fact that resonances occur as a function of load magnitude and near the impedance perturbations may mean that (in relation to the above hypothesis) each combination of layers in a participating mass is trying to attain resonance prior to the bottom layer of the combination becoming beyond the depth of dynamic load influence as frequency is swept. Indications are that full resonance at the impedance perturbations other than the one that fully resonates may not be occurring because of insufficient stress intensity and motion.

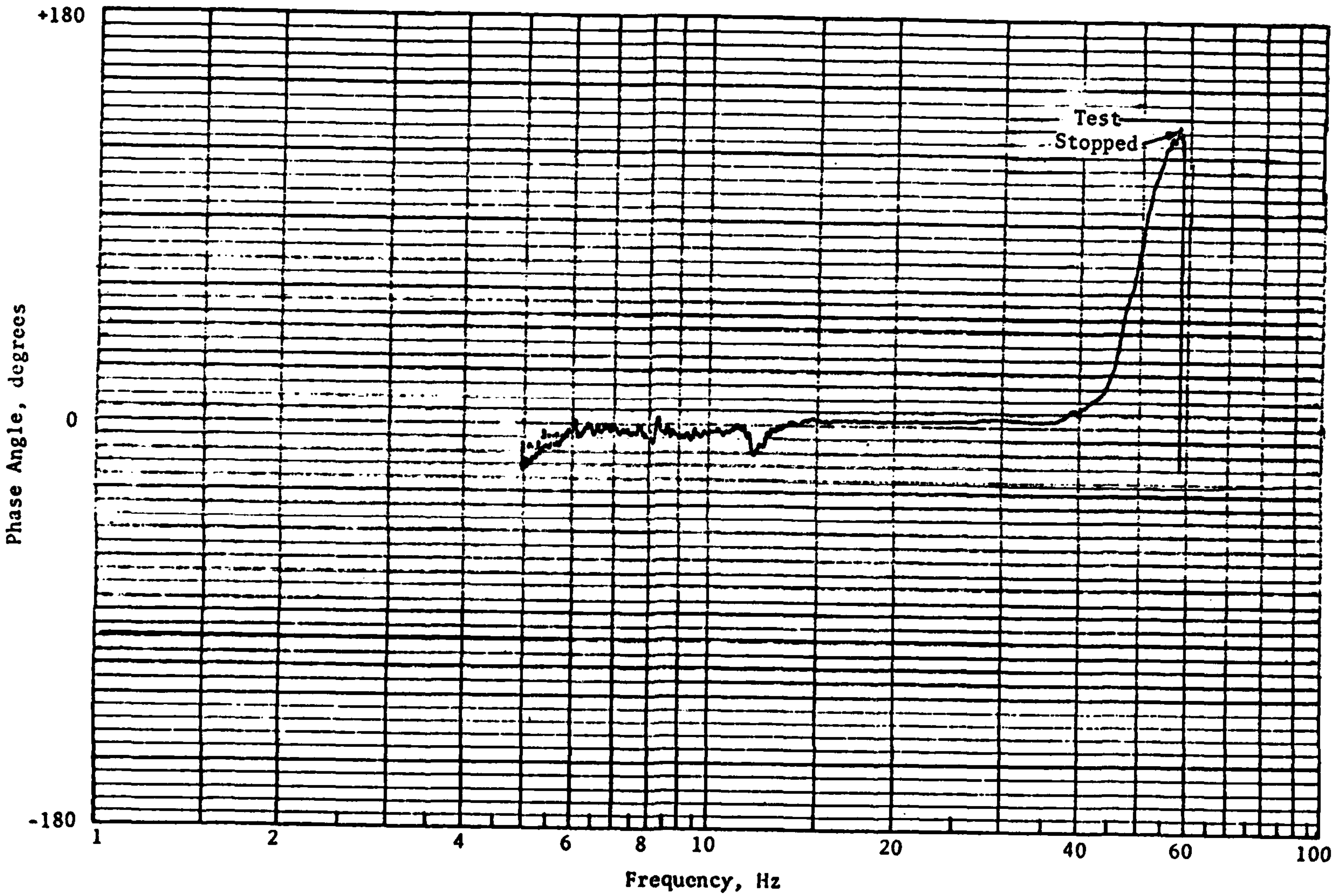
If the rail-soil system tested in this study behaved linearly (constant k and m) with respect to dynamic load, the complete impedance curve would not shift but would remain constant as dynamic load magnitude was changed. However, the complete impedance curves did shift to less stiffness with each additional dynamic load level, which means the rail-soil systems were not linear elastic with increasing dynamic load. A family of impedance curves develop which are describing the nonlinear relation of system response to increasing dynamic load. Figure D-4 shows this for a postinjection impedance test on the WES embankment rail heads. Figure D-4 is also typical of results for the RIR.

In consideration of the above hypothesis and discussion, changes in the dynamic response of a layer or layers within an embankment due to lime slurry injection stabilization should cause preinjection and postinjection shifts along an impedance curve (stiffness changes) at the locations of the influences of the stabilized layers. As discussed in the main body of this report in Sections 5.2.3.8 and 5.2.2.1, no major dynamic stiffness changes occurred in the WES embankment test results. Therefore, no significant changes or shifts occurred in the WES embankment mechanical impedance curves from preinjection to postinjection which would have tended to substantiate the above hypothesis.



