MoDOT

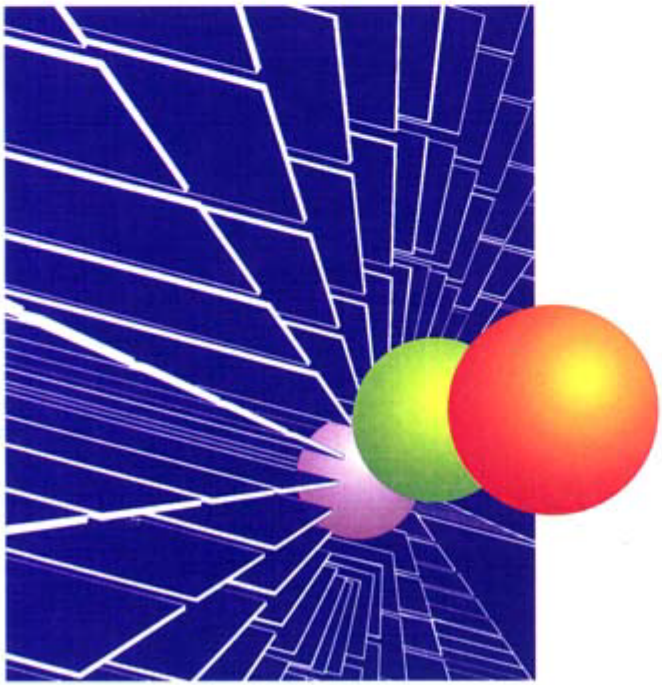
Research, Development and Technology

University of Missouri-Columbia

RDT 00-007

Slope Stabilization Using Recycled Plastic Pins - Constructability

RI 98-007



July, 2000

Final Report

RDT 00-007
Research Investigation 98-007

Slope Stabilization Using Recycled Plastic Pins – Constructability

PREPARED FOR

MISSOURI DEPARTMENT OF TRANSPORTATION
RESEARCH, DEVELOPMENT AND TECHNOLOGY
JEFFERSON CITY, MISSOURI

BY:

J. Erik Loehr
Assistant Professor of Civil Engineering

John. J. Bowders
Associate Professor of Civil Engineering

Hani Salim
Assistant Professor of Civil Engineering

Department of Civil and Environmental Engineering
University of Missouri - Columbia

DATE SUBMITTED: July, 2000

The opinions, findings, and conclusions expressed in this publication are those of the principal investigators and the Missouri Department of Transportation: Research, Development and Technology.

They are not necessarily those of the U.S. Department of Transportation, Federal Highway Administration. This report does not constitute a standard or regulation.

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. RDT 00-007	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Slope Stabilization Using Recycled Plastic Pins – Constructability		5. Report Date July 2000
		6. Performing Organization Code University of Missouri – Columbia
7. Author(s) J. Erik Loehr, John J. Bowders, Hani Salim		8. Performing Organization Report No. RDT 00-007 / RI 98-007
9. Performing Organization Name and Address University of Missouri – Columbia Dept. of Civil and Environmental Engineering; E2509 Engineering Building East Columbia, MO 65211-2200		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Missouri Department of Transportation Research, Development and Technology P. O. Box 270-Jefferson City, MO 65102		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code MoDOT
15. Supplementary Notes The investigation was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.		

16. Abstract

A new technology for stabilizing slopes was developed by the Civil & Environmental Engineering Department at the University of Missouri-Columbia for the Missouri Department of Transportation. The technology uses Recycled Plastic Pins (RPPs) manufactured by pressure-molding recycled polyethylene with a compliment of sawdust and other by-products. The RPPs were manufactured by a Missouri firm. There are 20 to 30 other manufacturers in the United States.

The constructability of the RPP technology was demonstrated by stabilizing two slides located in an embankment located on Interstate 70 at milepost 62 in Missouri. The embankment had a history of repeated slope failures, each of which was repaired by pushing the soil back in place.

For this demonstration, the failed slopes were regraded to their original configuration and reinforced with a total of 317 pins (includes both slopes), or RPPs, installed on a 3-foot grid pattern using a mast-mounted vibratory hammer system. The RPPs were installed in November 1999. The RPPs were driven flush with the ground surface. The rate of installation of the pins averaged approximately 80 feet per hour.

Instrumentation was installed on the RPPs and in the slope to monitor performance. Performance monitoring to date, June 2000, shows the slopes to be stable. A longer monitoring period is required before definitive conclusions on the performance can be made.

The unit cost for installation of the RPP stabilization technology at the Emma, MO site was $3.90/ft^2 – twenty-five percent less than for the traditional technique of using rock armor. The RPP technology offers an additional alternative for stabilization of shallow slope failures. Additional demonstrations and longer performance monitoring periods are warranted to document the range of applicability and validate the design methodology for the RPP stabilization technology.

17. Key Words Surficial slope failures Slope stabilization Reinforced slopes Recycled plastic		18. Distribution Statement No restrictions. This document is available to the public through National Technical Information Center, Springfield, Virginia 22161	
19. Security Classification (of this report) Unclassified	20. Security Classification (of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 (06/98)

Acknowledgements

The authors express their sincerest appreciation to the following individuals and entities for their cooperation and support that assured the success of this project:

- Pat Carr, The Judy Company, Kansas City Missouri,
- Kurt DeRuy, Tamko Composite Products, (formerly DuraBoard), Lamar Missouri,
- Mike McGrath, Area Engineer, District 2 Missouri Department of Transportation,
- Tom Fennessey, Sr. Materials Engineer, Soils & Geology Section, Missouri Department of Transportation,
- Mark Virkler, University of Missouri – Columbia, Director of Mid-America Transportation Consortium for MU,
- Sam Kiger, Chair of Civil & Environmental Engineering, University of Missouri – Columbia,
- The Undergraduate Research Scholars Program, University of Missouri-Columbia, and
- Mr. David Schelp, Emma Market, Emma, Missouri.

Executive Summary

The primary objective of the project entitled "Slope Stabilization Using Recycled Plastic Pins" was to demonstrate the constructability of a stabilization scheme using recycled plastic pins (RPPs) by stabilizing a single slope at a site representative of MoDOT slope failures. The selected demonstration site is the southern side of the eastbound entrance ramp embankment to Interstate 70 near Emma Missouri (Milepost 62). Four slide areas have experienced repeated failures at the site. Two of the slides were stabilized using the RPP technology. The other two slides are being used as control sections. The two reinforced slides and one unreinforced slide were instrumented to measure and document performance. After seven months, all of the slopes have remained stable. Field instrumentation monitoring is ongoing to provide more definitive validation of the stabilization technique.

A limit-state design methodology was developed and used to determine the number, spacing and length of the pins needed to stabilize the slides. The procedure does not account for group or interaction effects. Such behavior may increase the effectiveness of the reinforcement but requires fundamental analyses and testing prior to being incorporated into the design. Critical parameters identified for design are: depth of sliding surface, pin capacity, and pin spacing. Notable findings from preliminary parametric analyses are that soil strength plays a secondary role in the stability of RPP reinforced slopes and that improvement in stability of a slope due to reinforcement is similar whether using recycled plastic ("weak" reinforcement) or steel ("strong" reinforcement) members of the same size and spacing.

Installation required slightly more than 4 days for driving a total of 317 pins into the two slides. During the installation process the RPPs were notably very durable under even hard driving conditions. It was found that driving the pins perpendicular to the slope face was less problematic than driving them vertically. A mast driving system, one that supports both the RPP and the driving hammer, was much more effective, accurate, and required less skill to operate. In order to maintain peak driving efficiency, a guided (mast-type) driving system is required. Average installation rates ranged from 70 to 100 ft/hour using the mast driving system.

A preliminary economic comparison for stabilizing the Emma slides showed costs for the RPP technology ($3.90/ft^2) to be slightly less than estimated costs for traditional rock armor ($5.40/ft^2) and many times less than for traditional soil nailing ($19.00/ft^2).

An extensive evaluation of the engineering and material properties of the recycled plastic was performed concurrent with the field demonstration. The recycled plastic was found to have a peak tensile strength ranging from 1.3 ksi to 1.8 ksi and a peak compressive strength of 3.0 ksi. Peak bending moments were 1.0 kip-ft for a nominal 4-inch by 4-inch cross-section. Coupons of the recycled plastic were exposed for a period of up to a year in an acidic (pH=5) solution, ultraviolet light, tap water, kerosene, and freeze/thaw environments. Only the kerosene environment caused a notable reduction in the strength of the material.

The results of two years of creep testing were used in an Arrhenius model to predict the impact of sustained loading on the RPPs. The findings showed that current field loadings would not cause failure of the RPPs for 1000 years. These results diminish concerns about creep deformations for the slope stability application of RPPs.

Stabilizing slopes with recycled plastic pins shows much promise for an alternative repair method for shallow slope failures. In order to achieve wide ranging acceptance of the technology, several tasks remain including: demonstrating the applicability or range of applications of the technology, validating the design procedure through performance monitoring data, and more completely assessing the economics of the technique.

Table of Contents

Acknowledgements i

Executive Summary ii

Table of Contents iii

List of Figures iv

List of Tables v

Introduction 1

Demonstration Site 1

Site Investigation and Laboratory Testing 1

Installation and Constructability 3

Economics 7

Future Enhancements to Driving System 7

Instrumentation and Monitoring 8

Development of Preliminary Design Procedure 13

Engineering and Material Properties 17
- Tension, Compression, Shear and Bending 17
- Interface Friction 18
- Bending Creep 19

Conclusions 24
- Constructability: 24
- Material Properties: 25
- Design Methodology: 25
- Preliminary Economic Comparison: 25

Recommendations 25

References 27

Principal Investigator and Project Members 28

Appendix A – Emma Site Boring Logs and Subsurface Cross-sections 29

Appendix B – Consolidated-Undrained Triaxial Test Results 48

Appendix C – Measured Bending Moments for Instrumented Pins 51

Appendix D – Slope Inclinometer Data 57

Appendix E – Recycled Plastic Material Properties from Exposure Tests 63

Appendix F – Bending Creep Test Data and Arrhenius Modeling Calculations 67

List of Figures

Figure 1. Plan view of the demonstration site located north of Emma, Missouri at I-70 Milepost 62, with the four slope failures designated as S1 through S4. 2

Figure 2. Layout of plastic pins at the Emma demonstration site for slide areas S1 and S2 showing the varying penetration depths. 4

Figure 3. Initial equipment used for installation of recycled plastic pins at the Emma, Missouri demonstration site. 5

Figure 4. Crawler mounted drilling rig used for installation of recycled plastic pins at the Emma, Missouri demonstration site. 6

Figure 5. Plan view of location of instrumentation for Slide S1 Emma Missouri 8

Figure 6. Plan view of location of instrumentation for Slide S2 Emma Missouri 9

Figure 7. Plan view of location of instrumentation for Slide S3 Emma Missouri 9

Figure 8. RPP showing the location of resistance strain gages for measuring strains on the pins in the field. 10

Figure 9. Typical strain versus depth along instrumented Pin A (mid-slope, slope S2), as of 17 April 2000, located at the Emma, Missouri demonstration site. Note: 1000 microstrains is equal to 0.1 percent strain. 11

Figure 10. Measured bending moment as a function of depth in Pin B, Emma Missouri demonstration site – 17 April 2000. 12

Figure 11. Precipitation data collected from Sweet Springs Missouri weather observation station 3.5 miles east of the Emma Missouri demonstration site. Note: Pins were installed in November 1999. 12

Figure 12. Distribution of limiting lateral resistance per unit length of slope: (a) limiting resistance for the soil, pins, and anchorage and (b) composite limiting resistance. Data shown are for 4-in. x 4-in. x 8-ft. pins on 3-ft. spacing in soil with c=0, ϕ=21 degrees, and γ=110 pcf. 14

Figure 13. Method for computing the limiting pin resistance as a function of depth of sliding: (a) moment distributions due to limiting soil pressure and reduced lateral pressure and (b) limiting soil pressure and reduced limiting soil pressure. 16

Figure 14. Elevation view of tilt table test for interface friction between Emma soil and RPPs. The contact area between the soil and the RPPs was 95 square inches (600 cm^2). . 19

Figure 15. Setup for testing creep behavior of recycled plastic pins under bending loads. L was 20 inches and the pin cross-section was 2 in. x 2 in. 20

Figure 16. Deflection versus time response for RPP loaded with a 50 Lb at the free end of a simple cantilever (Figure 15) under various temperatures from 70 to 176 degrees. With the exception of the specimens at 70 degrees, all specimens failed at the final data point. 21

Figure 17. Typical Arrhenius plot For bending creep test on 2in.x2in.x24in. RPP loaded as a cantilever with a 50 Lb weight at the unsupported end. Plots for other loading conditions are included in the Appendix F. 22

Figure 18. Method for estimating time to failure resulting from bending creep of RPP 23

List of Tables

Table 1. Geotechnical index properties for selected samples from the Emma site. 3
Table 2. Properties of samples for consolidated-undrained type triaxial tests 3
Table 3. Summary of the penetration and installation rates for mast-mounted hammer on slope failures S1 and S2. 6
Table 4. Cost comparison for alternative slope stabilization methods. 7
Table 5. Summary of tension, compression, shear and bending test results on recycled plastic material used in the RPPs. 17
Table 6. Summary of tension and compression properties of recycled plastic material subjected to various exposure environments. 18
Table 7. Conditions and results of the creep bending tests on the RPPs 22

Introduction

The primary objective for the project entitled "Slope Stabilization Using Recycled Plastic Pins" was to demonstrate the constructability of stabilization schemes using recycled plastic pins (RPPs) by stabilizing a single slope at a site representative of Missouri Department of Transportation (MoDOT) slope failures. Secondary objectives of the project included: evaluating basic material and engineering properties of recycled plastic pins when subjected to various potentially deleterious environments; development and evaluation of potential driving equipment; development of a preliminary design procedure; and initiation of an instrumentation and monitoring program for evaluating the effectiveness of the RPP stabilization scheme. The activities and findings of the project are presented in this report. Emphasis is given to constructability of the recycled plastic pin stabilization method.

Demonstration Site

Demonstration site #1 was selected in May 1999. The site is located on the eastbound entrance ramp to Interstate 70 at Emma, Missouri (milepost 62). Four general slide areas were observed in the embankment just prior to the field demonstration as shown in Figure 1. Three slide areas, denoted S1, S2, and S3, are located along the south side of the embankment with an additional slide, denoted S4, on the north side of the embankment. Information acquired from local maintenance personnel indicated that the Emma site slopes had repeatedly failed over a period of a decade or more with failures occurring as frequently as several times each year. Previous slope repair activities included simple regrading of the slopes to their original configuration as well as dumping of concrete rubble over the crest of the slope. All previous stabilizing measures proved unsuccessful. Slide areas S1 and S2 were selected as test areas for installation of RPPs; slide areas S3 and S4 are being used as control sections. The embankment height is nominally 23-ft. (7-m) and the slope varies from 2.5H:1V to 2H:1V.

Site Investigation and Laboratory Testing

The Emma site was drilled and sampled in June 1999 by MoDOT drilling personnel with assistance from University of Missouri – Columbia (MU) researchers. Undisturbed soil samples were recovered in 3-inch diameter Shelby tubes. The samples were extruded from the tubes in the field, wrapped in foil and sealed using hot wax. Samples were then transported to MU Geotechnical Engineering laboratories for testing. Index properties were performed on selected samples throughout the site and consolidated-undrained triaxial tests (with pore water pressure measurements) were performed on specimens from the suspected shear zone.

The site soils were found to be a mixture of lean to fat clays with water contents ranging from 15 to 30 percent and averaging about 24 percent (Appendix A). The soils ranged from soft to stiff. The average liquid limit was 50 and plasticity index was 28 (Table 1) and the soils classified as low plasticity and high plasticity clays. Boring logs and estimated subsurface cross-sections are provided in Appendix A.

Observation of the site conditions and the existing failures at the site indicated that the shear zone was likely to be in the fat clay layer. Specimens for shear strength testing were selected from the suspected shear zone (Table 2). Specimens were trimmed to a diameter of 1.5 inches and a height of 3.0 inches from the 3-inch diameter Shelby tube samples. Consolidated-

undrained triaxial tests were performed to determine the effective shear strength parameters ($\bar{c}$ and $\bar{\phi}$). The specimens were backpressure saturated to degrees of saturation above 95 percent, consolidated to effective stresses of 8 psi and 20 psi respectively, and then sheared at a rate of 3.9 x 10^{-5} in/min (0.001 mm/min). The resulting data (stress difference vs. axial strain and modified Mohr-Coulomb diagram) are provided in Appendix B. The measured effective friction angle was 15 degrees and the effective cohesion was 275 psf.

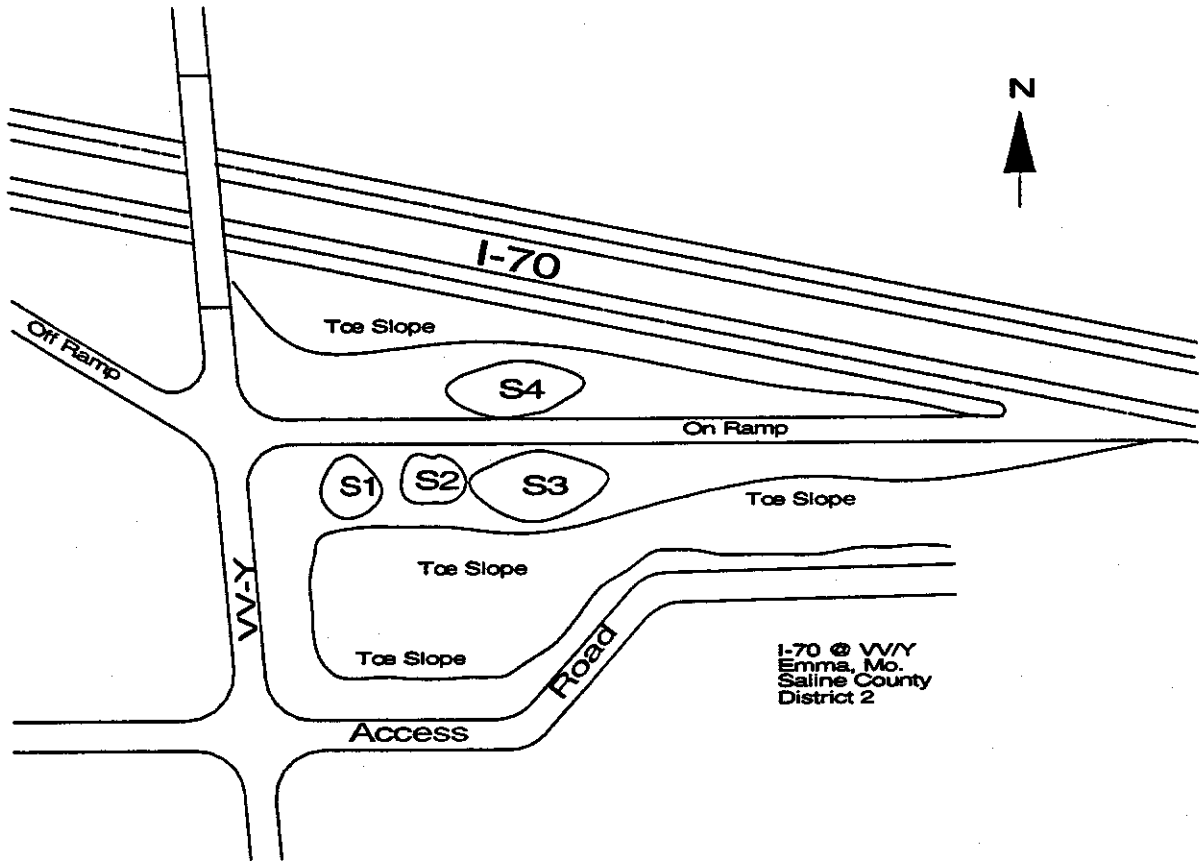


Figure 1. Plan view of the demonstration site located north of Emma, Missouri at I-70 Milepost 62, with the four slope failures designated as S1 through S4.