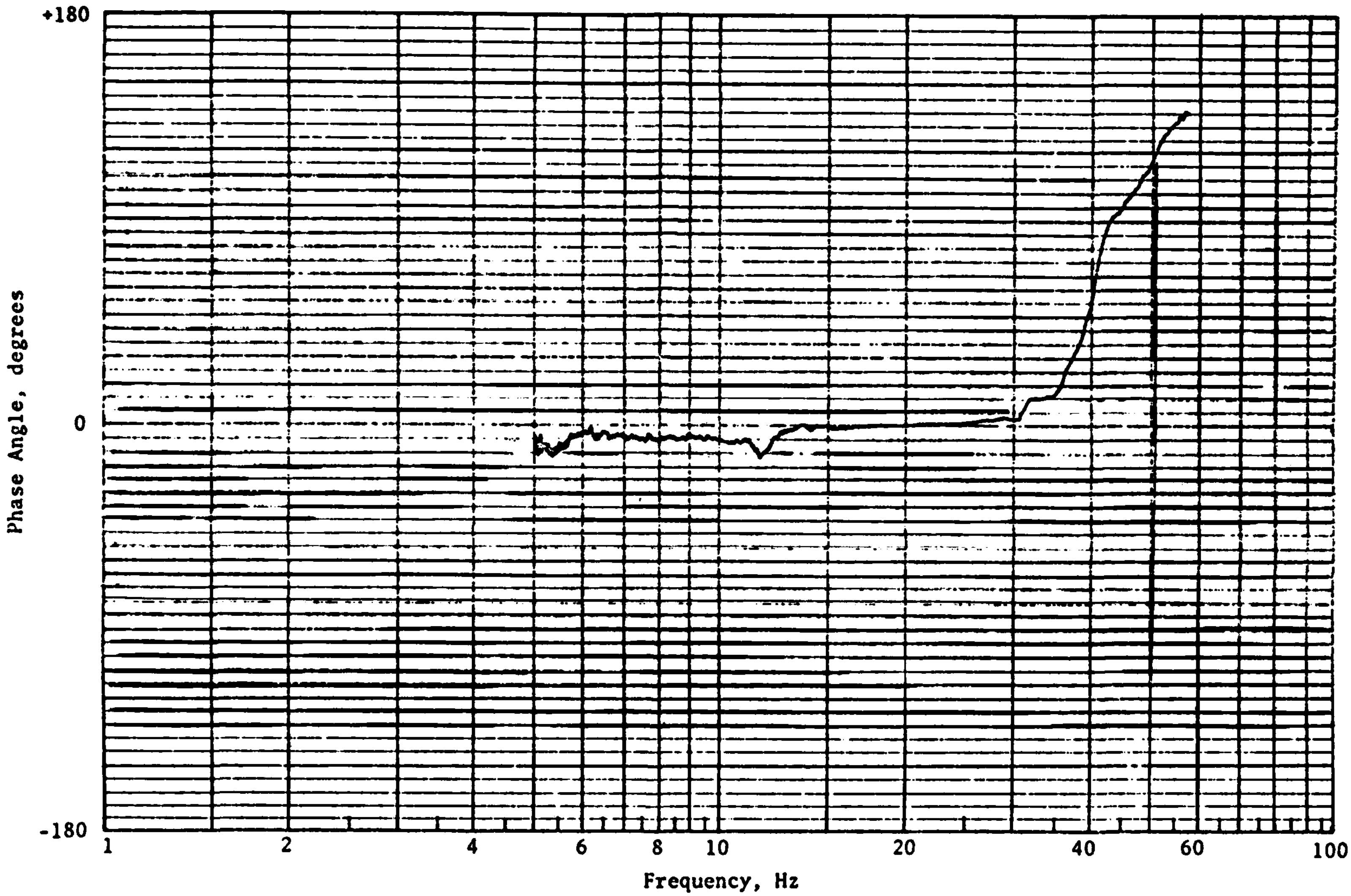
a. 0.5-kip Single Peak Dynamic Load

FIGURE D-2. PHASE ANGLE PLOTS FOR THE PREINJECTION CONDITION MECHANICAL IMPEDANCE TEST EQUIPMENT CHECK LOADING TROUGH THE RAIL HEADS ON THE WES TEST EMBANKMENT (SHEET 1 OF 6)



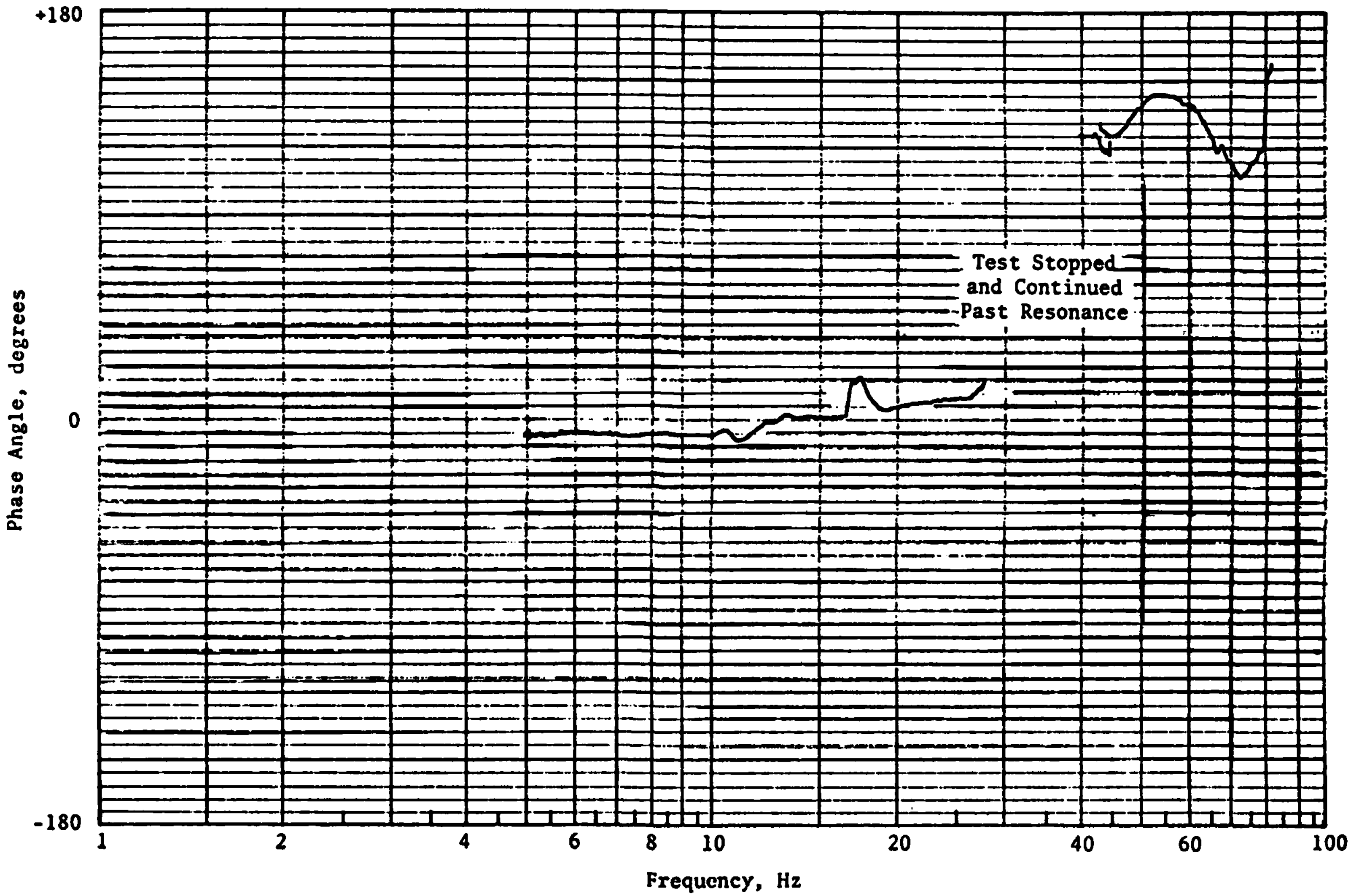
b. 1.0-kip Single Peak Dynamic Load

FIGURE D-2. (SHEET 2 OF 6)



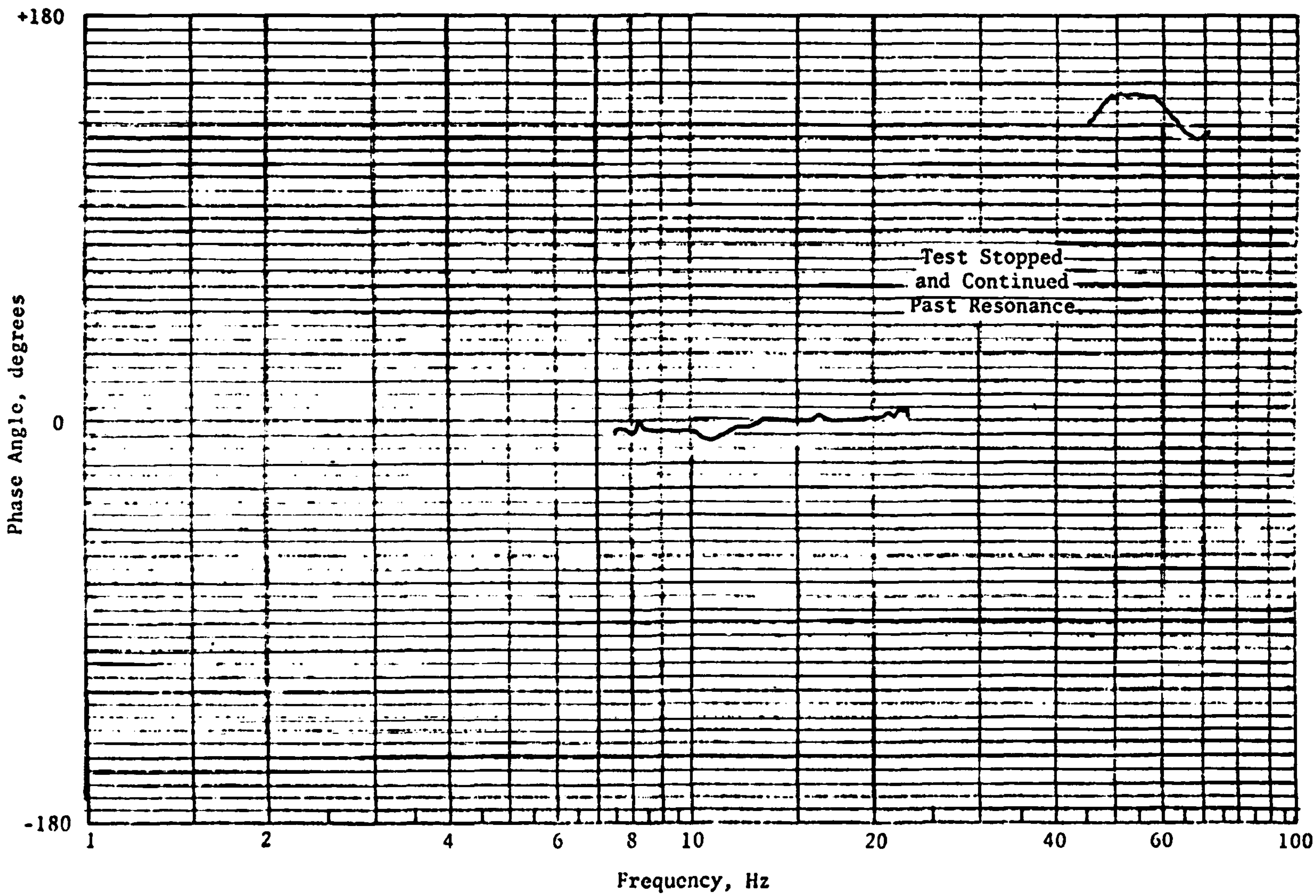
c. 1.5-kip Single Peak Dynamic Load

FIGURE D-2. (SHEET 3 OF 6)