The shear strength data was used for analyses of the stability of the slopes at the Emma site. The effective friction angle was held constant (15 degrees) and a search was performed for the failure surface with a factor of safety of 1.0 by varying the effective cohesion. A failure surface with a maximum depth of 8 feet was determined for effective stress strength parameters of friction angle at 15 degrees and cohesion at 50 psf. While the cohesion observed from back analyses is not identical to that measured on the test specimens, it is in reasonable agreement. Observation of the previous slides at the site indicates that the deepest part of the failure surface could be about 8 feet below the surface. It is also noted that the embankment soils are stratified and heterogeneous in nature ranging from low to high plasticity. The strength is likely to vary similarly. Thus, while the laboratory measured strengths are likely to be from the slide shear zone, they may not be representative of the strengths everywhere along the sliding surface, but are concluded to be reasonably representative of the soils at the Emma site.

Table 1. Geotechnical index properties for selected samples from the Emma site.

Boring Location	Depth ft	Depth m	Liquid Limit (%)	Plastic Limit (%)	Plastic Index	USCS Class.
MU 17	2.5	0.76	50	22	28	CL
MU 16	2.5	0.76	49	21	28	CL
MU 15	5	1.52	56	23	33	CH
MU 12	2.5	0.76	54	23	31	CH
MU 12	10	3.05	32	22	10	CL
MU 11	7.5	2.29	57	20	37	CH
MU 9	5	1.52	52	21	31	CH
MU 6	5	1.52	39	26	13	ML
MU 6	7.5	2.29	49	19	30	CL
MU 6	10	3.05	56	23	33	CH
MU 5	5.5	1.68	56	21	35	CH
		Average =	50	22	28	

Table 2. Properties of samples for consolidated-undrained type triaxial tests

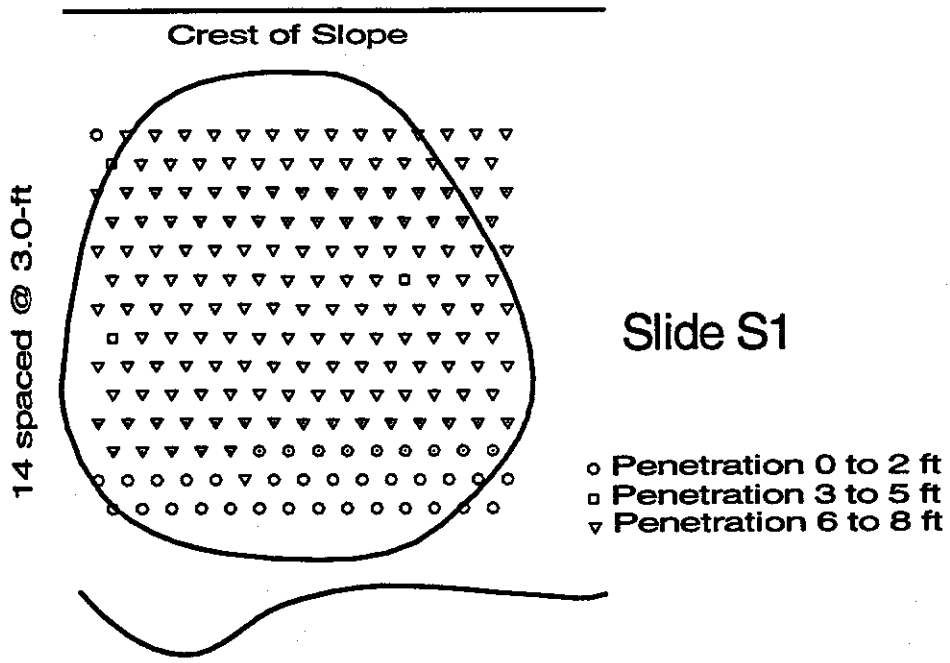
Specimen No.	Location	Depth (ft)	Dry Unit Weight (pcf)	Water Content (%)	Liquid Limit	Plasticity Index	Effective Cohesion (psf)	Effective Friction Angle (°)
274-1	MU 12	2.5-5	94	23.6	54	31	275	15
274-2	MU 12	2.5-5	94	23.1	54	31		

Installation and Constructability

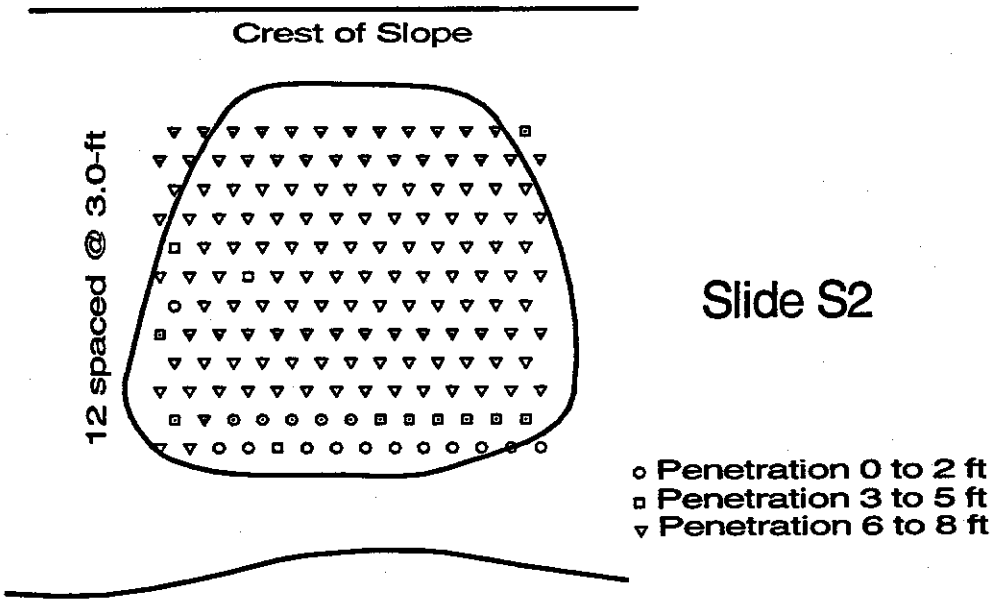
A total of 317 RPPs were installed in slides S1 and S2 during October and November 1999. The pins were installed in a 3-ft (0.91-m) staggered grid with every other row offset by 1.5-ft (0.46-m) (Figure 2). Pins were driven perpendicular to the face of the slope in slide S1 and vertically in slide S2. Not all of the RPPs could be driven to the full 8-ft (2.4-m) length due to the presence of concrete rubble from previous repair attempts. Actual installed lengths are shown in Figure 2. Conditions at the site were generally dry throughout construction allowing for maximum traction and maneuverability of the driving equipment.

Pin installation activities at the field demonstration site were initiated in October 1999. The initial installation equipment used at the site consisted of an Okada OKB 305 1250 ft-lb (1695 N-m) energy class hydraulic hammer mounted on a Case 580 backhoe (Figure 3). This equipment proved unacceptable for several reasons. The rubber-tired backhoe was difficult to maneuver on the slope and caused excessive rutting while trying to reach the top of the slope. Maintaining a fixed position during driving also proved difficult with the backhoe tending to slide down slope even with the outriggers placed, thereby further damaging the slope and making driving pins with the correct alignment and placement extremely difficult. The average penetration rate obtained using this equipment was 2.3-ft/min (0.7-m/min), which was significantly less than that obtained during field driving trials held in March 1999. In addition, play in the backhoe boom and the inability to maintain precise alignment of the hammer and pin

during driving resulted in an excessive number of pins being broken during installation (22 broken of 45 attempts). Set up time between installation of pins was also excessive due to difficulty in navigating on the slope and the need to constantly reposition the equipment. As a result of these problems, the rate of installation (including time for set up and repositioning) averaged only 32.8-ft/hr (10-m/hr). This rate was deemed unacceptable and installation was halted.



(a) Slide Area S1 – Pins installed perpendicular to face of slope



(b) Slide Area S2 – Pins installed vertically

Figure 2. Layout of plastic pins at the Emma demonstration site for slide areas S1 and S2 showing the varying penetration depths.

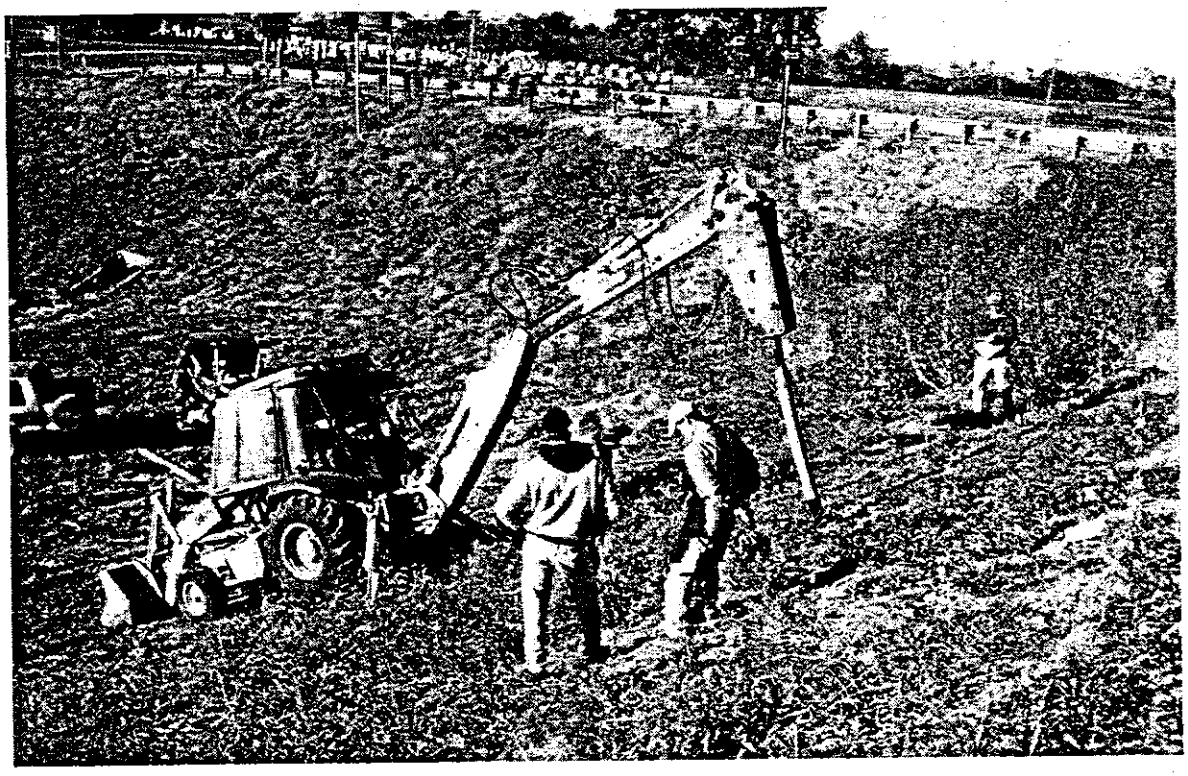


Figure 3. Initial equipment used for installation of recycled plastic pins at the Emma, Missouri demonstration site.

Installation at the field demonstration site resumed in early November 1999 using a Davey-Kent DK 100B crawler mounted drilling rig (Figure 4) supplied by the Judy Company of Kansas City, Kansas. The rig is equipped with a mast capable of 50-degree tilt from vertical forward, 105-degree tilt backward, and side-to-side tilt of 32-degrees from vertical. This rig offered numerous advantages over previously used equipment. The drilling mast ensured that the hammer and pin remained aligned during driving without requiring any movement of the chassis. In addition, the crawler-mounted rig was much easier to maneuver on the slope thereby reducing set up time between pins. The rig was equipped with a Krupp HB28A hydraulic hammer drill attached to the mast providing a maximum of 295 ft-lbs (400 N-m) of energy at a maximum frequency of 1800 blows/min. The hammer energy is further augmented by a push/pull of 18,000-lbs (80 kN) supplied by the drill mast.

Penetration rates (not including set up time) and installation rates (including set up time) measured during installation using the mast-mounted system are summarized in Table 3. The mast-mounted hammer clearly outperformed all previous installation equipment that was evaluated. Penetration rates for pins driven perpendicular to the slope reached 10-ft/min (3.0-m/min) and averaged 5.2-ft/min (1.6-m/min). Penetration rates for pins driven vertically were only slightly lower reaching a maximum of 9.6-ft/min (2.9-m/min) and averaging 4.1-ft/min (1.3-m/min). Installation rates were also dramatically higher than observed previously because of reduced set up times reaching a maximum of 124.0-ft/hour (37.8-m/hour) at peak production. The average installation rate for installation of all pins was 80-ft/hour (25-m/hour). Installation rates generally increased during installation of pins for each slide as experience was developed indicating that installation rates for future installations may be closer to the maximum rates achieved for the field demonstration.

Figure 4. Crawler mounted drilling rig used for installation of recycled plastic pins at the Emma, Missouri demonstration site.

Table 3. Summary of the penetration and installation rates for mast-mounted hammer on slope failures S1 and S2.

	Slide S1 (Perpendicular Installation)		Slide S2 (Vertical Installation)	
	Penetration Rate ft/min (m/min)	Installation Rate ft/hr (m/hr)	Penetration Rate ft/min (m/min)	Installation Rate ft/hr (m/hr)
Average Rate	5.2 (1.6)	96.0 (29.3)	4.1 (1.2)	68.0 (20.7)
Maximum Rate	10.0 (3.1)	124.0 (37.8)	9.6 (2.9)	98.0 (29.9)
Minimum Rate	0.1 (0.1)	59.0 (18.0)	0.5 (0.2)	33.0 (10.1)

Limitations of the Davey-Kent drilling rig necessitated that pins installed in a vertical alignment were driven with the rig being backed up the slope. While not critical, this feature did result in slightly lower installation rates for pins driven vertically as compared to pins driven perpendicular to the face of the slope (Table 3). An alternative rig, the Crawlair ECM-350 extendible boom rig manufactured by Ingersoll-Rand, capable of driving pins vertically from a forward position was also used at the site. The Crawlair rig was equipped with an EVL-130 430-lb. (1.9-kN) air hammer operating at a maximum frequency of 2100-blows/min attached to a chain drive capable of 3000-lb. (13.3-kN) of down force. While the rig was more maneuverable than the Davey-Kent rig, the hammer and drive system lacked the power and down force necessary to achieve acceptable penetration rates and its use was discontinued after a few attempts.

Comparison of the drill mast system versus the backhoe-mounted system revealed the mast system to be much more effective, accurate, and require less skill to operate. The crawler system also caused much less damage to the slope than the rubber-tired equipment. The crawler system did become marginally stable when operating on the steepest parts of the embankment (>2H:1V) and had to be tethered to the top of the slope in some locations.

Economics

A total of 317 pins were driven into slides S1 and S2 in slightly over four days. The RPPs were supplied at no charge by Tamko Composite Products; however, their market value is conservatively estimated to be $20 per pin (4inx4inx8ft). The cost for the RPPs would therefore have been $6340. The cost of installation (equipment and labor) was $5250 resulting in a total cost of approximately $11590 to stabilize the two failures.

In order to compare stabilization alternatives, the total cost for the RPP stabilization was calculated on a unit area basis (Table 4), i.e., the total cost was divided by the total area of the slope faces. For the Emma site, the unit cost using RPPs to stabilize the slope was $3.91/ft^2 ($42/m^2). Alan Miller of MoDOT provided an estimate for using rock armor of $5.40/ft^2 ($58/m^2). Pat Carr of The Judy Company estimated the cost of traditional soil nailing to be $19.00/ft^2 ($200/m^2). Based on these estimates, the RPP stabilization method is the least costly. As experience is gained and installation technology improved, the unit cost for RPP stabilization is expected to decrease.

Table 4. Cost comparison for alternative slope stabilization methods.

Stabilization Method	Cost/Unit Area of Slope Face ($/ft^2)	($/m^2)	Comments
Recycled Plastic Pins	3.90	42	Based on actual costs for Emma site
Rock Armor	5.40	58	Estimated cost provided by A. Miller
Soil Nailing	19.00	200	Estimated cost provided by P. Carr

Future Enhancements to Driving System

Several enhancements of the installation equipment are currently being considered. One alternative being considered is to mount the mast system on equipment with booms to expand the reach of the driving system. Candidate equipment for this purpose includes crawler-mounted excavators (track-hoes) and extendible boom excavators ("grade-alls"). The additional reach provided by such equipment would enable the equipment to remain off the slope during installation, which will further limit damage to the slope and reduce set up time. An excavator/grade-all mounted mast system will also have greater swing range than the crawler mounted system used allowing a larger number of pins to be driven without movement of equipment.

Several mechanical problems slowed progress during installation at the demonstration site. The connection between the hammer and the pins proved particularly troublesome as several different connections failed during installation. In the early stages of installation with the mast-mounted hammers, a welded drive head was used to receive the drill bit connector allowing the transfer of energy from the hammer to the RPPs. The repetitive impacts from the hammer inevitably caused the welded connections to fail. While several spare connectors were kept on site, construction was stopped on two occasions to permit re-welding of the connectors. In the

latter stages of installation, a mechanical compression connection used during previous field driving trials (March 1999) was adapted for use with the mast-mounted hammers. This connection performed much better than the welded connections but eventually caused failure of the drill bit connector due to incompatibility between the steel used for the connection and the steel used in the hammer.

Instrumentation and Monitoring

The Emma site was instrumented to monitor stability of the stabilized slope and measure forces and deformations in reinforcing members (RPPs) to enable improvements to design and construction methods. Two inclinometers and two piezometers were installed in each of the RPP stabilized slides (S1 and S2) to monitor lateral movements and locate the phreatic surface (should one exist) (Figures 5 and 6). Control slide S3 was instrumented with one inclinometer and one piezometer (Figure 7). The inclinometers were installed on 7 Dec. 1999.

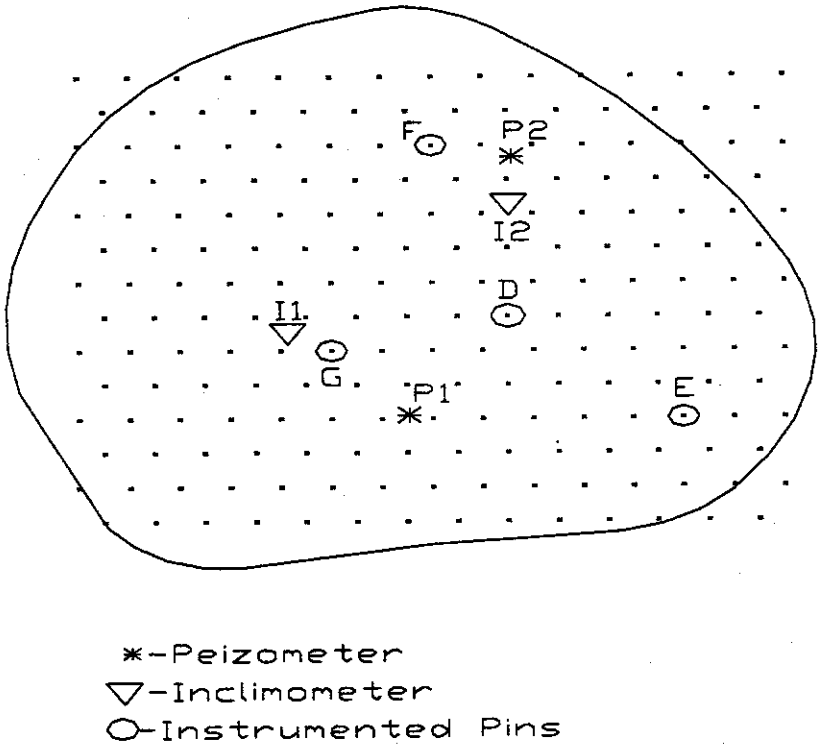


Figure 5. Plan view of location of instrumentation for Slide S1 Emma Missouri.