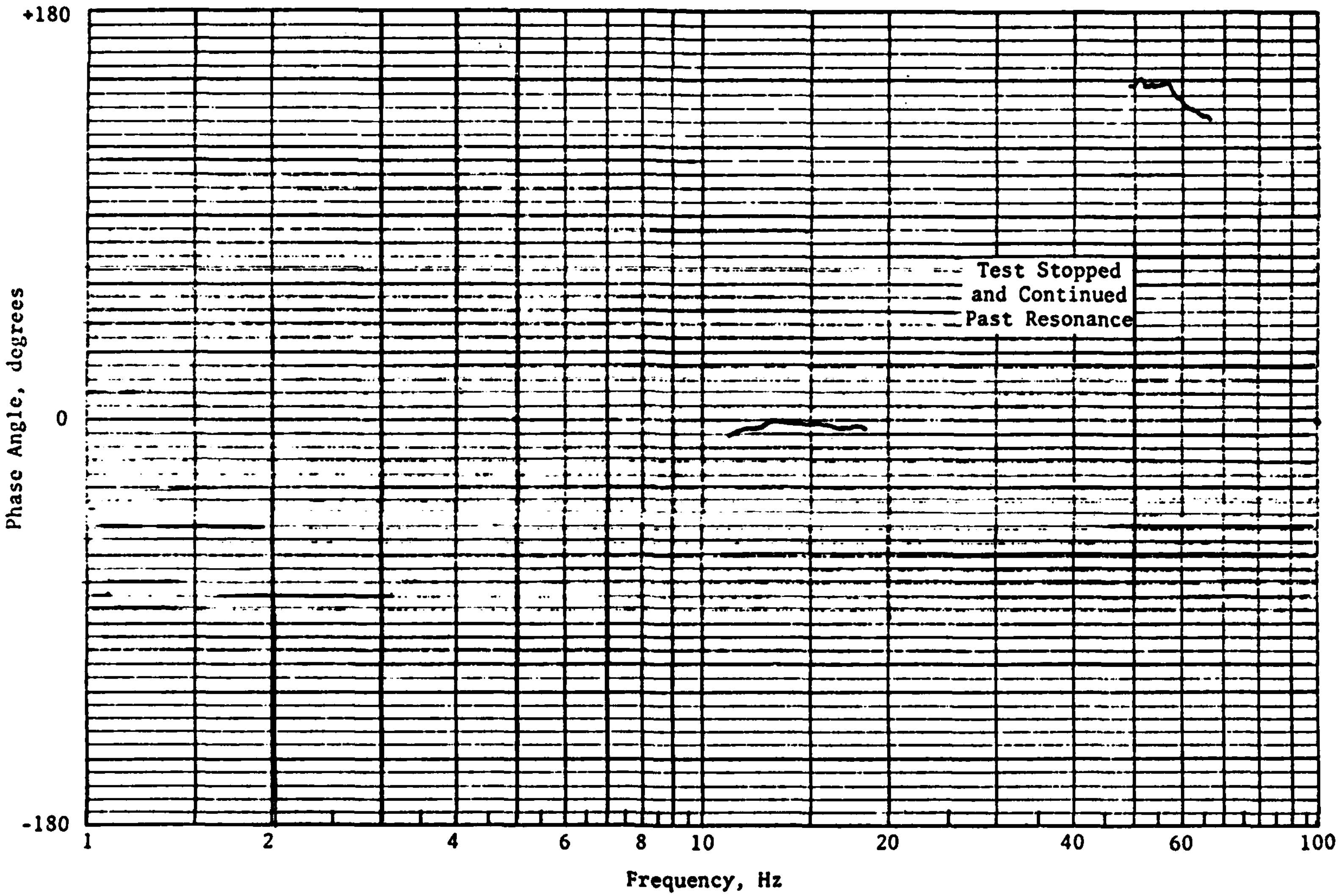
d. 4.0-kip Single Peak Dynamic Load

FIGURE D-2. (SHEET 4 OF 6)



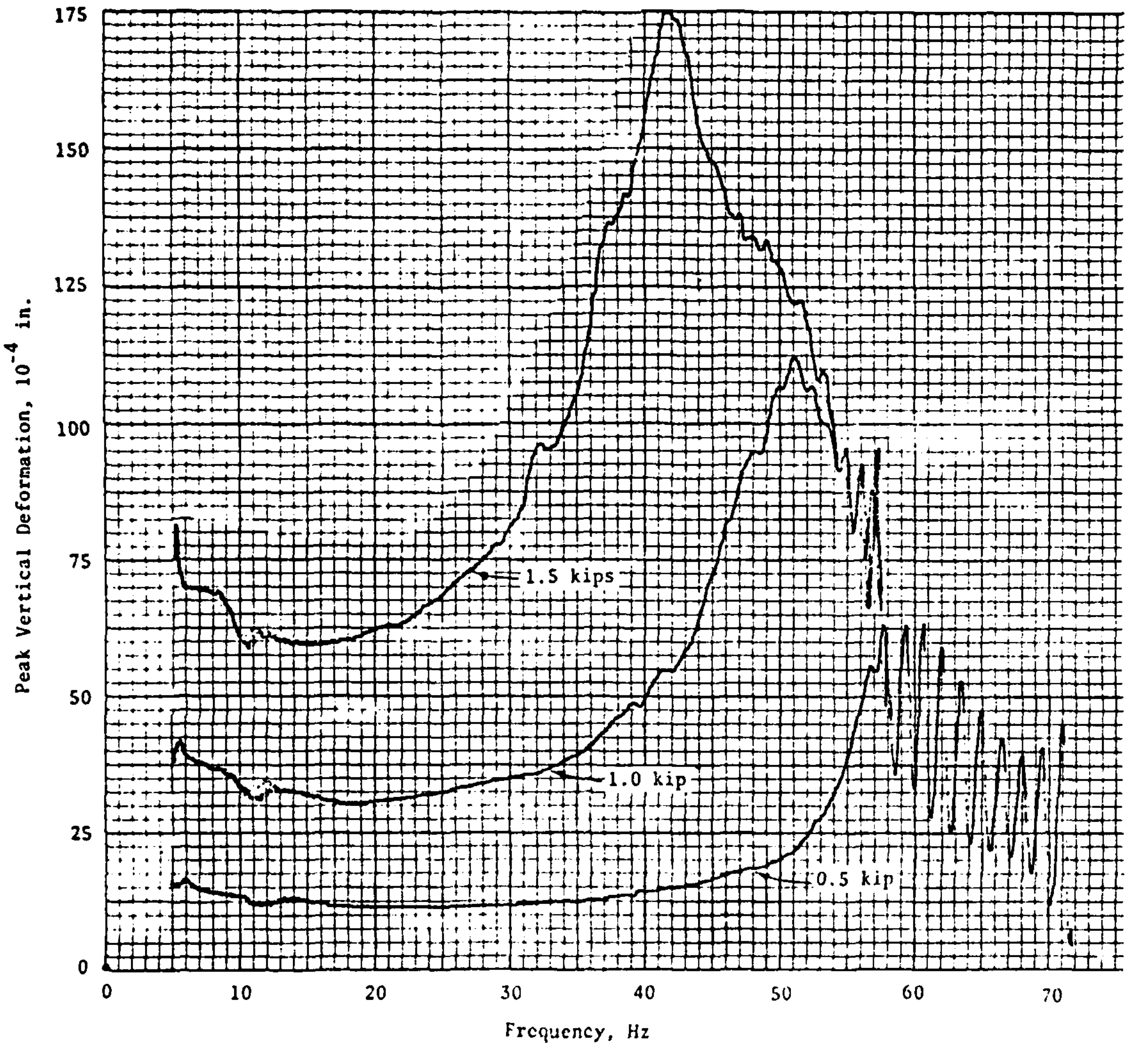
e. 6.0-kip Single Peak Dynamic Load

FIGURE D-2. (SHEET 5 OF 6)



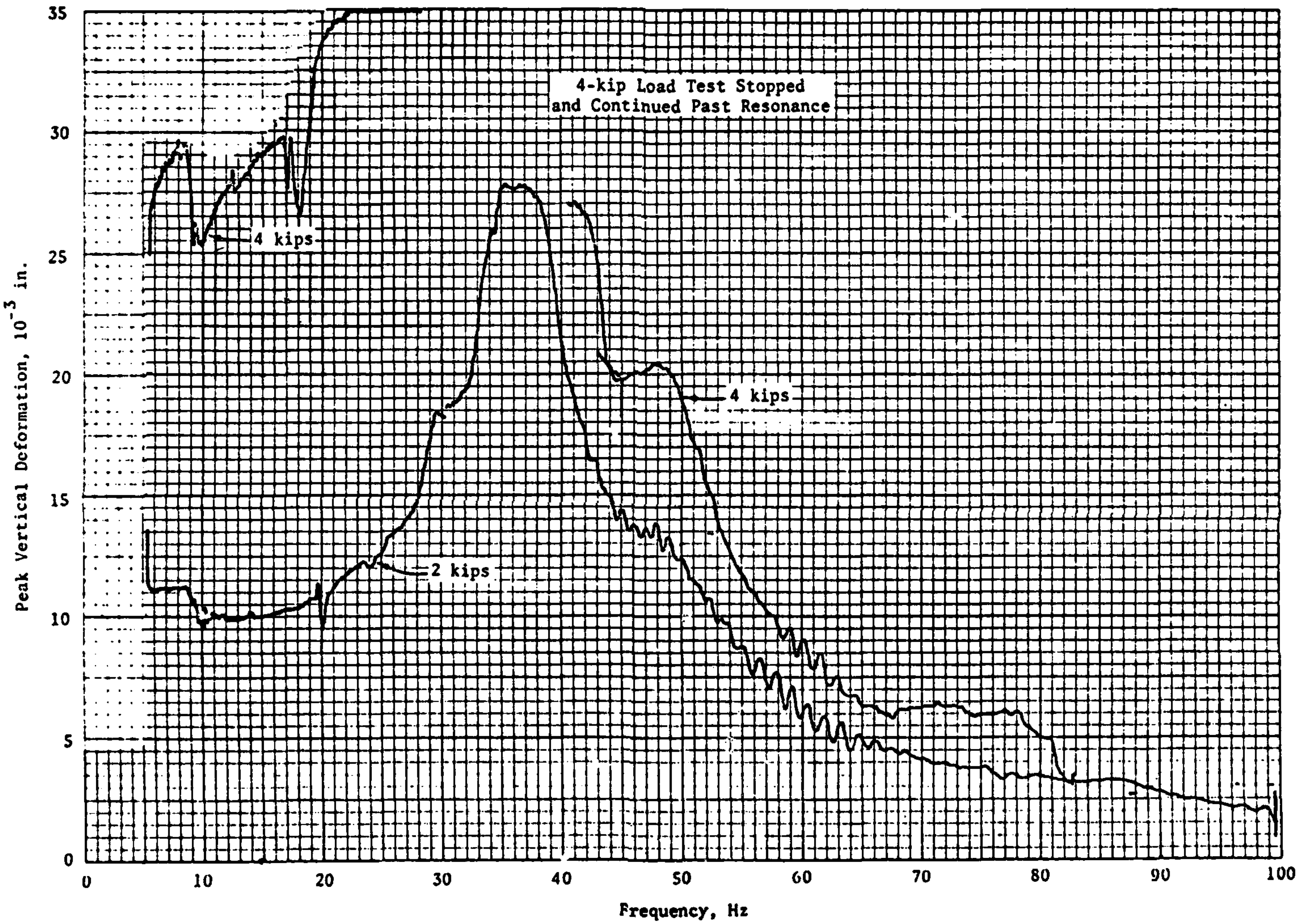
f. 8-kip Single Peak Dynamic Load

FIGURE D-2. (SHEET 6 OF 6)



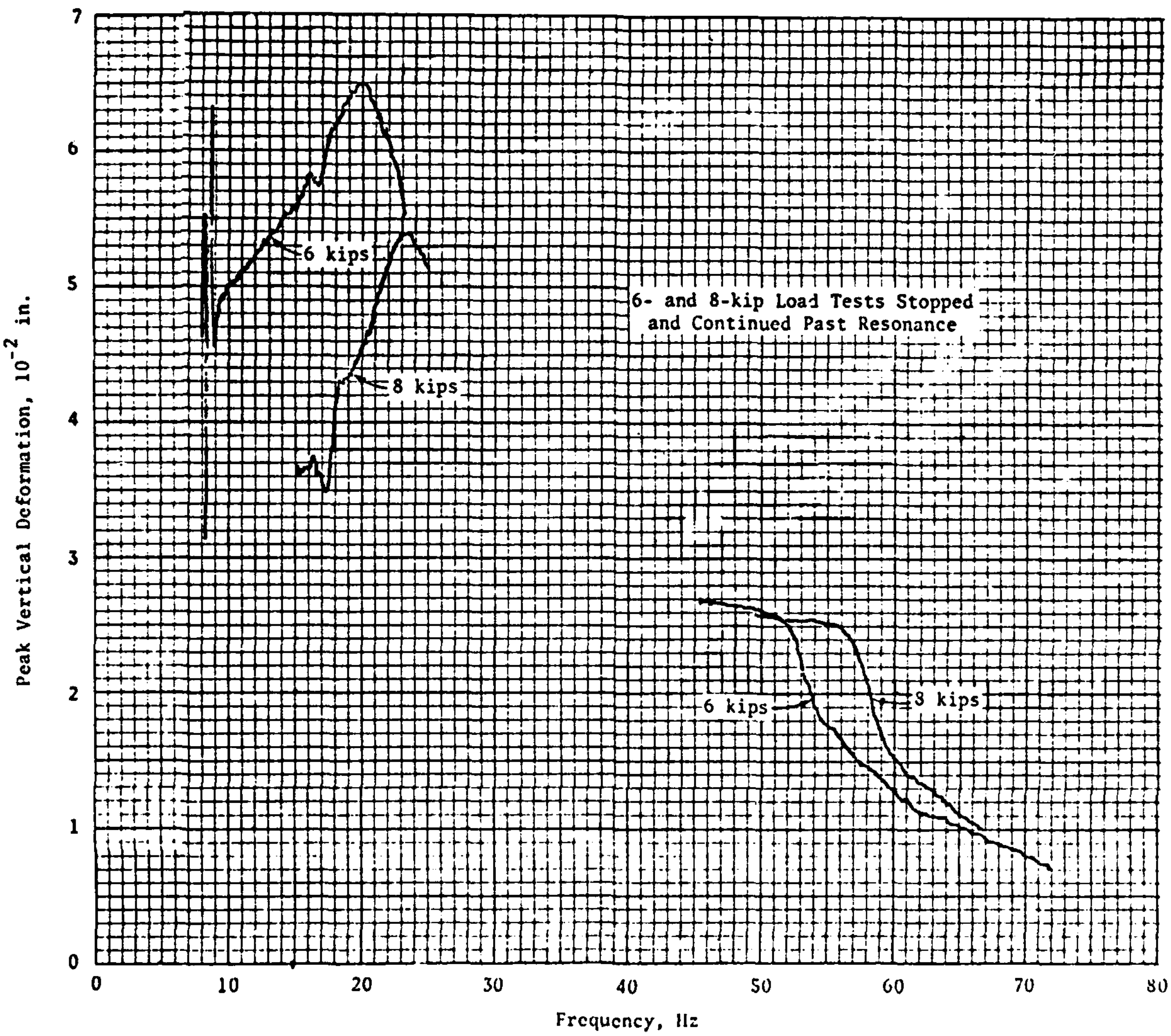
a. 0.5- to 1.5-kip Single Peak Dynamic Load