Ten RPPs were instrumented, each with 20 strain gages, for measuring strain in the RPPs from which bending moment, shear and lateral loading in the pins can be calculated (Figure 8). Four instrumented pins were installed in slides S1 and S2. Two instrumented pins were installed in the control slide (S3). Initial strain readings were taken on 6 Dec. 1999, shortly after installation. Additional readings have been taken at approximately one month intervals. The 6th set of strain gage readings was taken at the Emma site on 17 April 2000. These data sets include the initial readings, one set of readings in which incorrect data was recorded, three sets of usable data and one set of readings in which new initial points were established for each strain gage.

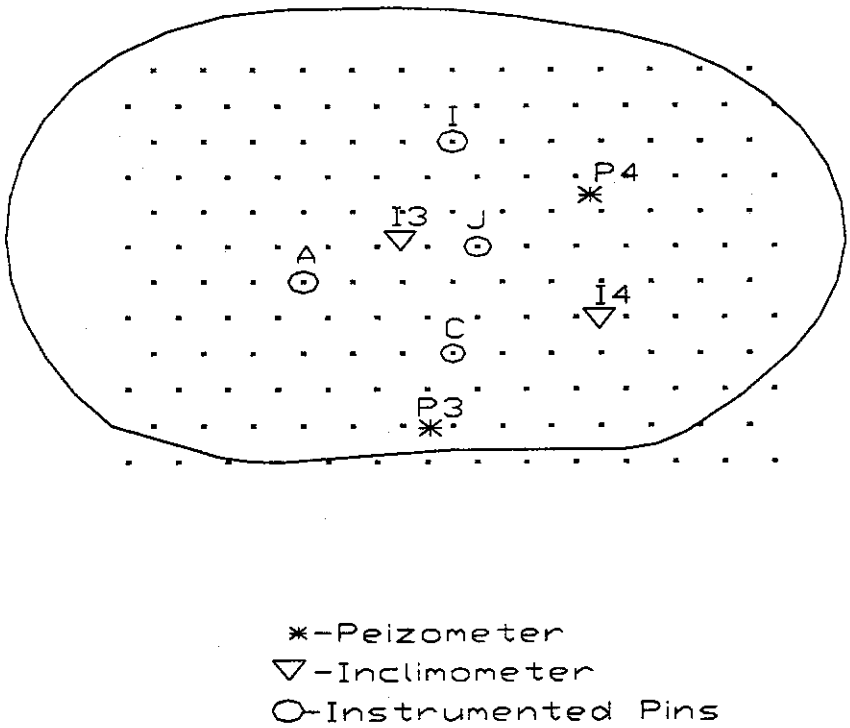


Figure 6. Plan view of location of instrumentation for Slide S2 Emma Missouri.

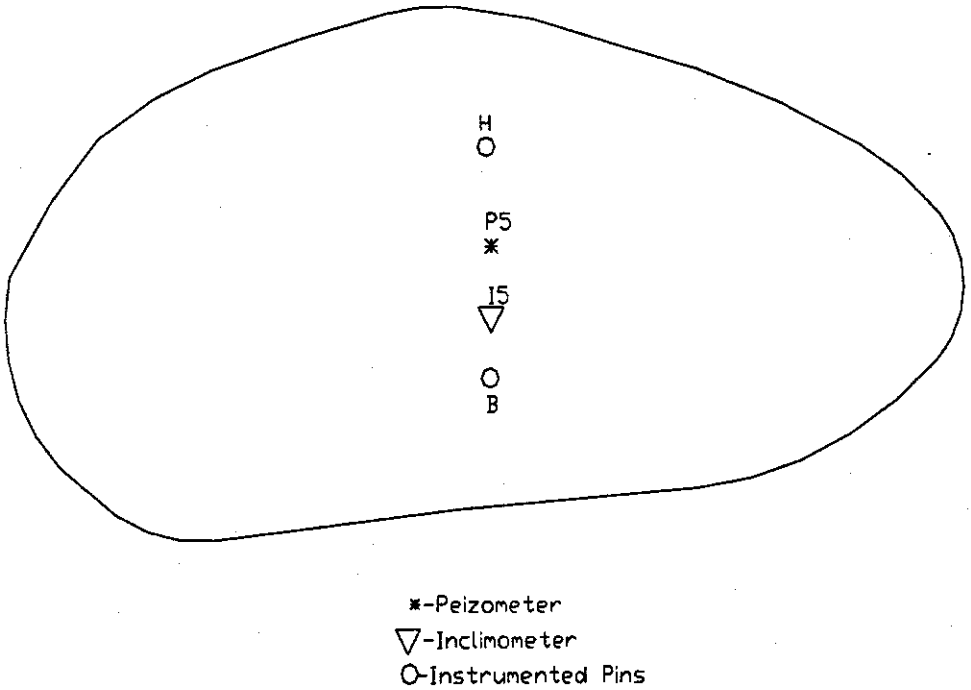


Figure 7. Plan view of location of instrumentation for Slide S3 Emma Missouri.

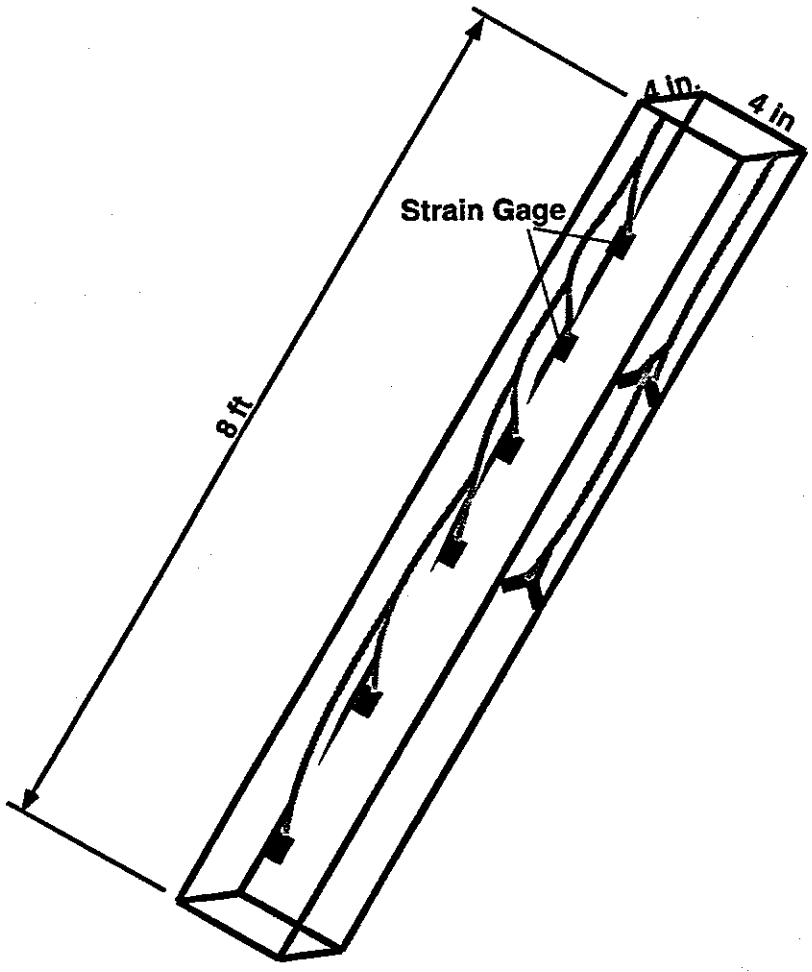


Figure 8. RPP showing the location of resistance strain gages for measuring strains on the pins in the field.

The initial method for reading the gages was to set the readout device to the original balanced (zero voltage output) position for each particular gage. The strain readings obtained in this manner represented the total strain in the gage including any initial strains imposed during installation. Initial strains determined from the initial set of readings were then subtracted from subsequent readings to produce the total strains experienced since installation. This method required approximately two days to complete a set of readings for all gages at the site. In order to reduce the time required to take a set of readings, a new method was subsequently established in March 2000 after consultation with the strain gage manufacturer. In this method, a single balanced position was used for all gages thereby eliminating the need to re-zero the readout unit for each gage before reading. New baseline strains for each gage were established in March 2000 using the new method. Strains measured prior to establishing the new method of reading were added to all readings measured since inception of the new measuring procedure to produce total strains since installation. The new reading procedure dramatically simplifies the reading procedure thereby reducing the chances of error and reducing the time required to make all measurements to approximately 1 day. The strain reading process could be further expedited by using a combination of switch boxes and automated data loggers. These considerations will be incorporated into plans for an expanded field demonstration program.

An example of the measured strain data is shown in Figure 9, which indicates that total strains (as of 17 April 2000) are still very small (1000 micro strains = 0.1 percent strain). Gages 1-6 and 11-16 are located on opposite sides (down-slope and up-slope, respectively) of the pin and can be used to determine the developed bending moment in the pins. The strains reported in Figure 9 are not of opposite sign and equal magnitude, which is an indication of loading other than pure bending. This necessitated making some simplifying assumptions regarding the nature of the bending occurring in the pins before the state of stress in the pins could be computed and analyzed. Gages 7-10 and 17-20 are located on the sides of the pins (the faces parallel to the dip of the slope) and are for determining shear in the pins.

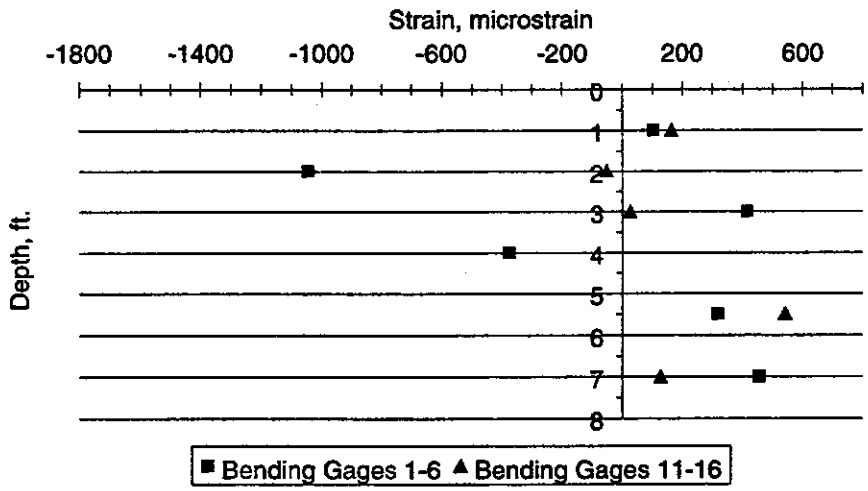


Figure 9. Typical strain versus depth along instrumented Pin A (mid-slope, slope S2), as of 17 April 2000, located at the Emma, Missouri demonstration site. Note: 1000 microstrains is equal to 0.1 percent strain.

A procedure for calculation of both bending moments and shear forces using strain gage data from the instrumented RPPs has been established. In the procedure, shear and bending moment diagrams are determined for each pin by using the measured strains at the outer fiber of the pin to determine the flexural stress (σ). The bending moment is then back calculated from the flexural stress equation. A typical plot of bending moment is shown in Figure 10.

Plots of the bending moments developed in each pin are provided in Appendix C. These figures show that only a small portion of the bending moment capacity (1000 ft-lb) has been mobilized and little change in the bending moments has occurred. This is in agreement with the fact that precipitation has been below normal (Figure 11) and therefore negligible slope movements were expected.

Slope inclinometer readings have been taken in February 2000 and June 2000. The February 2000 inclinometer data is shown in Appendix D. The A-axis is taken to be parallel to the dip of the slope, and the B-axis is parallel to the strike of the slope. The maximum recorded movement parallel to the dip of the slope is approximately 0.3 in. (8 mm), at inclinometers 1 and 3. These inclinometers are located near the center of slides S1 and S2, respectively. The maximum recorded movement parallel to the strike is approximately 0.6 in. (15 mm), at

inclinometers 1 and 4. The small movements recorded by the inclinometers verify that little movement has occurred in the slope and correlates with the low bending moments measured to date. The reason for the measured longitudinal movements (strike direction) exceeding the lateral movements (dip direction) is subject to discussion. One possible explanation of this observation is that the inclinometer casing is shifting slightly within the borehole walls. This explanation is supported by the fact that some movements appear to be in the direction opposite of what would be expected. Regardless of the reason(s) for the longitudinal movements, the measured movements are small and are likely below the "detection limit" for inclinometers placed with sand backfill.

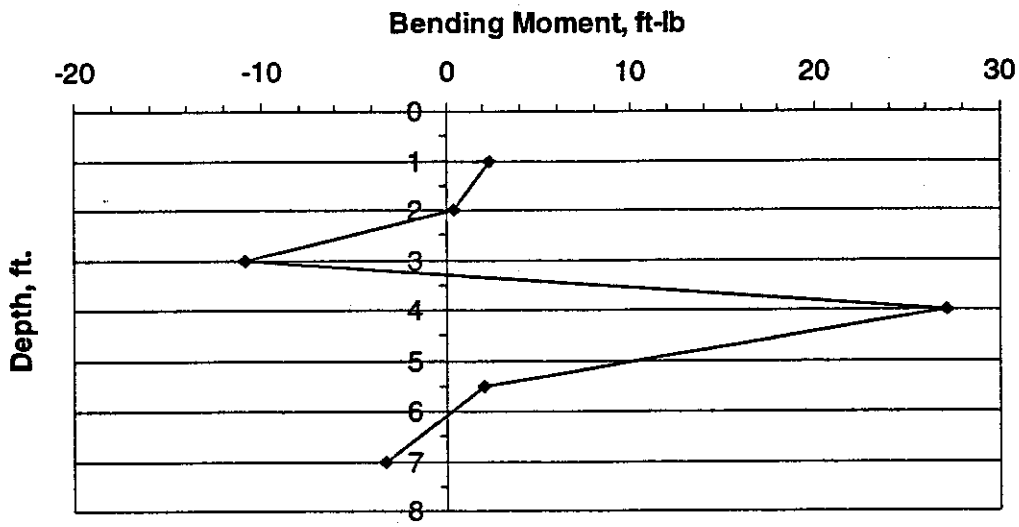


Figure 10. Measured bending moment as a function of depth in Pin B, Emma Missouri demonstration site – 17 April 2000.

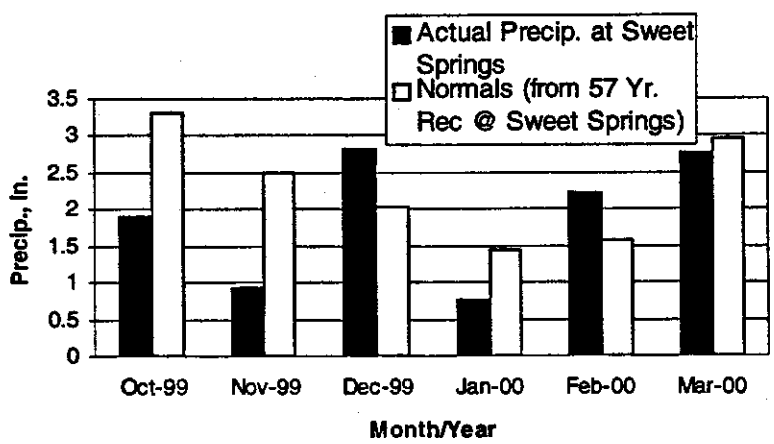


Figure 11. Precipitation data collected from Sweet Springs Missouri weather observation station 3.5 miles east of the Emma Missouri demonstration site. Note: Pins were installed in November 1999.

In addition to strain, and inclinometer measurements, water levels are being monitored using standpipe piezometers. To date two sets of water level measurements have been recorded. On 17 April 2000, water levels were at 15 and 18 ft. below ground surface in the bottom and top piezometers in slide S1, respectively. In slide S2, the waters levels were at 5.5 and 15 ft. below

ground surface in the bottom and top piezometers, respectively. These levels indicate that the water surface is not within the failure zone. Future water level measurements will be recorded simultaneously with strain gage readings.

Local precipitation data is also being collected for the site to monitor rainfall over time. David Schelp, a resident of Emma approximately 0.6 mi. (1 km) south of the site, has been provided with a rain gage and data record log and has volunteered to record the daily precipitation in Emma. In addition, daily high and low temperatures and precipitation measurements are being collected from the National Climatic Data Center for the NOAA cooperative weather station in Sweet Springs, Missouri located 3.5 mi. (6 km) east of the Emma site (NCDC, 2000). Data from the weather observation station is generally not available until approximately two to three months after the fact. Data obtained to date from the weather observation station are shown in Figure 11. The dual observations provide local site rainfall conditions in a timely manner, backed up with the accuracy of professional weather observations.

Development of Preliminary Design Procedure

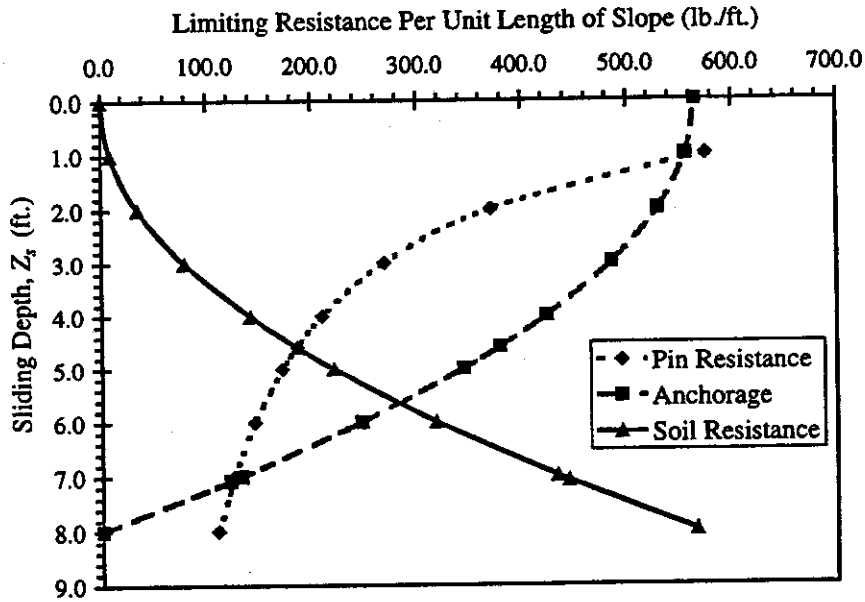
The fundamental objective for design of stabilization schemes using slender reinforcement is to determine the resistance that can be provided by individual pins and subsequently the number of pins required to increase the stability of a slope to an acceptable level. Given the lateral resistance of individual pins, the mechanics of stability analysis for slopes reinforced with structural members is relatively straightforward and well established. Thus, the primary need for design of RPP stabilization schemes is to determine or estimate the resisting forces provided by each reinforcing member.

A preliminary method for predicting the lateral resistance of individual pins has been developed. The method uses a limit state design approach wherein a series of potential failure mechanisms are considered in developing the overall distribution of lateral resistance along a reinforcing member. The two general limit states that are considered include: (a) failure of the soil around or between reinforcing members and (b) failure of the reinforcing members themselves in bending or shear as a result of excessive stresses being imposed by the retained soil.

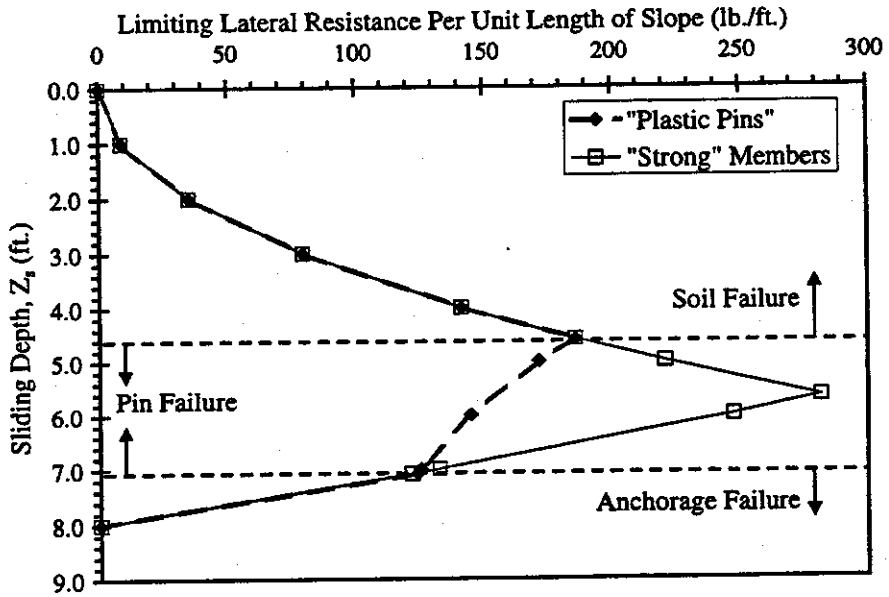
Figure 12 illustrates the general approach for developing the distribution of limiting lateral resistance for a single reinforcing member. In this method, separate limiting resistance curves are developed based on the available resistance from the soil surrounding the pin, the capacity of the pin, and the anchorage length as shown in Figure 12a. Based on these curves, a composite limiting resistance curve is developed that corresponds to the most critical component of resistance at each sliding depth as shown in Figure 12b.

The limiting resistance curve for the soil is computed by integrating the pressure that will lead to passive failure of the soil or to flow of the soil between reinforcing elements (the limiting soil pressure) over the length of the pin above the depth of sliding. At present, the method developed by Ito and Matsui (1975) is used for predicting the limiting soil pressure. The resistance provided by the length of the reinforcing element extending below the sliding surface (the anchorage length) is similarly computed by integrating the limiting soil pressure over the length of the reinforcing element extending from the sliding surface to the end of the member. Combining the curves for the limiting soil resistance and limiting anchorage resistance results in

a limiting resistance curve that is suitable for a reinforcing member of infinite strength as shown in Figure 12b.



(a) Limiting resistance due to limiting soil pressure, pin capacity, and anchorage length



(b) Composite limiting resistance considering all limit conditions

Figure 12. Distribution of limiting lateral resistance per unit length of slope: (a) limiting resistance for the soil, pins, and anchorage and (b) composite limiting resistance. Data shown are for 4-in. x 4-in. x 8-ft. pins on 3-ft. spacing in soil with c=0, ϕ=21 degrees, and γ=110 pcf.

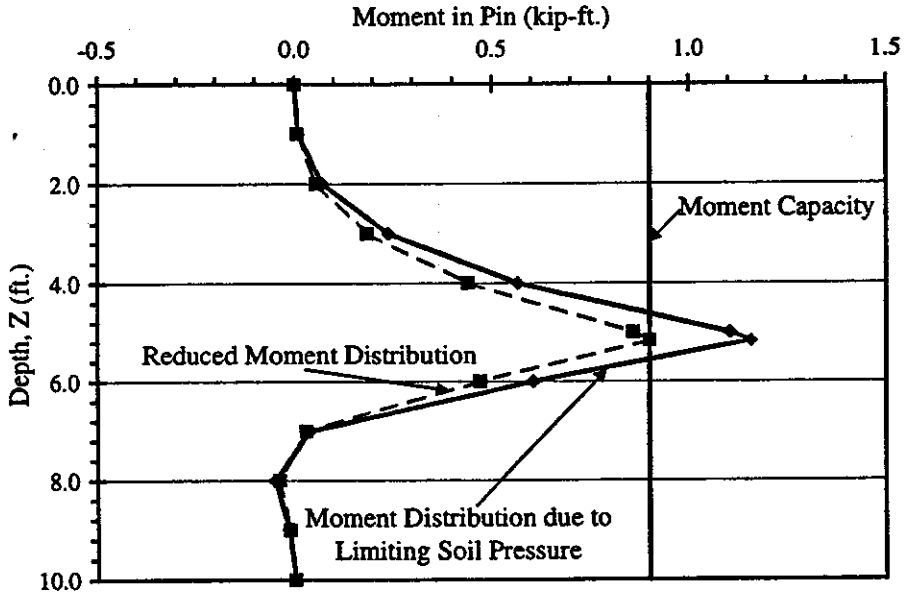
Application of lateral stresses to a pin that equal the limiting soil pressure may lead to bending moments or shear forces that exceed the capacity of the pin. In this case, the pin will

fail prior to the limiting resistance of the soil surrounding the pins being fully mobilized, and the limiting resistance predicted by considering failure of the soil alone will be unconservative. To account for the potential for failure of the pins, a reduction factor, α_m, is computed as the ratio of the moment capacity of the pin to the maximum moment induced due to the limiting soil pressure distribution as shown in Figure 13. A "reduced" limiting soil pressure distribution is then determined by reducing the limiting soil pressure distribution by the factor α_m (Figure 13). The resulting lateral pressure distribution limits the induced moments in the pin to be less than the moment capacity of the pin. A similar reduction factor, α_s, is subsequently computed considering the shear capacity of the pin with the smaller of the two reduction factors being used to compute a final reduced lateral pressure distribution. The limiting resistance distribution for the pin is then simply calculated by integrating the reduced lateral pressure distribution over the length of the pin from the ground surface to the sliding surface. While not currently included in the design method, consideration of axial resistance in compression or tension could be considered in a manner similar to that used for bending and shear.

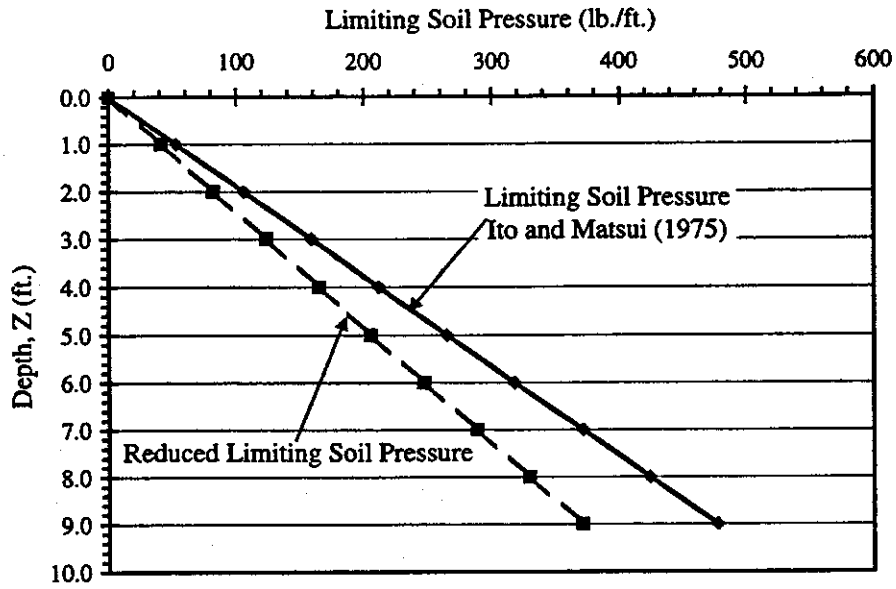
As shown in Figure 12b the dominant failure mechanism for shallow sliding surfaces is passive failure of the soil surrounding the pins. The capacity of the reinforcing members tends to govern design for intermediate depth sliding surfaces below which failure of the pins tends to occur due to lack of sufficient anchorage length.

The computed distribution of limiting lateral resistance using the procedure described above depends on a number of parameters including pin spacing, sliding surface depth, pin strength and stiffness, and soil strength and stiffness. A series of parametric studies were performed to evaluate the importance of various parameters contributing to the limiting resistance distributions for the recycled plastic pins. Results from these studies indicate that the depth of sliding and the pin moment capacity are the most critical parameters for design of stabilization schemes using recycled plastic pins with the moment capacity controlling design for a wide array of problems. Additional factors that likely affect the true distribution of lateral resistance have also been identified including pin inclination, group effects, and creep of the recycled plastics. The magnitudes of these factors are not well understood, however, as the current state of knowledge is limited in these areas. As a result, these additional factors are not currently considered in design. These limitations currently give precedence to placing reinforcing members in distributed arrays instead of in closely spaced, or reticulated groups since no advantage due to the potentially beneficial interaction of closely spaced members can be counted on during design.

It is important to point out that there are benefits to having the pin capacity control design. Firstly, while the properties of recycled plastic are somewhat variable, they are not as variable as the properties of the soil. Thus, by having the reinforcing members control design, the importance of accurately knowing soil properties is diminished and the reliability of the design is increased. In addition, the theories used to establish the limiting soil pressure that the soil can provide are still a subject of debate (DeBeer and Carpentier, 1977; Hull and Poulos, 1999). Having the pin capacities control design reduces the necessity of having a highly accurate theory for predicting the loads imposed by the soil on reinforcing members.



(a) Moment distribution induced by limiting soil pressure and reduced moment distribution



(b) Computed limiting soil pressure and reduced limiting soil pressure

Figure 13. Method for computing the limiting pin resistance as a function of depth of sliding: (a) moment distributions due to limiting soil pressure and reduced lateral pressure and (b) limiting soil pressure and reduced limiting soil pressure.

Engineering and Material Properties

An extensive program of materials testing was completed in order to: (1) determine the basic engineering and material properties of the recycled plastic pins; (2) determine the potential variability of these properties; and (3) determine how these properties change when the recycled plastic is subjected to various potentially detrimental environments.

Tension, Compression, Shear and Bending

Tests performed include simple tension and compression as well as shear and bending tests. Results for all tests are summarized in Table 5. Two different production runs of RPPs were received from Tamko Composite Products (formerly DuraBoard) for the project. The first batch of RPPs and recycled plastic sheeting was received in Spring 1998, about 1 year prior to the start of this study. Testing was initiated on these materials in the Spring of 1998. The second batch of RPPs was received in July 1999 and was used at the field demonstration site (Emma) as well as being tested in the laboratory.

As shown in Table 5, for batch #1, the tensile strength of the recycled plastic is nominally about 1.8 ksi. The compressive strength of the recycled plastic is nominally 2.9 ksi or about that of low-end concrete. The peak shear stress is 1.2 ksi and the peak bending moment is 1.1 kip-ft. For batch #2, the average peak tensile stress decreased to 1.3 ksi (from 1.8 ksi for batch #1) and the average compressive strength was about the same at 2.8 ksi. The peak moment for both batches was nominally 1.0 kip-ft.

Table 5. Summary of tension, compression, shear and bending test results on recycled plastic material used in the RPPs.

	Tension Tests				Compression Tests				Shear Tests		Bending Tests	
	Young's Modulus (ksi)		Peak Stress (ksi)		Young's Modulus (ksi)		Peak Stress (ksi)		Peak Stress (ksi)		Peak Moment (kip-ft)	
	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.
Batch #1 (Control for Exposure Tests)	126	15	1.8	0.2	105	6.0	2.9	0.3	1.2	0.2	1.1	0.2
Batch #2	146	33	1.3	0.2	145	13	2.8	0.1	N/D	N/D	1.0	0.2

Based on the strength results for the two batches, there are some differences in the materials. The results are based on the averages of 10 to 16 tests from each batch. Prior to testing, the outward appearance of batch #2 specimens showed better consistency and less surface voids. Batch #2 RPPs were assumed to be of higher quality and strength material. However, examination of the cross-sections of the broken specimens after testing, showed that batch #2 specimens contained more voids on the interior than batch #1 specimens. This likely accounts for the slight decrease in the peak tension stress for the batch #2 specimens. The average bending strengths, critical to the slopes application were similar for both batches of

RPPs. Continued testing of material properties of pins delivered to future sites is recommended until the variability of the product is better established.

Tension and compression specimens were also prepared from the recycled plastic material and placed in several exposure environments for up to one year in order to evaluate the durability of the material. The environments included: acidic solution (pH 5), ultraviolet (UV), tap water, kerosene, and freeze/thaw. One specimen was removed from each environment and tested at selected times (7, 14, 28, 63, 330 and 375 days). The tensile and compressive strengths after one year of exposure are shown in Table 6. These data show the compressive strengths were essentially not impacted. The tensile strengths did show some deterioration from the unexposed condition with the greatest impact observed for the specimens immersed in kerosene. Samples immersed in kerosene showed a reduction in peak tensile stress of approximately 50 percent after one year of exposure. While this observation is a cause for some concern, it is important to note that it is unlikely that the pins would be subjected to such a high concentration of kerosene or other hydrocarbon, even in the unlikely event of a direct spill from a tanker truck or rail car. Details of all exposure tests are presented in Appendix E.

Table 6. Summary of tension and compression properties of recycled plastic material subjected to various exposure environments.

Exposure Environment[1]	Tension		Compression	
	Young's Modulus (ksi)	Peak Stress (ksi)	Young's Modulus (ksi)	Peak Stress (ksi)
No Exposure	129	1.8	107	3.0
pH 5	114	1.5	106	2.5
UV	112	1.4	91	2.5
Tap Water	112	1.5	104	2.4
Kerosene	97	0.9	86	2.2
Freeze/Thaw[2]	105	1.6	100	3.0

[1]Data shown are for tests conducted on specimens after 375 days of exposure in the noted environment with the exception of the tension values, which are for specimens after 330 days of exposure.
[2]Data are for specimens subjected to 334 freeze/thaw cycles (-15 to +45 F degrees)

Interface Friction

The friction at the interface of dissimilar materials is known as interface friction and is generally represented using an interface friction angle (δ). Interface friction is a factor in reinforcement effectiveness, particularly in regard to pullout resistance or pile/pin side shear. A series of direct shear and small-scale tilt table (Figure 14) tests were performed to determine the peak and residual interface friction angles for the interface between the recycled plastic pins (RPP) and soil from the Emma demonstration site. The soil used for the interface tests was from the upper two feet of soil at the site. The contact area between the soil and the RPP in the direct shear tests was 5 square inches (30 cm^2) and for the tilt table was 95 square inches (600 cm^2). The evaluation considered the variability of surface texture of the RPP with soil and the comparison of peak and residual interface friction angles between direct shear tests and tilt table tests. The following observations were noted from these tests:

- For direct shear tests, δ_{peak} and $\delta_{residual}$ for the soil-RPP interface were approximately 25 and 20 degrees, respectively. Although a cohesion (c) intercept was measured, it would not be included in design.

- For tilt table tests, the average δ_{peak} was approximately 35 degrees and average $\delta_{residual}$ was approximately 30 degrees. The measured δ_{peak} and $\delta_{residual}$ were about the same for different normal loads.
- δ_{peak} and $\delta_{residual}$ from tilt table tests were approximately 10 degrees higher than the direct shear results. This was likely due to the rougher surfaces of the plastic pins used in the tilt table tests. The tilt table utilized full-scale pins while the direct shear device could only accommodate a 2.5-in. (62mm) wide section of the pin, thus limiting the representativeness of the specimen.

The interface friction angle between the RPPs and soil in the shear zone is expected to be lower than the values measured in these interface tests. The measured effective stress friction angle of the soil in the shear zone was 15 degrees. Typically, an interface friction angle cannot be greater than the friction angle of the soil at the interface since failure will develop in the zone of weakest shear resistance (the zone with the lowest friction angle). A design interface friction angle for the RPP interface with the Emma site soil should therefore be approximately 15 degrees. At this time, the preliminary design methodology is independent of interface friction; however, a design process that incorporates pin group or interaction effects may be partially dependent on interface friction.

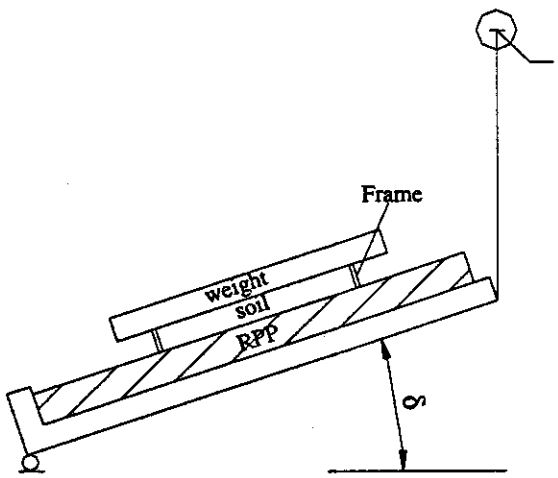


Figure 14. Elevation view of tilt table test for interface friction between Emma soil and RPPs. The contact area between the soil and the RPPs was 95 square inches (600 cm^2).

Bending Creep

The major constituent of the recycled plastic pins used for this project is polyethylene, a long chain polymer. The characteristics of this material make it more susceptible to deformations under sustained loads, i.e., creep, than many engineering materials such as steel, concrete, and wood. In the slope stabilization application, the RPPs are subjected to bending loads, which may be sustained over a period of time. It is likely that the pins will alternate between periods of loading (wet climatic periods) and reduced loading (climatic dry periods when the soil in the slope has higher strength). In the extreme event, bending creep may become a critical parameter in the RPP performance. Knowledge of the potential impact or magnitude of deformation of the RPPs under sustained loading is important to understanding the long-term

stability afforded by the pins. An extensive laboratory evaluation of the bending creep process of the RPPs was conducted and an approach for evaluating the impact of bending creep on the RPPs was developed.

Bending creep testing was performed on scaled RPPs having 2 in. x 2 in. cross-sections and lengths of 24 inches. The scaled RPPs were placed in frames and loaded as cantilever beams as shown in Figure 15. Pins were loaded with either single or multiple point loads along their length. The deflection of the loaded pins was measured over time. Tests at room temperature (70°F) have been underway for 700 days and are continuing. Tests at elevated temperatures of 95, 133, 154 and 176 degrees Fahrenheit have been completed. Typical results of deflection versus time are shown in Figure 16. The behavior shown is typical of the RPPs tested at the various temperatures (Appendix F).

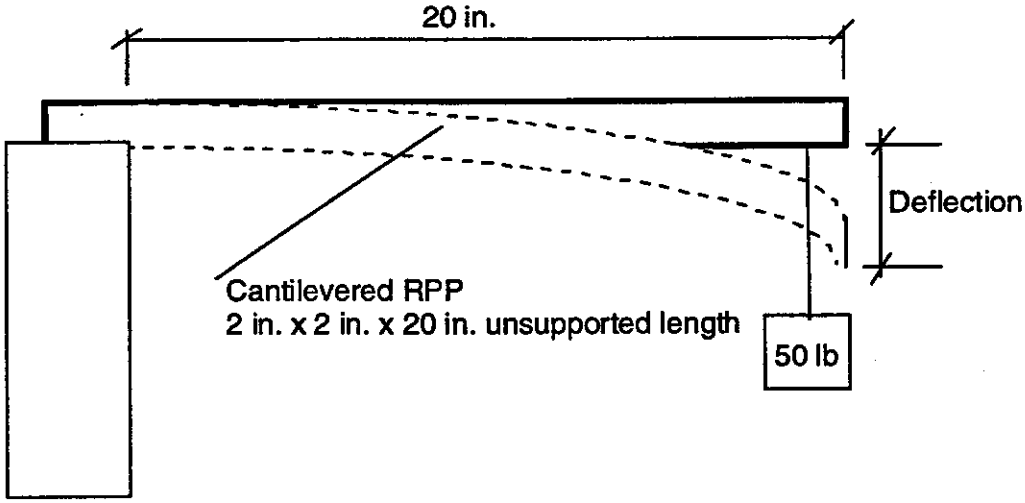


Figure 15. Setup for testing creep behavior of recycled plastic pins under bending loads. L was 20 inches and the pin cross-section was 2 in. x 2 in.

By its very nature, creep is a long-term phenomenon. For example, the RPPs being tested at 70 degrees F have been under load for almost two years but have not failed. Failure was defined as breakage of the RPP. The tests at elevated temperatures were established in order to accelerate the creep process. Results from the accelerated testing are being used along with the Arrhenius method (Koerner et. al 1990) to estimate the long-term creep behavior for the RPPs in the field.