FIGURE D-3. VERTICAL DYNAMIC DEFORMATION FOR THE PREINJECTION CONDITION MECHANICAL IMPEDANCE TEST EQUIPMENT CHECK LOADING THROUGH THE RAIL HEADS ON THE WES TEST EMBANKMENT (SHEET 1 OF 3)



b. 2.0- to 4.0-kip Single Peak Dynamic Load

FIGURE D-3. (SHEET 2 OF 3)



c. 6.0- to 8.0-kip Single Peak Dynamic Load

FIGURE D-3. (SHEET 3 OF 3)

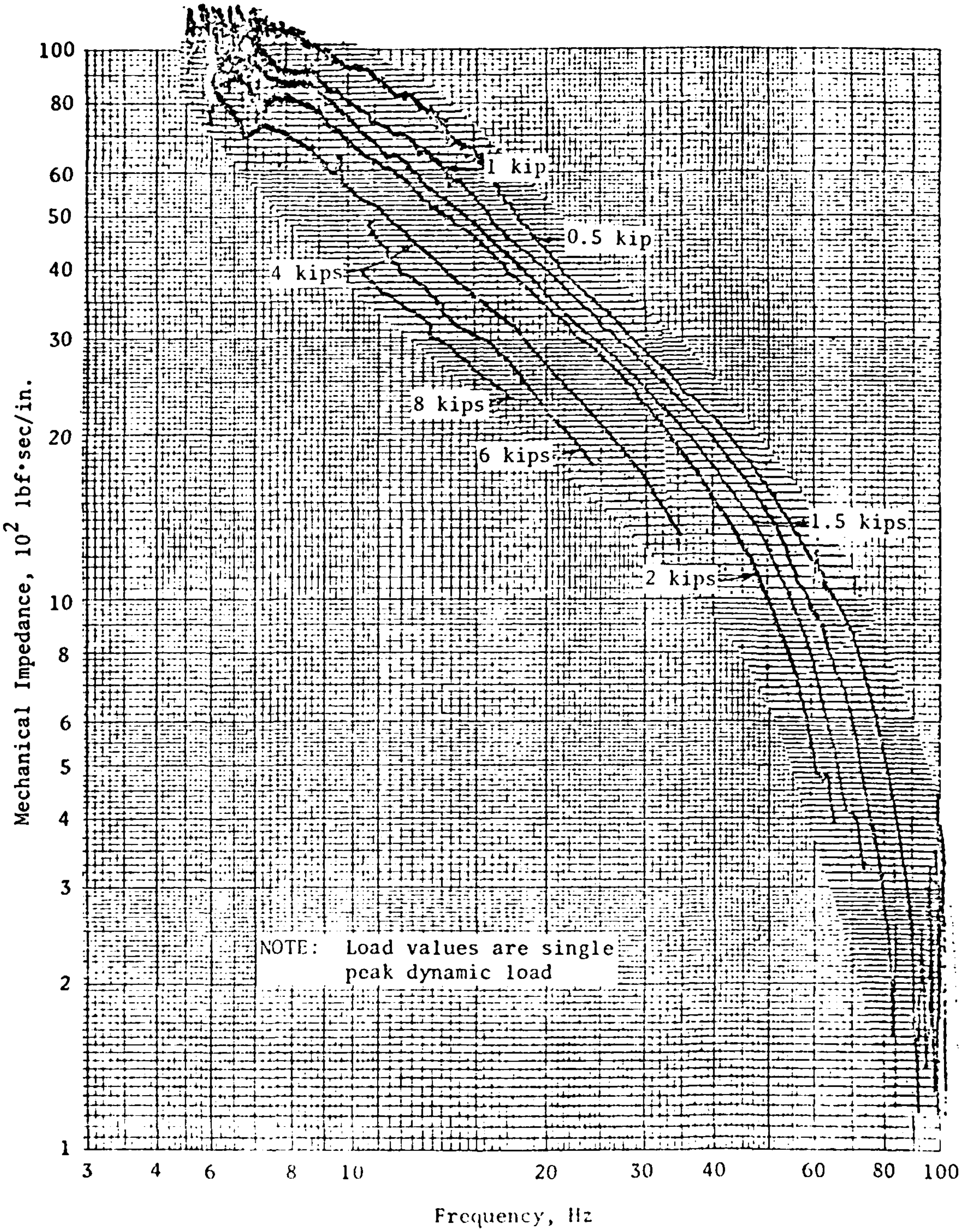


FIGURE D-4. FAMILY OF POSTINJECTION MECHANICAL IMPEDANCE RESULT CURVES FOR LOADING THROUGH THE RAIL HEADS IN THE INJECTED PORTION OF THE WES TEST EMBANKMENT

The RIR dynamic stiffness results did show a significant change between lime slurry treated and untreated sites. Figure D-5 shows an example, from the RIR site B, of mechanical impedance curves at the 2-kip single peak dynamic load level. A curve for a test on rail heads is shown as well as a curve for a test on the crossties. The noise around 5 to 6 Hz is believed due to the vibrator. Seen in Figure D-5 are different overall stiffness patterns between tests on crossties and rail heads. An example of the above hypothesis for mechanical impedance test interpretation is shown in Figure D-5 where point A_R (of the rail head test) is assumed to be resonance for the rails, and the first lower frequency perturbation (B_R) is assumed to be caused by the crossties. As seen on the crossties impedance curve, the point B_T is the assumed resonance of the crossties and is at the perturbation frequency B_R on the rail head curve.

Figure D-6 is an example, from the RIR site B, of rail head test mechanical impedance curves at the 2-kip single peak dynamic load level for lime-treated and untreated areas. In Figure D-6, considering the above mechanical impedance hypothesis, point A is assumed resonance of the rails, point B is the perturbation assumed to be caused by the crossties (discussed above), point C is the perturbation assumed to be caused by the top 1.5 ft of unstabilized material verified in the Dutch cone data, and point D is the perturbation assumed to be caused by the lime-stabilized zone. Around point D is where the curves should shift, which is caused by the higher stiffness stabilized zone. Figure D-7 is for the 12-kip dynamic load level in the same test series as Figure D-6. Figure D-7 better shows the curve shift in the vicinity of point D, with the treated area curve showing higher stiffness. At higher frequencies than point D, the curves come back together for the shallow depth unstabilized material. Point E in Figures D-6 and D-7 is the perturbation assumed to be caused by the unstabilized subgrade, and at frequencies less than point E the curves are parallel. The portions of the curves between points E and D represent the contribution of the material that was stabilized in the treated site. For mechanical impedance test interpretations such as this above, the complete family of curves developed at a test location should be used in the interpretation in order to verify the curve perturbations, shifts, and resonances.

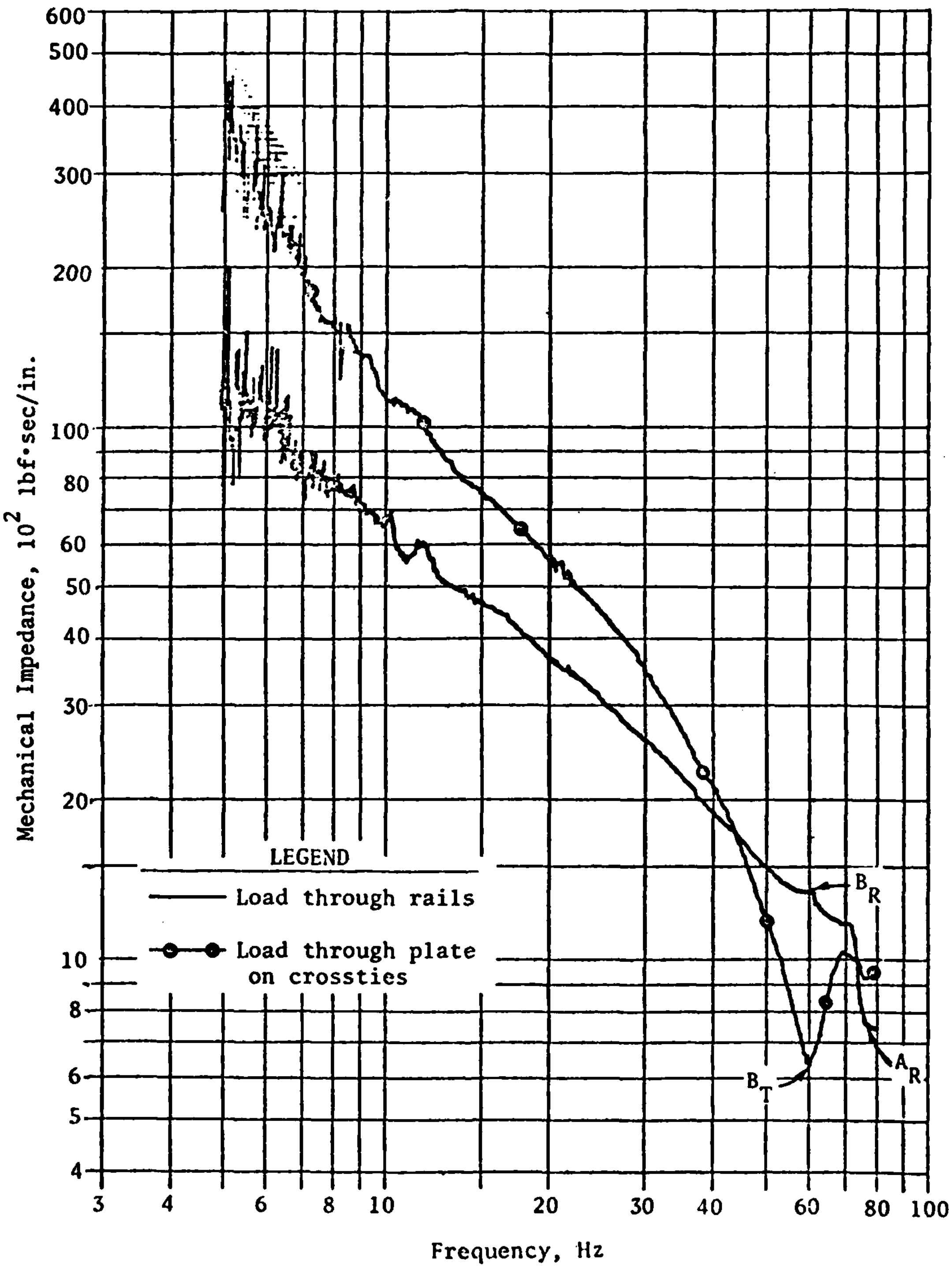


FIGURE D-5. MECHANICAL IMPEDANCE TEST RESULT CURVES FOR THE 2-KIP SINGLE PEAK LOAD THROUGH THE RAIL HEADS AND THROUGH CROSSTIES IN THE INJECTED PORTION OF SITE B OF THE RIR, 1979