An example of an Arrhenius plot is shown in Figure 17. The plot is the natural logarithm of the inverse of the time required for the RPP to break versus the inverse of the temperature at which the test was conducted. The plot shown includes data for tests at 70, 95, 133, 154, and 176 degrees F. As the test temperature is increased, the time to reach the breakage point is reduced. The slope of the line on the Arrhenius plot is known as the activation energy (E_{act}/R), where R is the Universal Gas constant (8.314 J/mol-°K). Knowing the activation energy (slope of the Arrhenius plot), reaction rate intercept on the Arrhenius plot and the temperature of the actual site (T_{site}), the time for the RPP to creep to the breaking point under field (temperature, assumed 70 degrees F) conditions can be estimated (Eq 1).

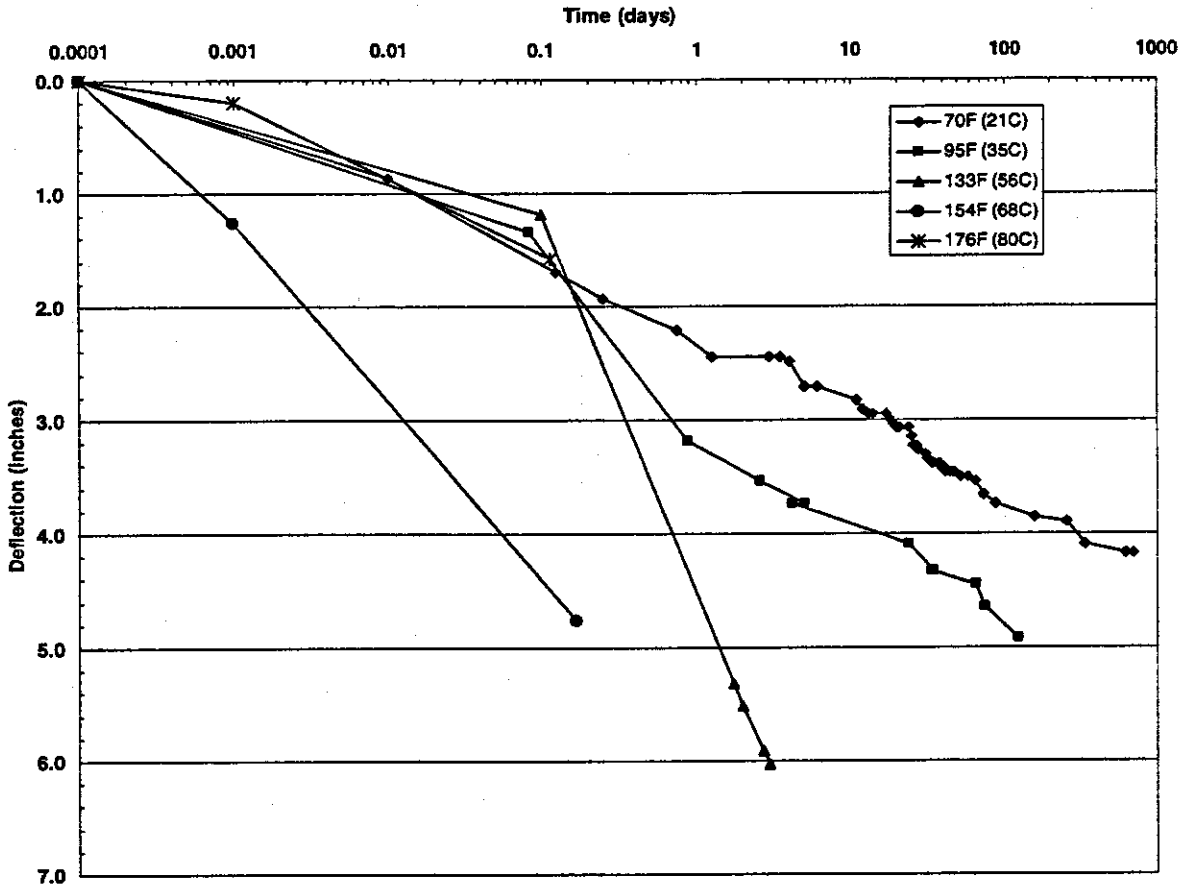


Figure 16. Deflection versus time response for RPP loaded with a 50 Lb at the free end of a simple cantilever (Figure 15) under various temperatures from 70 to 176 degrees. With the exception of the specimens at 70 degrees, all specimens failed at the final data point.

$$\ln\left(\frac{1}{t}\right) = \ln A - \left(\frac{E_{act}}{R}\right)\left(\frac{1}{T_{site}}\right) \qquad \text{Eq. 1}$$

Stress level along with temperature will affect the creep behavior of a material and RPPs are no exception. The higher the stress level or the closer it is to the ultimate strength of the material, the faster the creep rate and shorter the time to failure. The creep behavior of the RPPs was evaluated under four different loading conditions. Three of the loadings involved a single point load (50, 35 or 21 lbs) and one loading was distributed, i.e., five 10 lb loads distributed evenly along the RPP. The loading conditions, maximum moments, and time to reach failure as predicted from the Arrhenius method are shown in Table 7. In addition, the moment capacity and ratio of tensile stress (due to the applied loading) to tensile strength are shown in the table. Specimens were loaded to 70%, 50% and 30% of the ultimate tensile strength for the point loading condition.

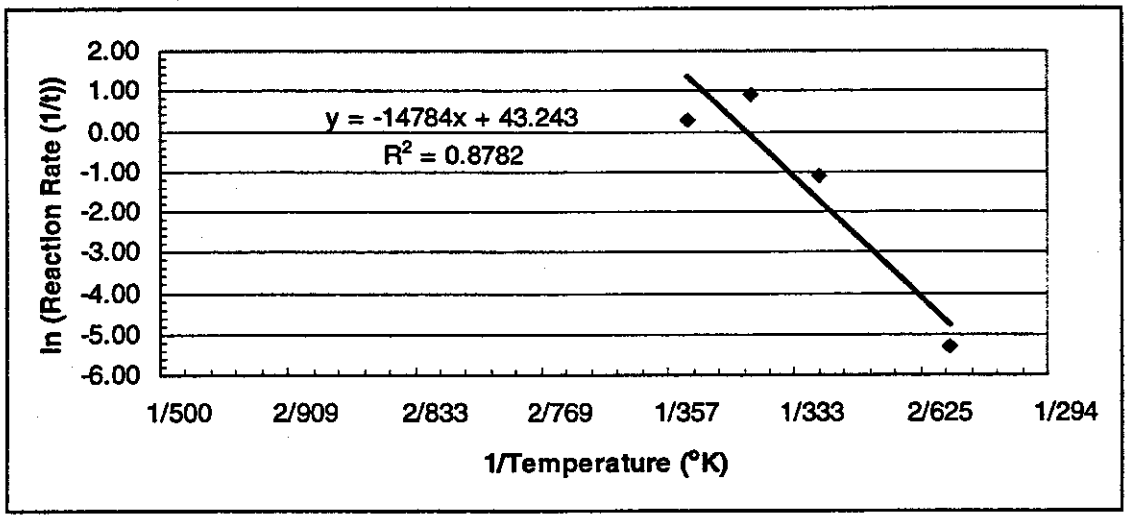


Figure 17. Typical Arrhenius plot for bending creep test on 2in.x2in.x24in. RPP loaded as a cantilever with a 50 Lb weight at the unsupported end. Plots for other loading conditions are included in the Appendix F.

Table 7. Conditions and results of the creep bending tests on the RPPs

Loading Condition	Max Moment in Creep Test Specimen, M_{test} (ft-lb)	Moment Capacity for a 2"x2" RPP, M_{max} (ft-lb)	Ratio of M_{test}/M_{max}*100% (%)	Ratio of Tensile stress in Creep to Tensile Strength ($\sigma_{Tcreep}/\sigma_{TRPP}$) (%)	Time to Reach Failure Due to Bending Creep At 70°F (years)
(1)	(2)	(3)	(4)	(5)	(6)
50 Lb Single Point Load	80	200	40	70	3.1
35 Lb Single Point Load	55	200	28	50	830
21 Lb Single Point Load	33	200	17	30	1700
Five 10 Lb loads @ Equal Spacing	49	200	25	45	6000

The time to reach failure (under creep loading, at 70°F) versus percentage of the tensile strength of the RPP is plotted in Figure 18. The data in this plot enable one to predict the effective creep lifetime of an RPP in the field. The following steps illustrate the method:

1. Measure the strain on an RPP in the field and calculate the bending moment (M) for the pin.

   (at the Emma field site on April 17, 2000, Pin C showed a bending moment of 400 ft-lbs., see Appendix C)

2. Use the calculated moment to calculate the tensile stress (σ_T) in the extreme fiber of the RPP.

$\sigma_T = \frac{My}{I}$ where M is the calculated moment in the field RPP, y is the distance from the neutral axis to the extreme fiber and I is the moment of inertia for the given section.
(for Pin C at Emma, M = 400 ft-lb, y = 1.75 inches and I = 12.5 inches^4, thus σ_T = 672 psi.)

3. Check the ratio of the calculated tensile stress to the maximum tensile stress for the pin and given section.

   $$\text{Ratio} = \frac{\sigma_{T\,field}}{\sigma_{T\,max}}$$

   (for Pin C at Emma σ_{Tmax} is 1800 psi and the ratio of tensile stresses is 37% rounded to 40%)

4. On the plot of time to failure (t_f) versus percentage of maximum tensile stress (Figure 18) locate the calculated percentage of maximum tensile stress and find the corresponding time to failure (t_f)

   (for Pin C at Emma, the percentage of maximum tensile stress is 40% and the resulting time to bending creep failure is found to be approximately 1000 years.)

The above procedure can be used to estimate the design life of the RPPs in the slope stabilization application. If the estimated time to failure is too low, the engineer can modify the design to reduce the stress level in the pins and thereby increase the design life. Options for reducing the stress include increasing the number of pins, increasing the size of the pins or modifying the cross-section to increase their moment of inertia. Continued monitoring of moments developed in the instrumented pins at the Emma site will produce updated mobilized moments under field loading conditions which can be utilized in future design.

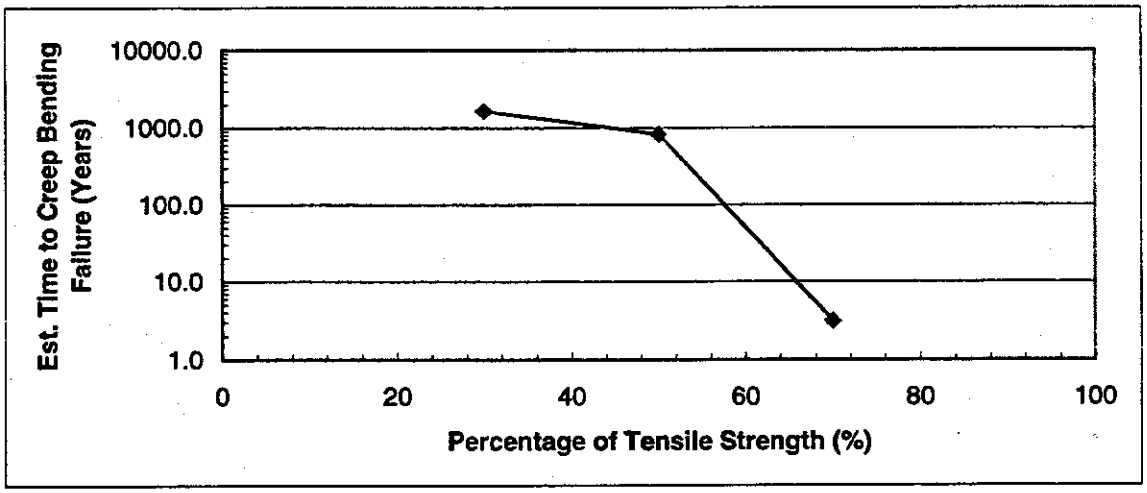


Figure 18. Method for estimating time to failure resulting from bending creep of RPP.

The method to predict creep failure detailed above may be conservative since it is entirely based on laboratory tests in which a single point load was used to generate the creep deformation. The data in Table 7 show that the creep deformation is much slower for similar specimens loaded with five equally distributed 10-lb weights than for single point loaded specimens. In fact, the estimated time to failure (at 70°F) for the laboratory pins with distributed loads is 6000 years or six times that estimated for point loaded specimens. The loading condition in the field is much closer to distributed loading than to point loading, thus the proposed method may be conservative in its lifetime estimates for the pins.

Finally, creep is due to sustained loading of a material. In the laboratory tests, the scaled RPPs were maintained under a constant load until failure. In slope stabilization applications, the loading may not be constant but rather cyclical in nature. During periods of wet weather, the water content of the soils increase and pore water pressures increase. These factors contribute to decreased soil strength and increased load driving the slope toward movement. Under these conditions, the RPPs will become loaded and act to resist the weight of the soil tending toward sliding. During dry periods, water content of the soil decreases, pore water pressures decrease, soil strength increases and the destabilizing forces decrease also. At these times, loads on the RPP may decrease. Under this cyclic behavior, which more or less coincides with the climatic seasons, the RPPs may not be subject to sustained loading for more than about 6 months at a time. This will further reduce any likelihood of failure due to creep of the RPPs. Ongoing monitoring of the loads in the reinforcing members are expected to reveal whether loading on the pins is cyclical or whether developed loads tend to be maintained once applied.

Conclusions

The primary objective of the project entitled "Slope Stabilization Using Recycled Plastic Pins" was to demonstrate the constructability of a stabilization scheme using recycled plastic pins (RPPs) by stabilizing a single slope at a site representative of MoDOT slope failures. Two slides on a continuous slope were stabilized using the RPP technology. Numerous valuable lessons have been learned including:

Constructability:

- RPPs are very durable in hard driving conditions.
- Perpendicular-to-face driving was less problematic in the field due to equipment constraints.
- The mast driving system was much more effective, accurate, and required less skill to operate.
- To maintain peak driving efficiency, a guided (mast-type) driving system is required.
- Average installation rates, including set up time, ranged from 70 to 100ft/hour (20 to 30 m/hour).

Material Properties:

- Tensile strength varied between the two different production runs of the recycled plastic pins.
- Tensile strength of RPPs was 1.3 ksi to 1.8 ksi.
- Compressive strength of RPPs was 3.0 ksi.
- RPPs are durable in harsh environments (acidic, ultraviolet, tap water, freeze/thaw).
- RPPs submersed in kerosene for one year showed a reduction in strength.
- Bending creep may not be critical in the field. Estimated time to failure under sustained loading in the field condition is around 1000 years for the Emma site.

Design Methodology:

- Developed limit-state design procedure for pin-reinforced slopes.
- Procedure does not incorporate group or interaction effects. Such behavior may increase the effectiveness of the reinforcing but requires fundamental analysis and testing to evaluate.
- Critical parameters for design are: depth of sliding surface, pin capacity, and pin spacing.
- Notable finding was that soil strength has only a secondary influence on stability of reinforced slopes.
- Improvement in stability is similar whether using RPPs (weak reinforcement) or steel (strong reinforcement) members of the same size and spacing.

Preliminary Economic Comparison:

- Cost of RPP stabilization (Emma site) was slightly less than estimated costs for rock armor and many times less than that for traditional soil nailing.

Recommendations

The constructability of using recycled plastic pins for stabilizing a slope failure were demonstrated through the tasks of this project. However, there remain three areas that must be addressed for the technology to become widely adopted and implemented. The areas include: applicability, validation and economics.

This project demonstrated that it is feasible to install RPPs into a slope and to stabilize the soil from sliding. What is needed now is to demonstrate that the technology is applicable to a range of conditions, soil types, site geometries, and to note any installation equipment limitations. This can be accomplished by using RPP technology at a number of sites with varying site conditions.

The second area is validation of the design of the RPP stabilization system. The design methodology used for the Emma site (reported herein) has lead to a stable slope; however, without instrumenting, monitoring and analyzing the performance, one has little idea of whether the design is conservative or possibly unconservative. Performance data is being collected for the Emma site; however, data must be collected for differing site conditions and designs, and analyses performed to validate or confirm the design methodology.

Finally, although the economics for the Emma site were very favorable (less costly than conventional stabilization methods), economic analyses of other installations are necessary in order to evaluate the potential for variations in costs. It is expected that the unit cost to stabilize slopes using the RPP technology will decrease as engineers, contractors and agencies become familiar with the design and construction techniques.

References

De Beer, E., and R. Carpentier (1977), "Discussion of 'Methods to Estimate Lateral Force Acting on Stabilizing Piles,' by T. Ito and T. Matsui," *Soils and Foundations*, Vol. 17, No. 1, pp. 68-82.

Hull, T.S., and H.G. Poulos (1999), "Discussion of 'Design Method for Stabilization of Slopes with Piles,' by S. Hassiotis, J.L. Chameau, and M. Gunaratne," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 125, No. 10, pp. 911-913.

Ito, T., and T. Matsui (1975), "Methods to Estimate Lateral Force Acting on Stabilizing Piles," *Soils and Foundations*, Vol. 15, No. 4, pp. 43-59.

Koerner RM, Halse YH and Lord AE (1990) "Long-term durability and aging of geomembranes," Waste Containment Systems: Construction, Regulation and Performance, Geotechnical Special Publication No. 26, pp 106-134.

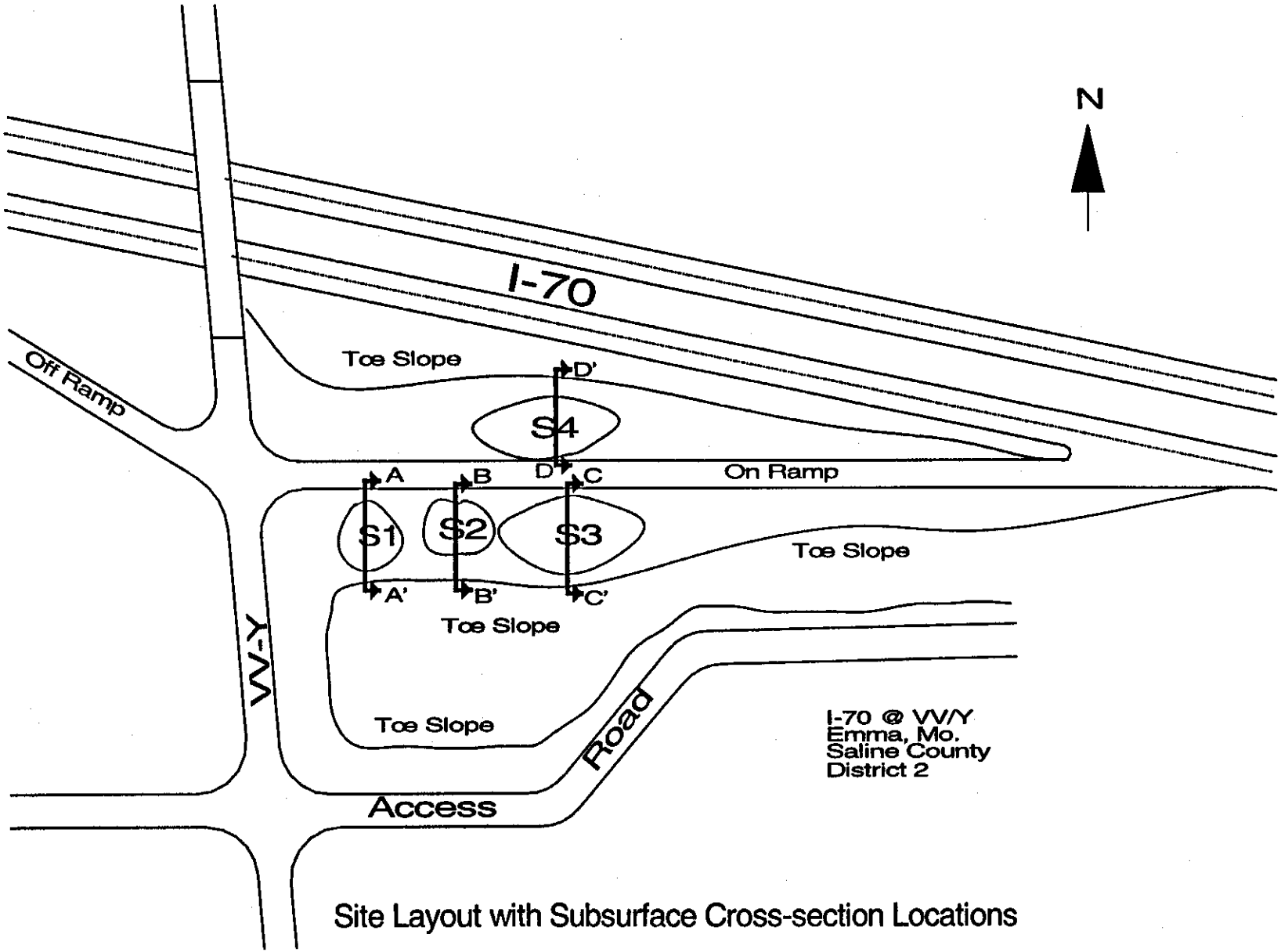
National Climatic Data Center (2000), *Weather Observation Station Records*, http://www.ncdc.noaa.gov/ol/ncdc.html.