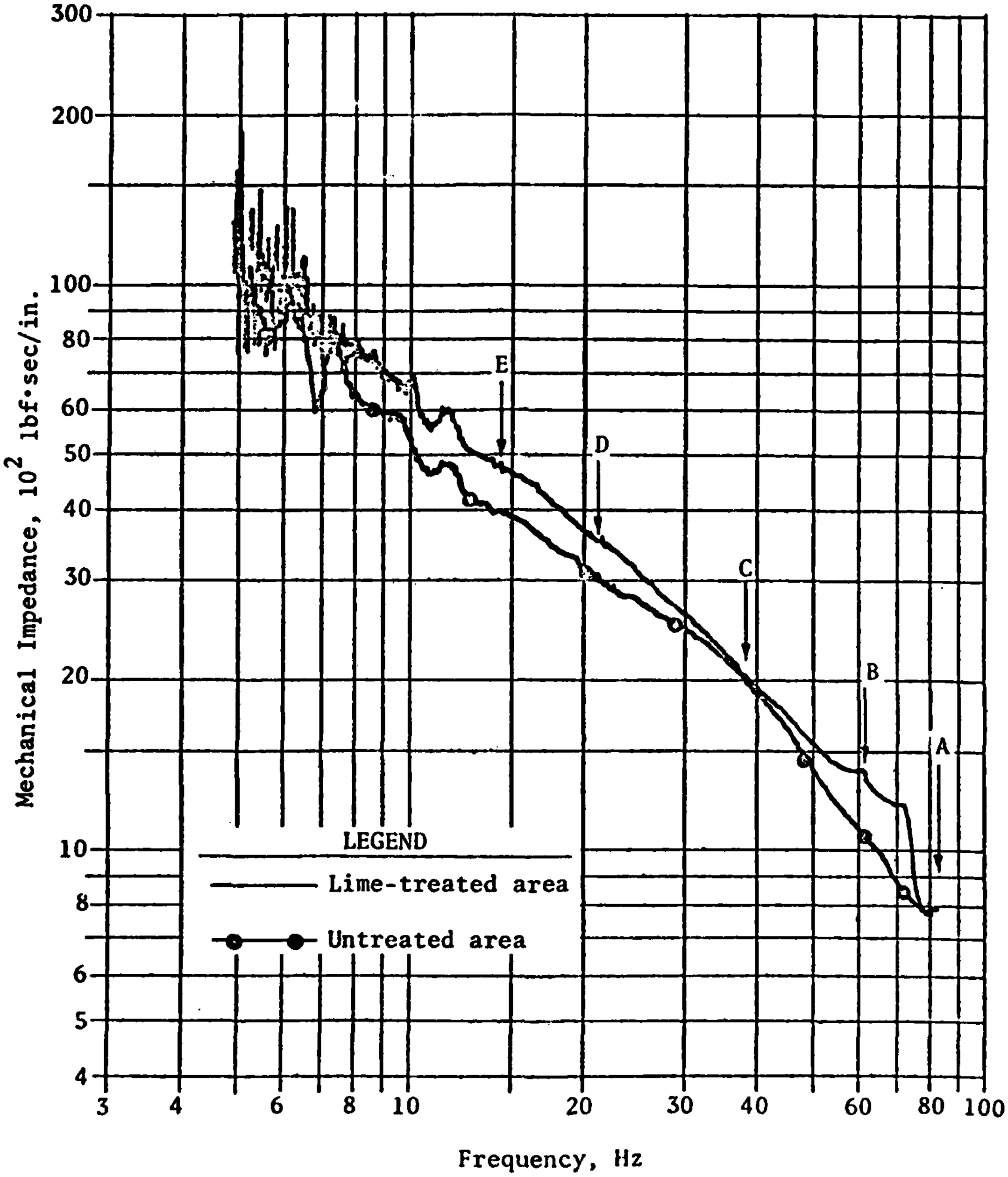


FIGURE D-6. MECHANICAL IMPEDANCE TEST RESULT CURVES FOR THE 2-KIP SINGLE PEAK DYNAMIC LOAD THROUGH THE RAIL HEADS IN THE UNTREATED AND TREATED PORTIONS OF SITE B OF THE RIR, 1979

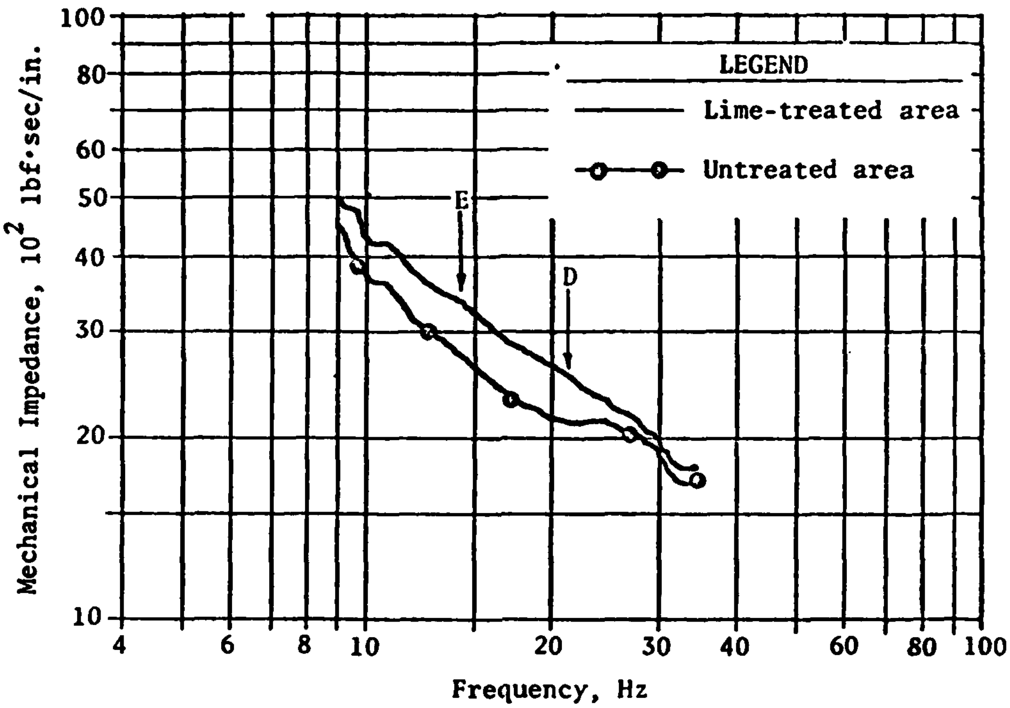


FIGURE D-7. MECHANICAL IMPEDANCE TEST RESULT CURVES FOR THE 12-KIP SINGLE PEAK DYNAMIC LOAD THROUGH THE RAIL HEADS IN THE UNTREATED AND TREATED PORTIONS OF SITE B OF THE RIR, 1979

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