Principal Investigator and Project Members

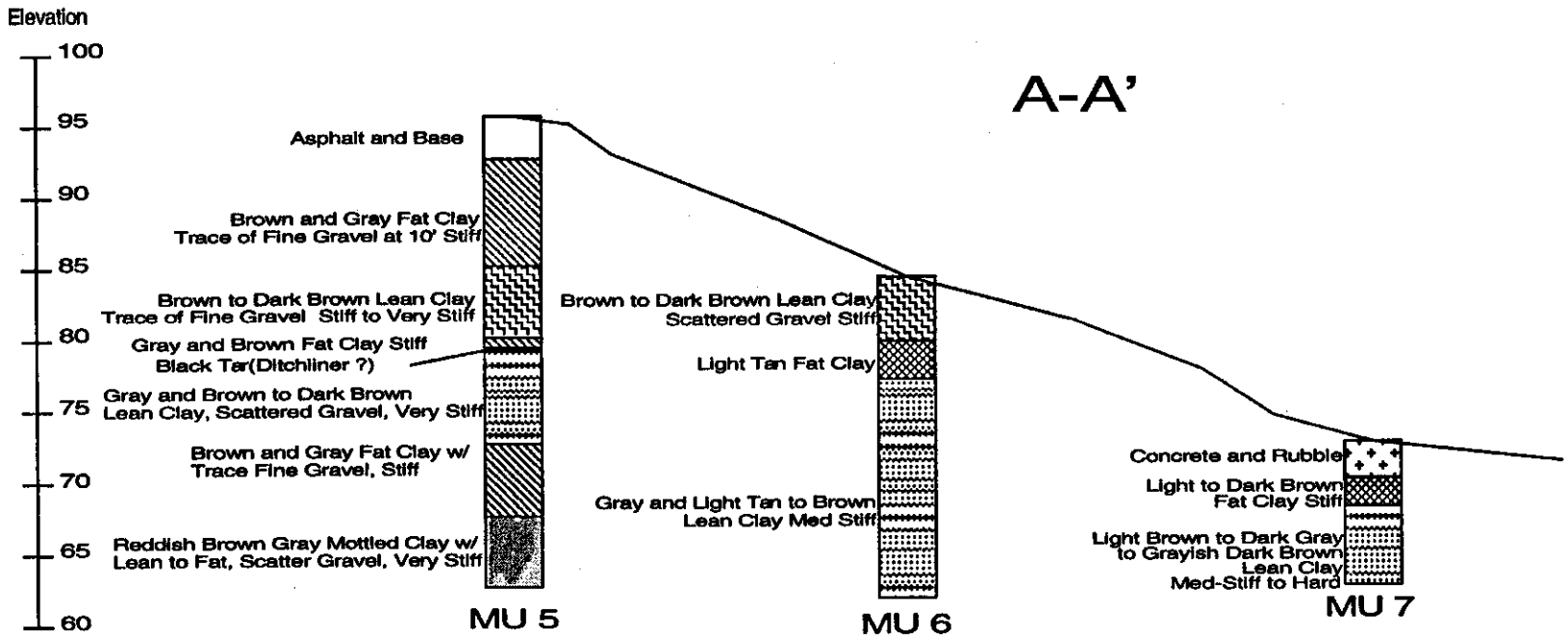
Name	Title	Responsibility
John Bowders	Associate Professor of Civil Engineering	PI, Overall project direction, reporting and field implementation
Erik Loehr	Assistant Professor of Civil Engineering	Co-PI, Development of design methodology, stability analyses, laboratory testing
Hani Salim	Assistant Professor of Civil Engineering	Co-PI, Materials properties testing & evaluation.
Todd Mooney	Research Assistant Professor of Civil Engineering	Res. Associate, Field instrumentation and performance monitoring

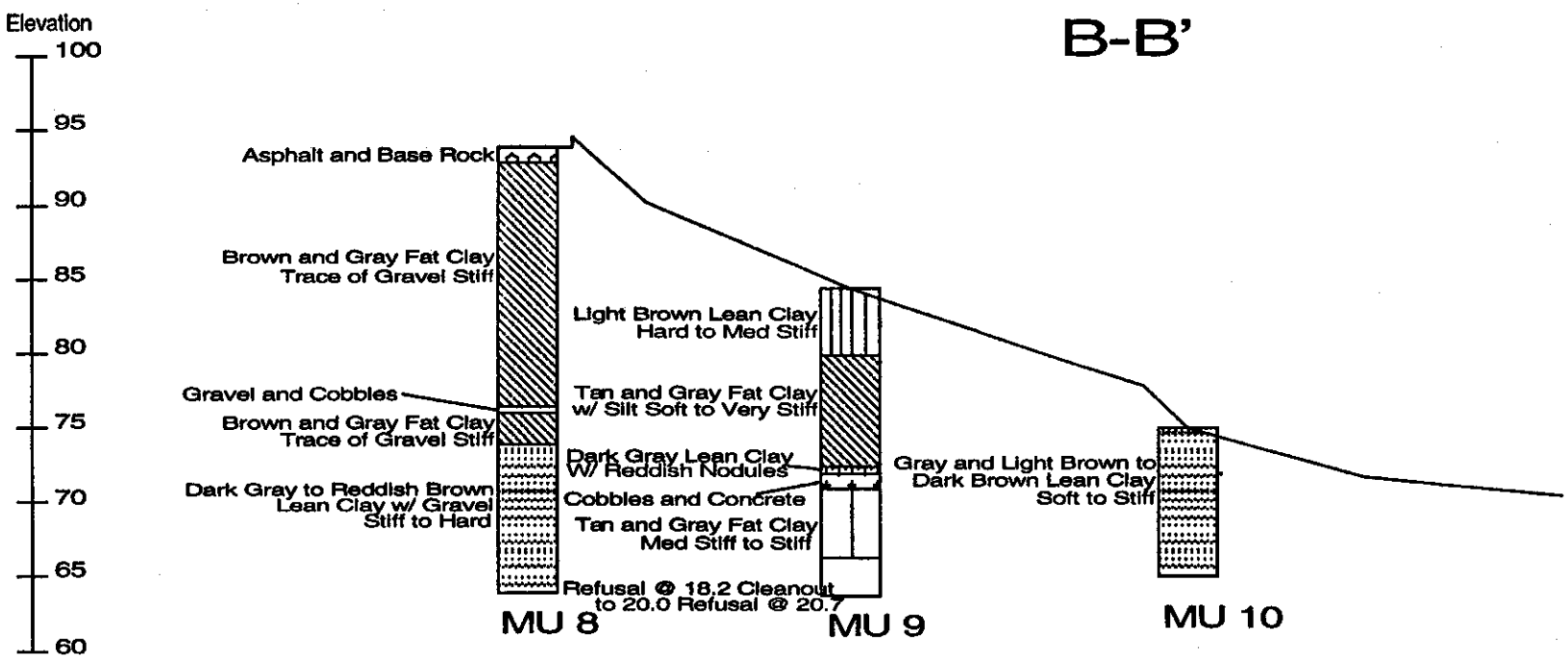
Appendix A –

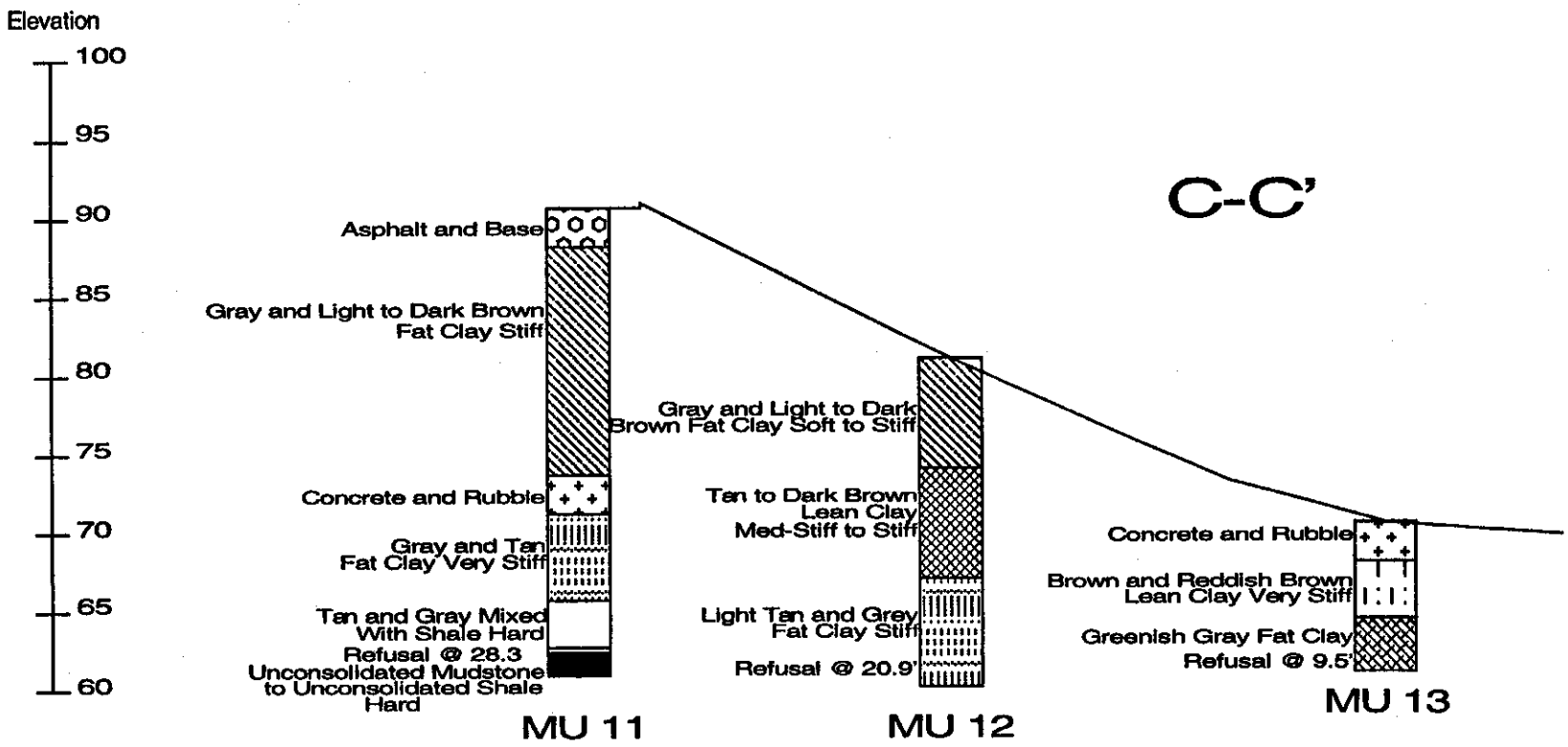
Emma Site Boring Logs and Subsurface Cross-sections

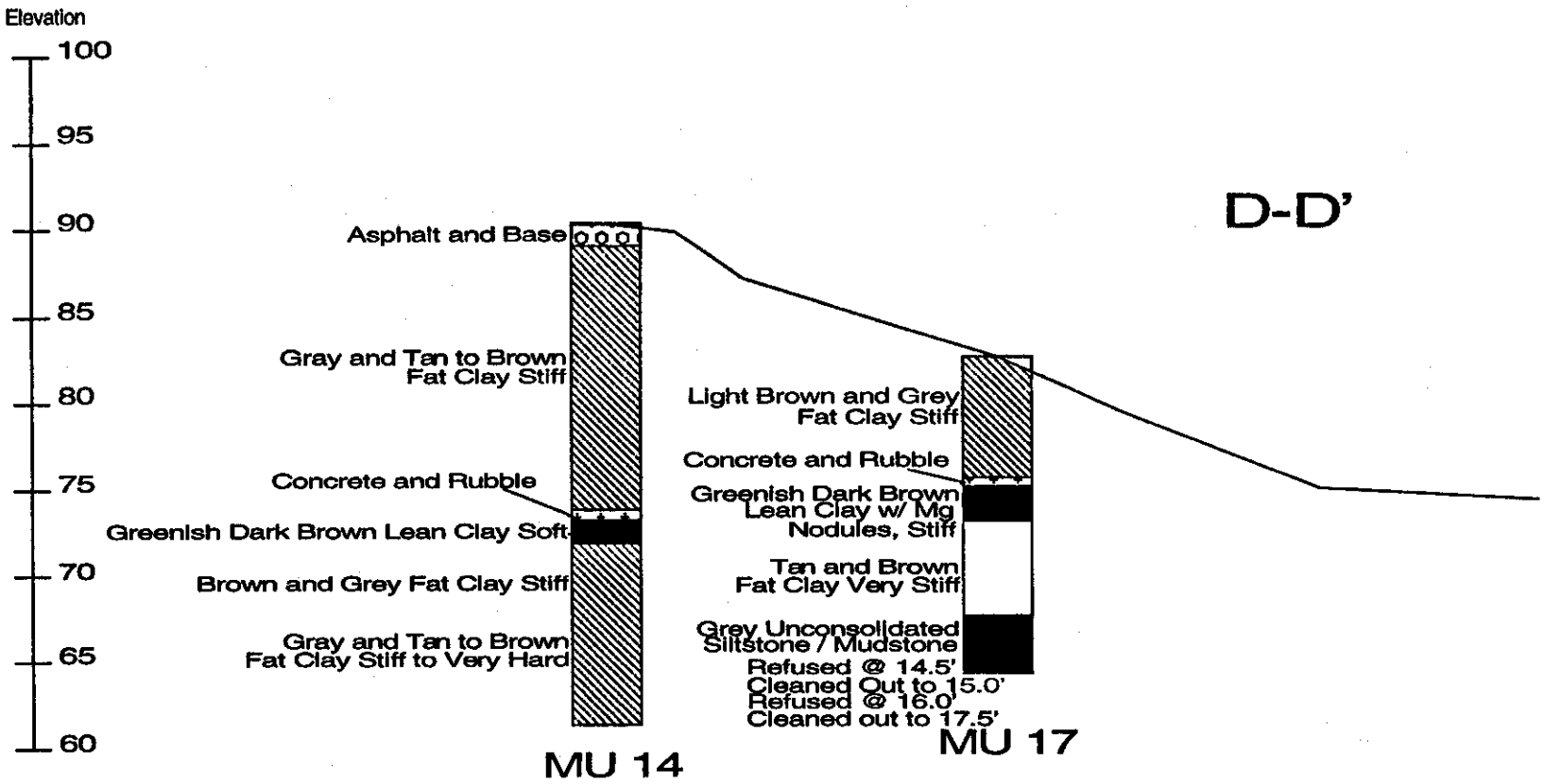


Site Layout with Subsurface Cross-section Locations


A-A'
Elevation
100
95
90
85
80
75
70
65
60
Asphalt and Base
Brown and Gray Fat Clay
Trace of Fine Gravel at 10' Stiff
Brown to Dark Brown Lean Clay
Trace of Fine Gravel Stiff to Very Stiff
Gray and Brown Fat Clay Stiff
Black Tar(Ditchliner ?)
Gray and Brown to Dark Brown
Lean Clay, Scattered Gravel, Very Stiff
Brown and Gray Fat Clay w/
Trace Fine Gravel, Stiff
Reddish Brown Gray Mottled Clay w/
Lean to Fat, Scatter Gravel, Very Stiff
MU 5
Brown to Dark Brown Lean Clay
Scattered Gravel Stiff
Light Tan Fat Clay
Gray and Light Tan to Brown
Lean Clay Med Stiff
MU 6
Concrete and Rubble
Light to Dark Brown
Fat Clay Stiff
Light Brown to Dark Gray
to Grayish Dark Brown
Lean Clay
Med-Stiff to Hard
MU 7


B-B'
Elevation
100
95
90
85
80
75
70
65
60
Asphalt and Base Rock
Brown and Gray Fat Clay
Trace of Gravel Stiff
Gravel and Cobbles
Brown and Gray Fat Clay
Trace of Gravel Stiff
Dark Gray to Reddish Brown
Lean Clay w/ Gravel
Stiff to Hard
MU 8
Light Brown Lean Clay
Hard to Med Stiff
Tan and Gray Fat Clay
w/ Silt Soft to Very Stiff
Dark Gray Lean Clay
W/ Reddish Nodules
Cobbles and Concrete
Tan and Gray Fat Clay
Med Stiff to Stiff
Refusal @ 18.2 Cleanout
to 20.0 Refusal @ 20.7
MU 9
Gray and Light Brown to
Dark Brown Lean Clay
Soft to Stiff
MU 10


C-C'
Elevation
100
95
90
85
80
75
70
65
60
Asphalt and Base
Gray and Light to Dark Brown Fat Clay Stiff
Concrete and Rubble
Gray and Tan Fat Clay Very Stiff
Tan and Gray Mixed With Shale Hard
Refusal @ 28.3
Unconsolidated Mudstone to Unconsolidated Shale Hard
MU 11
Gray and Light to Dark Brown Fat Clay Soft to Stiff
Tan to Dark Brown Lean Clay Med-Stiff to Stiff
Light Tan and Grey Fat Clay Stiff
Refusal @ 20.9'
MU 12
Concrete and Rubble
Brown and Reddish Brown Lean Clay Very Stiff
Greenish Gray Fat Clay
Refusal @ 9.5'
MU 13


D-D'
Elevation
100
95
90
85
80
75
70
65
60
Asphalt and Base
Gray and Tan to Brown
Fat Clay Stiff
Concrete and Rubble
Greenish Dark Brown Lean Clay Soft
Brown and Grey Fat Clay Stiff
Gray and Tan to Brown
Fat Clay Stiff to Very Hard
MU 14
Light Brown and Grey
Fat Clay Stiff
Concrete and Rubble
Greenish Dark Brown
Lean Clay w/ Mg
Nodules, Stiff
Tan and Brown
Fat Clay Very Stiff
Grey Unconsolidated
Siltstone / Mudstone
Refused @ 14.5'
Cleaned Out to 15.0'
Refused @ 16.0'
Cleaned out to 17.5'
MU 17

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation - 95.01'		Wn %	LL	PI	AASHTO Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	φ°	c psf	Consolidation Data: P_o ksf	P_c ksf	P_f ksf	e_o	e_c	e_f	C_c	C_v ft²/d
0.0-3.0' Asphalt and base rock.																			
3.0-10.0' Brown and gray fat clay, stiff to very stiff.																			
	@ 5.0'	35	50	30		1.00	0.50												
-Trace fine gravel below 10.0'.	@ 9.0'	35				2.50	0.50+												
10.0-15.0' Brown lean clay, trace fine gravel, stiff.	@ 11.0'	30				1.75	0.50												
-Dark brown below 14.0'.	@ 14.1'	33				2.00	0.50												
15.0-19.0' Gray and brown fat clay, medium stiff. -Black tar paper from 19.2 to 19.5'.	@ 15.2'	–				1.75	–												
	@ 18.5'	–				1.00	0.50												
19.0-23.0' Gray and brown lean clay, scattered fine gravel, very stiff.																			
	@ 19.0'	21				2.50	0.50+												
	@ 21.5'	21				2.50	0.50+												
-Dark brown below 22.4'.	@ 22.7'	–				3.00	0.50												
23.0-28.0' Brown and gray fat clay, trace fine gravel, stiff to very stiff.																			
	@ 24.5'	24				1.75	0.50+												
	@ 26.0'	–				2.75	0.50+												
28.0-33.0' Reddish-brown gray mottled lean to fat clay, very stiff to hard.																			
	@ 28.3'	22				3.00	0.50+												
	@ 31.0'	20				4.00	0.50												
Termination of boring at 33.0'.																			

*Elev. 100 = Benchmark at GE end of bridge on top of wing wall.

JOB NO.: R102-007
COUNTY: Bullitt
ROUTE: I-65
STATION: 201.6
GEN. LOC.: Eastbound Access Ramp at Rte. V & Rte. W
FILL HEIGHT:

Keys:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- Δ Slow or drained direct shear test
- ⊠ Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 1 of 18

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 64.38'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: ϕ'	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: c_v 10^{-4} ft²/d
0.0-3.0' Brown lean clay, with scattered gravel, hard.	@ 1.3'	30				4.00	0.75												
3.0-4.0' Dark brown lean clay, stiff.	@ 3.0'	23	30	13		2.00	0.40												
4.0-7.2' Light tan lean clay, soft, moist.	@ 6.1'	27	40	30		0.20	0.40												
7.2-9.0' Dark grayish-brown fat clay, stiff, moist.	@ 9.0'	37	50	30		2.00	0.00												
9.0-12.7' Light tan and gray lean clay, stiff to very stiff, moist.	@ 11.3'	21				3.00	0.00+												
	@ 12.1'	25				1.70	0.55												
12.7-17.0' Dark grayish-brown lean clay, medium stiff to stiff, moist.	@ 14.0'	--				4.00	0.75												
	@ 15.0'	25				0.00	0.45												
17.0-22.0' Light brown lean clay, stiff, moist.	@ 16.0'	37				1.00	0.00												
	@ 21.3'	33				1.00	0.55												
Termination of boring at 22.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: RI93-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 0
GEN. LOC.: E.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
▽ Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane" torsional shear device
S Slow or drained direct shear test
R Consolidated undrained triaxial shear test
R R test corrected for pore pressure

FIGURE 2 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation = 72.7'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tx tsf	Qu/2 psf	Shear Test Data: Type Test	B*	C psf	Consolidation Data: P_1 ksf	P_4 ksf	P_2 ksf	e_1	e_4	e_2	C_c	C_c 10^{-4} R'M
0.0-2.5' Concrete and rubble.																			
2.5-4.5' Light brown fat clay, stiff, moist. -Dark brown below 3.5'.	@ 4.75'	30				1.00	0.50												
4.5-6.7' Light brown lean clay, stiff, moist.																			
6.7-7.5' Dark gray lean clay, hard, moist.	@ 6.5'	31				4.00	0.50												
7.5-10.0' Dark grayish-brown lean clay, medium stiff, moist.	@ 9.1'	30				--	--												
Termination of boring at 10.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R/96-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 7
GEN. LOC.: E.B. Access Ramp at Rte. Y & Rte. W
FILL HEIGHT:

Key:
- $\bar{Y}$ Water Table
- P.P. "Pocket Penetrometer"
- Tx. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 3 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation = 73.7'	Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	B*	C psf	Consolidation Data: P_1 ksf	P_2 ksf	P_3 ksf	S_1	S_4	S_2	C_c	C_v (10^{-4} ft²/s)
0.0-2.5' Concrete and rubble.																		
2.5-4.5' Light brown fat clay, stiff, moist. -Dark brown below 3.5'.																		
4.5-6.7' Light brown lean clay, stiff, moist. @ 6.75'	20				1.50	0.50												
6.7-7.5' Dark gray lean clay, hard, moist. @ 6.5'	21				4.50	5.50												
7.5-10.0' Dark grayish-brown lean clay, medium stiff, moist. @ 9.1'	20				–	–												
Termination of boring at 10.0'.																		

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R[illegible]-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 7
GEN. LOC.: S.B. Access Ramp at Rte. V & Rte. W
FILL HEIGHT:

Key:
- V Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 3 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation - 83.58'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: Ø"	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: d_v 10^{-3} ft²/d
0.5-1.0' Asphalt and base rock.																			
1.0-20.0' Brown and gray fat clay, trace gravel, stiff to very stiff.																			
	@ 3.0'	24				1.75	0.92												
	@ 5.0'	24				1.75	0.92												
	@ 8.0'	28				2.00	0.95+												
	@ 13.0'	26				2.00	0.99												
	@ 16.0'	–				2.75	0.95+												
-Gravel and cobbles from 17.5 to 17.0'.	@ 18.0'	24				2.00	0.99												
20.0-22.0' Dark gray lean clay, very stiff.	@ 21.0'	–				2.25	0.95+												
22.0-29.0' Dark gray lean clay, with gravel, some organics, hard to stiff.	@ 23.0'	–				4.00	0.40												
	@ 29.0'	–				1.00	0.45												
29.0-30.0' Reddish-brown lean clay, with gravel, stiff to very hard.	@ 30.0'	27				1.00	0.30												
	@ 30.0'	–				0.00	0.95+												
Termination of boring at 30.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R196-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 8
GEN. LOC.: S.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 4 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 54.37'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: ϕ'	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_4	Consolidation Data: e_2	Consolidation Data: C_1	Consolidation Data: a_v $(10^{-4}\ ft^2/d)$
0.0-4.0' Light brown lean clay, hard to medium stiff, moist.																			
	@ 1.5'	10				4.00	0.00+												
	@ 3.0'	20				0.50	0.00												
4.0-12.0' Tan and gray fat clay, soft, moist.																			
	@ 6.5'	30	62	31		0.50	0.40												
-Very stiff below 7.0'.																			
	@ 9.3'	32				2.25	0.00+												
	@ 11.0'	34				2.25	0.00+												
12.0-13.0' Dark gray lean clay, moist.																			
13.0-13.5' Cobbles and concrete.																			
13.5-30.7' Gray and tan fat clay, medium to stiff, moist.																			
	@ 16.3'	31				0.50	0.50												
	@ 17.5'	36				1.00	0.45												
-Shelby tube refusal at 18.3'.																			
-Shelby tube refusal at 20.7'.	@ 20.4'	32				2.00	0.75												
Termination of boring at 20.7'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R150-007
COUNTY: Saline
ROUTE: I-79
STATION: MU 9
GEN. LOC.: E.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 6 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation ~ 74.16'	Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: ϕ'	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: a_1	Consolidation Data: a_2	Consolidation Data: a_3	Consolidation Data: C_v	Consolidation Data: C_v 10^{-4} R^2/d
0.0-2.0' Light brown lean clay, soft, moist.																		
@ 1.0'	27				0.25	0.40												
2.0-4.0' Dark brown lean clay, stiff, moist.																		
@ 3.3'	26				1.50	0.50												
4.0-10.0' Light brown and gray lean clay, stiff, moist.																		
@ 6.5'	26				2.00	0.80												
@ 9.0'	25				1.00	–												
Termination of boring at 10.0'.																		

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R/98-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 10
GEN. LOC.: E.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 6 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation = 96.10'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: Ø'	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: e_v (ft^2/d)
0.0-2.0' Asphalt and base rock.																			
2.0-7.0' Light brown and gray fat clay, stiff, moist.	@ 3.0'	19				1.50	0.70												
	@ 6.0'	24				1.50	0.60												
7.0-8.2' Gray and brown fat clay, stiff, moist.	@ 7.0'	38	57	37		1.50	0.50												
8.2-13.1' Brown fat clay, stiff to medium stiff, moist.																			
	@ 11.3'	33				1.00	0.00												
	@ 12.0'	–				0.75	0.05												
13.1-17.0' Dark brown and gray fat clay, very stiff, moist.	@ 14.0'	22				3.25	0.00+												
	@ 16.0'	26				2.00	0.00+												
17.0-19.0' Concrete.																			
19.0-26.0' Gray and tan fat clay, very stiff, moist.																			
	@ 21.3'	20				3.25	0.00+												
	@ 23.0'	18				3.0	0.00+												
26.0-29.0' Tan and gray weathered shale, hard to very hard, moist. -Shelby tube refusal at 26.7'. -SPT at 26.7-27.2', N_{60} = 61.	@ 26.4'	13				0.0+	0.50+												
	@ 26.5'	14				–	–												
	@ 27.0'	16				–	–												
-Shelby tube refusal at 28.2'. -SPT at 28.2-29.0', N_{60} = 72.	@ 29.0'	14				–	–												
Termination of boring at 29.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R96-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 11
GEN. LOG.: E.B. Access Ramp at Rte. Y & Rte. VV
FILL HEIGHT:

Key:
- Y Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 7 of 19

Sheet 1 of 1

Depth	Depth and Description	Elevation – 51.11'	Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: B*	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: c_v ($ft^2/ft^2/d$)
0	0.0-4.5' Light brown and gray fat clay, stiff to soft, moist.																			
		@ 1.3'	30				1.00	0.80												
		@ 3.5'	30	84	31		0.50	0.40												
5	4.5-7.5' Dark brown fat clay, very stiff, moist.																			
		@ 5.0'	25				2.75	0.90+												
	7.5-9.5' Light brown to tan lean clay, stiff, moist.																			
		@ 9.3'	23				1.00	0.70												
10	9.5-14.5' Dark brown lean clay, medium stiff to stiff, moist.																			
		@ 11.3'	27	33	10		0.50	0.30												
		@ 13.3'	23				2.00	0.90+												
15	14.5-20.5' Light tan and gray fat clay, stiff to very stiff, moist.																			
		@ 15.3'	21				2.00	0.80												
		@ 16.5'	21				1.00	0.40												
20	-Shelby tube refusal at 20.5'.	@ 20.5'	16				2.25	0.90												
	Termination of boring at 20.5'.																			
25																				
30																				
35																				
40																				

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R/96-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 18
GEN. LOG.: E.S. Access Ramp at Rte. Y & Rte. VV
FILL HEIGHT:

Key:
- W — Water Table
- P.P. — "Pocket Penetrometer"
- Tv. — "Torvane" torsional shear device
- S — Slow or drained direct shear test
- R — Consolidated undrained triaxial shear test
- R — R test corrected for pore pressure

FIGURE 8 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 76.02'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: ϕ'	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: C_v 10^{-4} ft²/d
0.0-2.0' Rubble and concrete.																			
2.0-6.1' Brown and reddish-brown lean clay, stiff, moist.	@ 3.0'	37				2.00	0.70												
	@ 5.0'	30				1.00	0.40												
6.1-8.0' Greenish-gray fat clay, hard, moist.	@ 8.0'	19				5.00	0.90+												
Termination of boring at 8.0'.																			

*Elev. 100 = Benchmark at SR end of bridge on top of wing wall.

JOB NO.: R106-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 19
GEN. LOC.: S.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 9 of 10

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation = 96.13'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	B'	C psf	Consolidation Data: P_1 ksf	P_2 ksf	P_3 ksf	e_1	e_2	e_3	C_c	c_v 10^{-4} ft²/d
0.0-1.3' Asphalt and base rock.																			
1.3-9.2' Brown and gray fat clay, stiff, moist.	@ 0.0'	–				1.25	0.75												
	@ 0.3'	30				1.00	0.75												
	@ 0.4'	–				1.75	0.90												
9.2-11.5' Brown fat clay, stiff, moist	@ 9.5'	–				1.25	0.90												
	@ 10.7'	–				1.75	0.95+												
-Tan and brown below 11.4'.	@ 11.0'	23				–	–												
11.5-16.5' Brown and gray fat clay, stiff, moist.	@ 12.2'	–				1.75	0.95+												
	@ 13.0'	30				2.00	0.90												
	@ 15.0'	–				1.00	0.90												
16.5-17.1' Concrete and gravel.																			
17.1-18.2' Greenish-dark brown lean clay, stiff, moist.	@ 18.0'	31				1.00	0.90												
18.5-21.0' Light brown and gray fat clay, very stiff, moist.	@ 20.0'	–				3.50	0.90+												
21.0-24.0' Gray and tan fat clay, very stiff to very hard, moist.	@ 21.0'	23				2.75	0.90+												
-SPT at 22.5-24.0', N_{60} = 30.	@ 23.3'	–				0.0+	–												
Termination of boring at 24.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: RJ00-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 14
GEN. LOC.: E.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
V Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane" torsional shear device
S Slow or drained direct shear test
R Consolidated undrained triaxial shear test
R R test corrected for pore pressure

FIGURE 10 of 10

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 12.34'		Wn %	LL	PI	AASHTO Class	P.P. tsf	Tv. tsf	Qu/2 tsf	Shear Test Data: Type Test	φ'	c' tsf	Consolidation Data: P_1 tsf	P_2 tsf	P_c tsf	e_1	e_c	e_2	C_c	P_c 10^{-3} ft²/d
0.0-1.0' Asphalt and base rock.																			
1.0-10.0' Grey and brown fat clay, trace fine gravel, stiff to very stiff.	3.0'	–				1.75	0.60												
	@ 6.3'	37	90	55		2.75	0.80+												
	@ 8.0'	–				1.75	0.80												
	@ 11.3'	31				2.00	0.80												
-Scattered gravel below 10.0'.	@ 13.0'	–				2.75	0.80+												
	@ 14.3'	30				2.00	0.80+												
10.0-20.0' Dark brown lean clay, hard to medium stiff, with scattered gravel layers. -Shelby tube refusal at 18.0'. -SPT at 20.0-21.0', N_{60} = 6.	@ 16.7'	–				0.50	0.80+												
	@ 20.0'	–				0.75	–												
20.0-25.0' Reddish-brown lean clay, trace fine gravel, hard.	@ 23.0'	21				4.00	0.50												
25.0-27.0' Gray shaley clay, scattered gravel, hard. -Shelby tube refusal at 27.0'. -SPT at 27.0-28.0', N_{60} = 50.	@ 26.0'	14				0.60	0.50+												
27.0-29.0' Red weathered sandstone, dense.	@ 28.0'	–				0.50	–												
Termination of boring at 29.0'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R200-007
COUNTY: Rutland
ROUTE: I-70
STATION: MU 10
BRA. LOC.: 1.5, Access Ramp M Rp. Y & Rp. W
FILL HEIGHT:

Key:
- ▽ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 11 of 18

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 63.61'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv tsf	Qu/2 psf	Shear Test Data: Type Test	B*	C psf	Consolidation Data: P_1 ksf	P_2 ksf	P_3 ksf	e_1	e_2	e_3	C_c	c_v (10^{-4} ft²/d)
0.0-7.4' Light brown lean clay, medium stiff, moist. -Very stiff from 0.0 to 2.0'.	@ 1.3'	–				2.30	0.90+												
	@ 3.0'	30	40	26		0.75	0.50												
	@ 6.3'	–				0.75	0.50												
7.4-8.0' Concrete rubble.																			
8.0-14.0' Greenish brown and light brown fat clay, very stiff to hard, moist.	@ 11.3'	27				3.00	0.90+												
-Shelby tube refusal at 14.0'.	@ 13.3'	–				3.00	0.90+												
14.0-18.4' Gray to light tan shale, hard to very hard, moist to dry. -Shelby tube refusal at 18.0'.	@ 16.4'	–				4.00	0.90+												
-Shelby tube refusal at 18.4'.	@ 18.0'	–				3.00	0.90+												
Termination of boring at 18.4'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: R/96-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 18
GEN. LOC.: E.B. Access Ramp at Rte. V & Rte. VV
FILL HEIGHT:

Key:
W Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane" torsional shear device
S Slow or drained direct shear test
R Consolidated undrained triaxial shear test
R R test corrected for pore pressure

FIGURE 13 of 13

Sheet 1 of 1

DEPTH AND DESCRIPTION Elevation – 52.53'		Wn %	LL	PI	ASTM Class	P.P. tsf	Tv ft²	Qu/2 psf	Shear Test Data: Type Test	Shear Test Data: ϕ°	Shear Test Data: C psf	Consolidation Data: P_1 ksf	Consolidation Data: P_2 ksf	Consolidation Data: P_3 ksf	Consolidation Data: e_1	Consolidation Data: e_2	Consolidation Data: e_3	Consolidation Data: C_c	Consolidation Data: c_v 10^{-3} ft²/d
0.0-7.0' Light brown and gray fat clay, medium stiff to stiff, moist.	@ 1.3'	22				1.00	0.90+												
	@ 3.5'	24	60	33		0.75	0.70												
	@ 6.0'	23				1.75	0.90												
7.0-7.5' Concrete rubble.																			
7.5-8.0' Dark greenish-brown lean clay, stiff, moist.	@ 8.5'	–				1.75	0.90												
8.5-15.0' Brown to tan fat clay, very stiff, moist.	@ 11.3'	22				2.00	0.90												
-Shelby tube refusal at 14.0'.	@ 13.0'	–				2.75	0.90												
15.0-19.4' Gray shale, very hard to very stiff. -Shelby tube refusal at 15.0'.	@ 15.5'	19				3.0+	0.90												
-Shelby tube refusal at 19.4'.	@ 16.0'	–				3.25	0.90+												
Termination of boring at 19.4'.																			

*Elev. 100 = Benchmark at SE end of bridge on top of wing wall.

JOB NO.: RI90-007
COUNTY: Saline
ROUTE: I-70
STATION: MU 17
GEN. LOG.: E.B. Access Ramp at Rte. Y & Rte. VV
FILL HEIGHT:

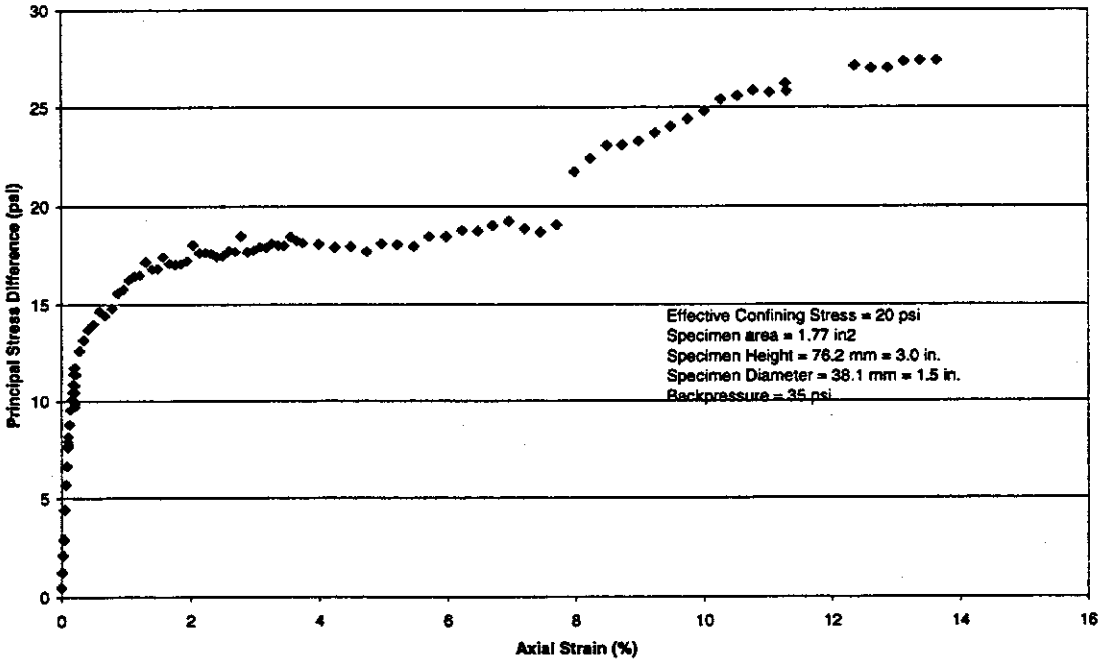
Key:
- $\bar{\phi}$ Water Table
- P.P. "Pocket Penetrometer"
- Tv. "Torvane" torsional shear device
- S Slow or drained direct shear test
- R Consolidated undrained triaxial shear test
- R R test corrected for pore pressure

FIGURE 13 of 13

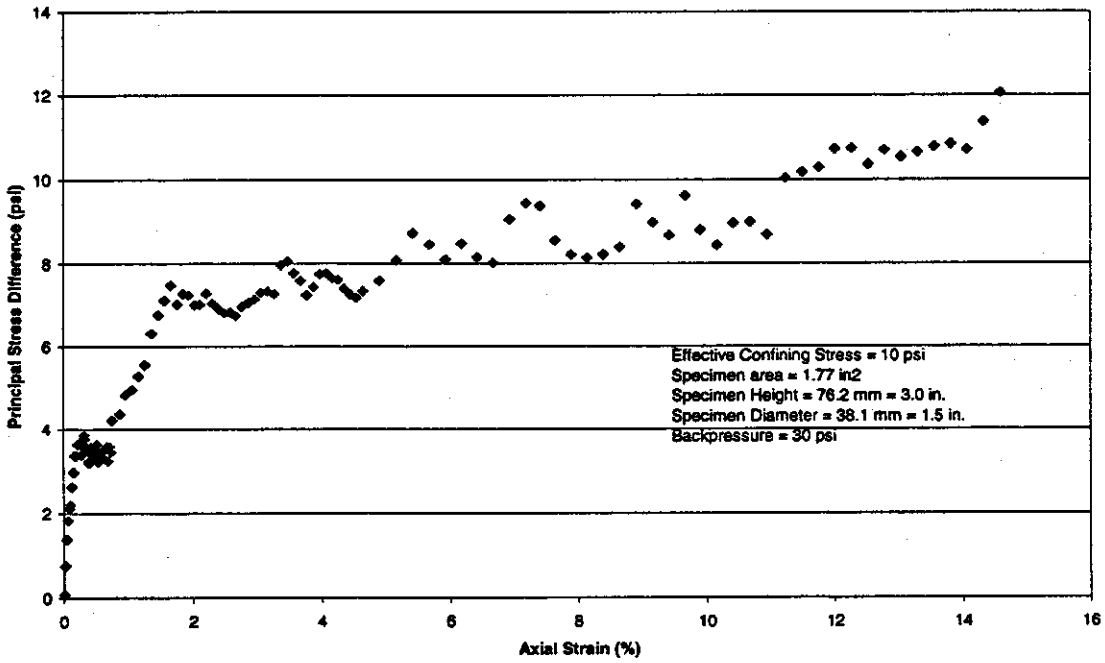
Appendix B –

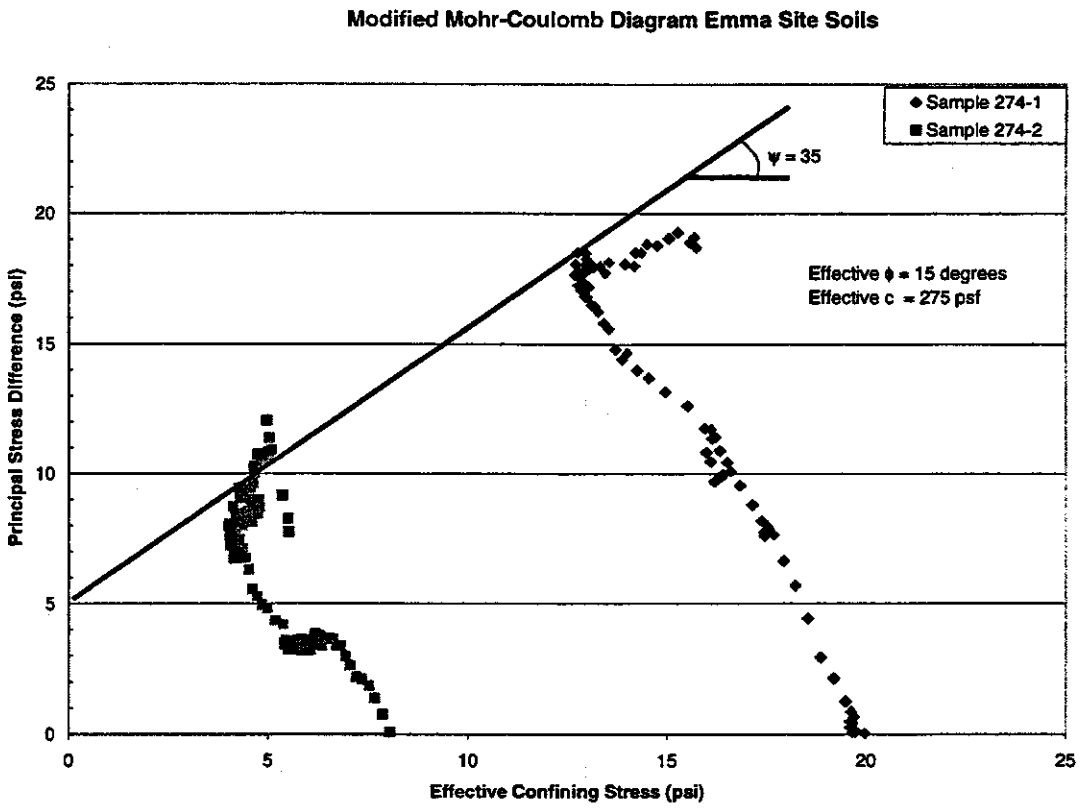
Consolidated-Undrained Triaxial Test Results

Principal Stress Difference vs. Axial Strain Emma Sample 274-1



Principal Stress Difference vs. Axial Strain Emma Sample 274-2

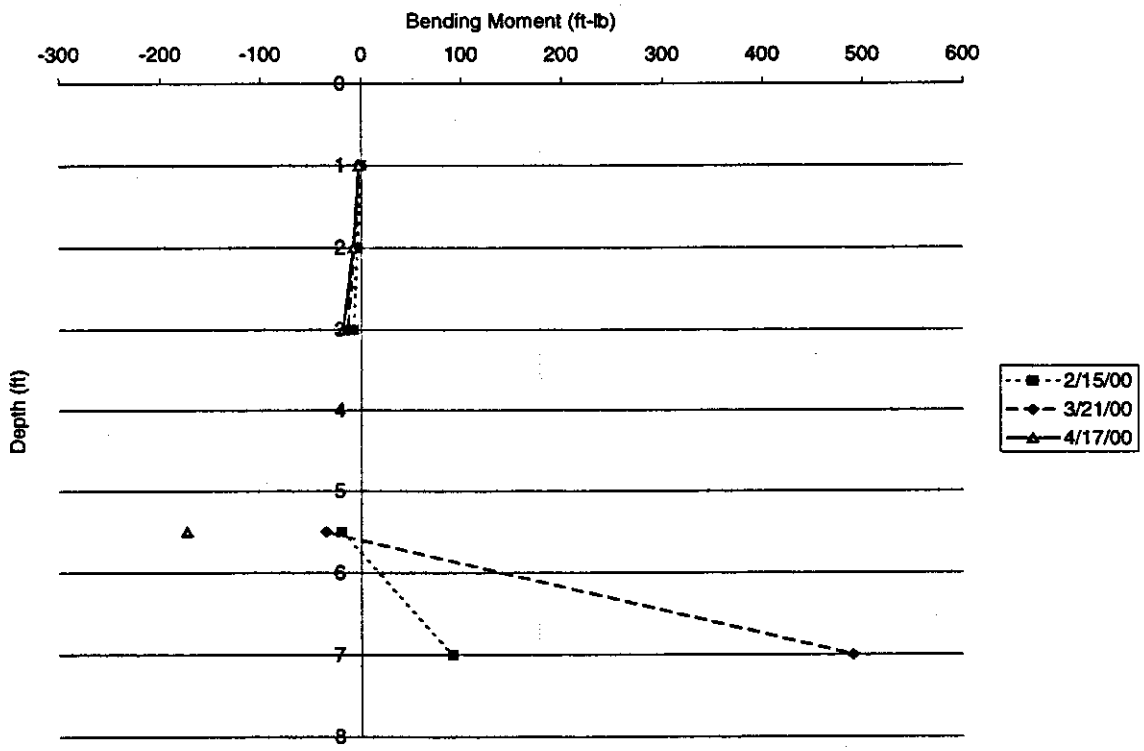



Modified Mohr-Coulomb Diagram Emma Site Soils
Sample 274-1
Sample 274-2
ψ = 35
Effective ϕ = 15 degrees
Effective c = 275 psf
Principal Stress Difference (psi)
Effective Confining Stress (psi)
0
5
10
15
20
25

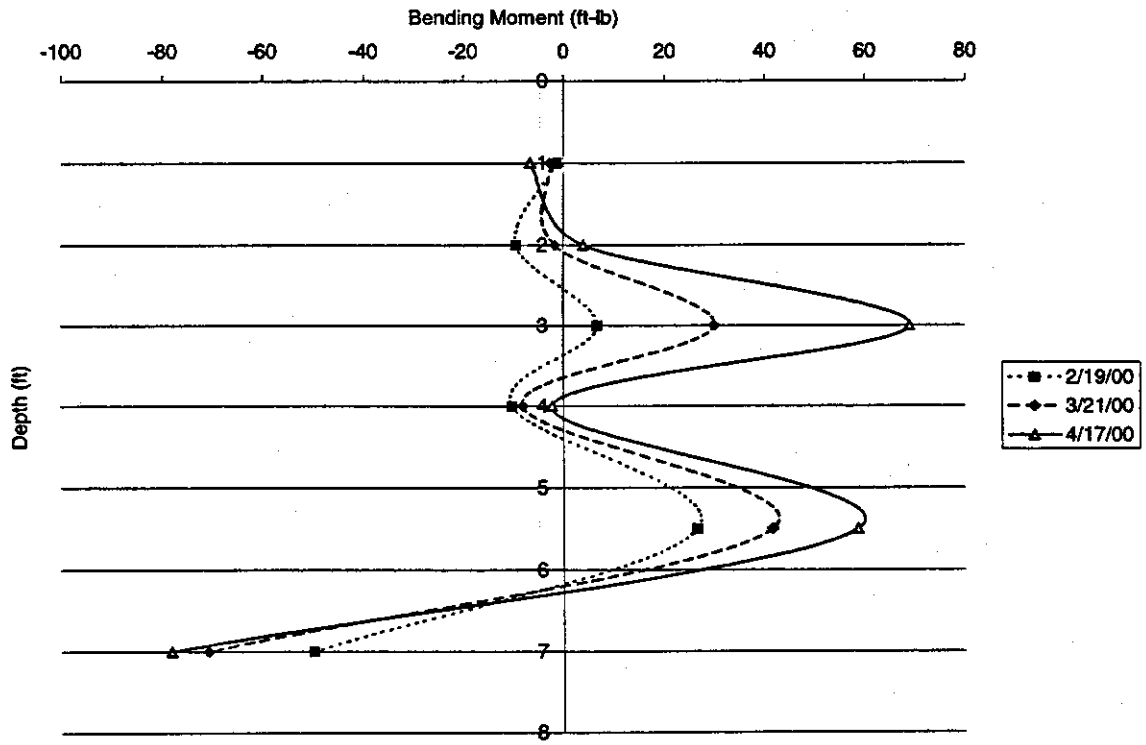
Appendix C –

Measured Bending Moments for Instrumented Pins

Slide S1 Pin Bending Moments:

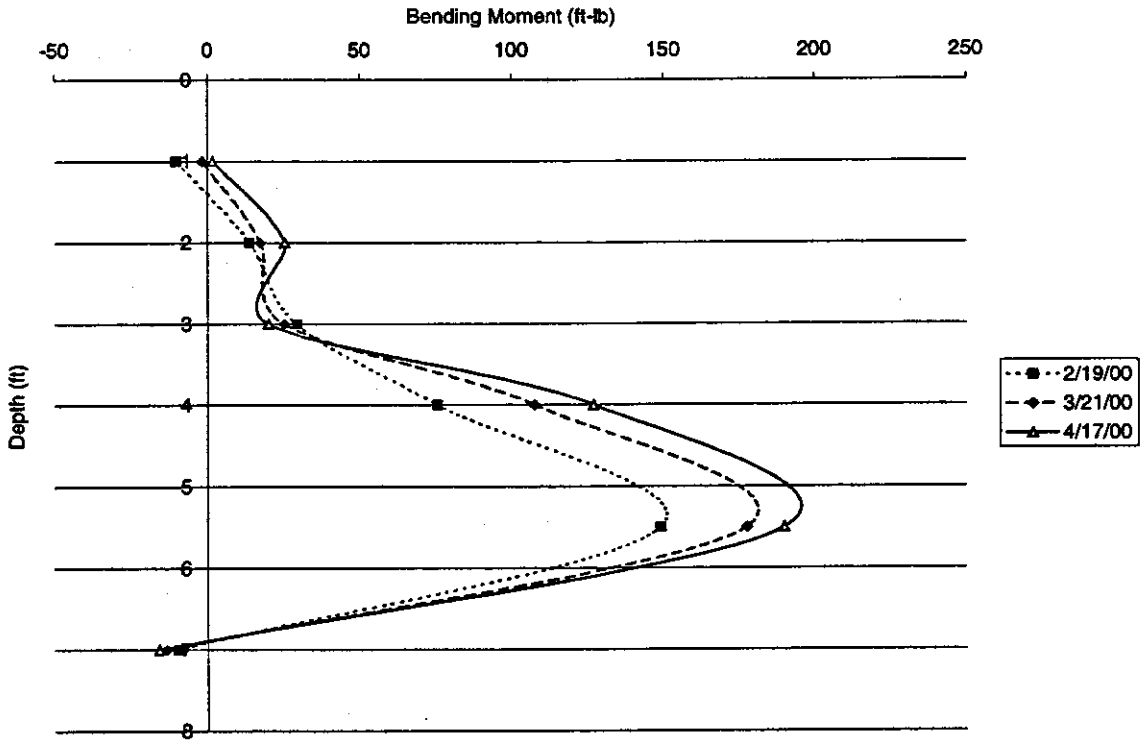


Pin F Bending Moments – Slide S1: Top Center

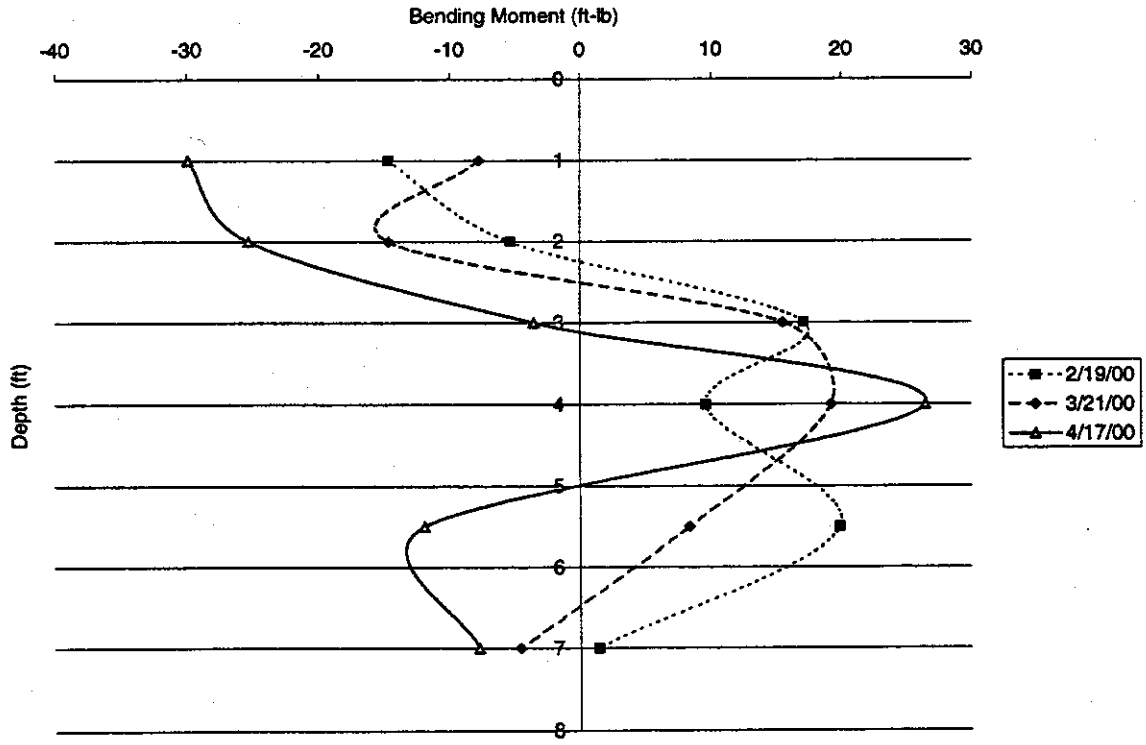


Pin D Bending Moments – Slide S1: Middle Center

Slide S1 Pin Bending Moments (cont'd):

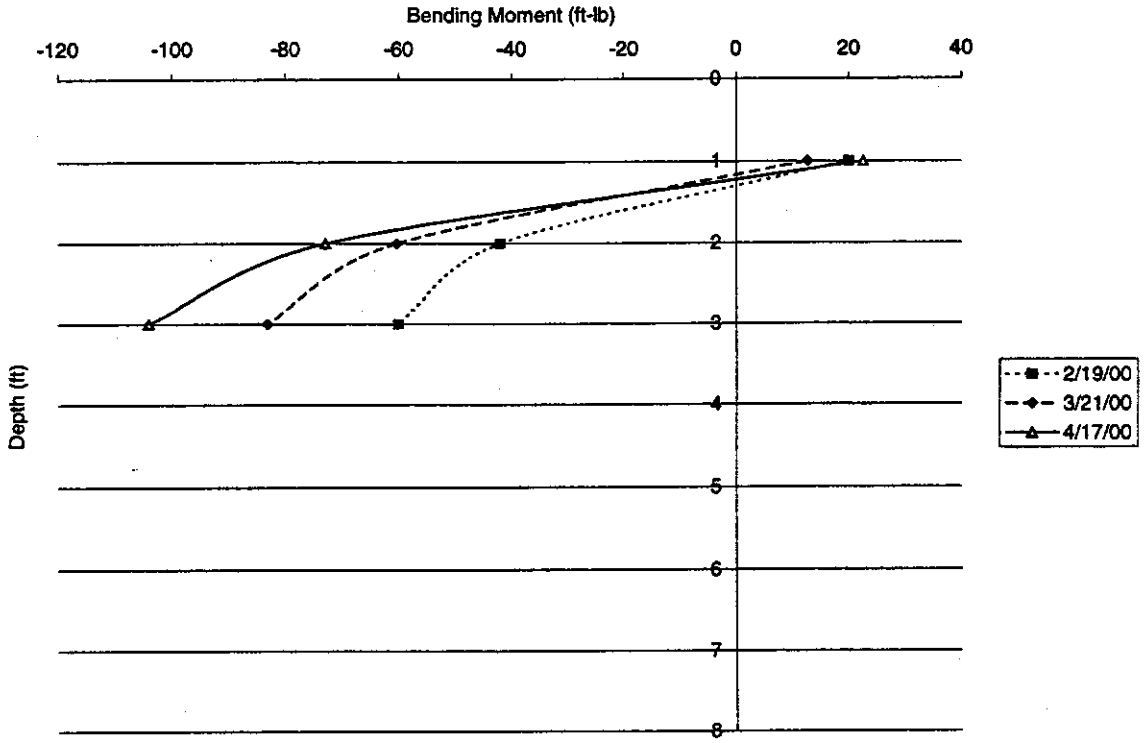


Pin G Bending Moments – Slide S1: Middle Left

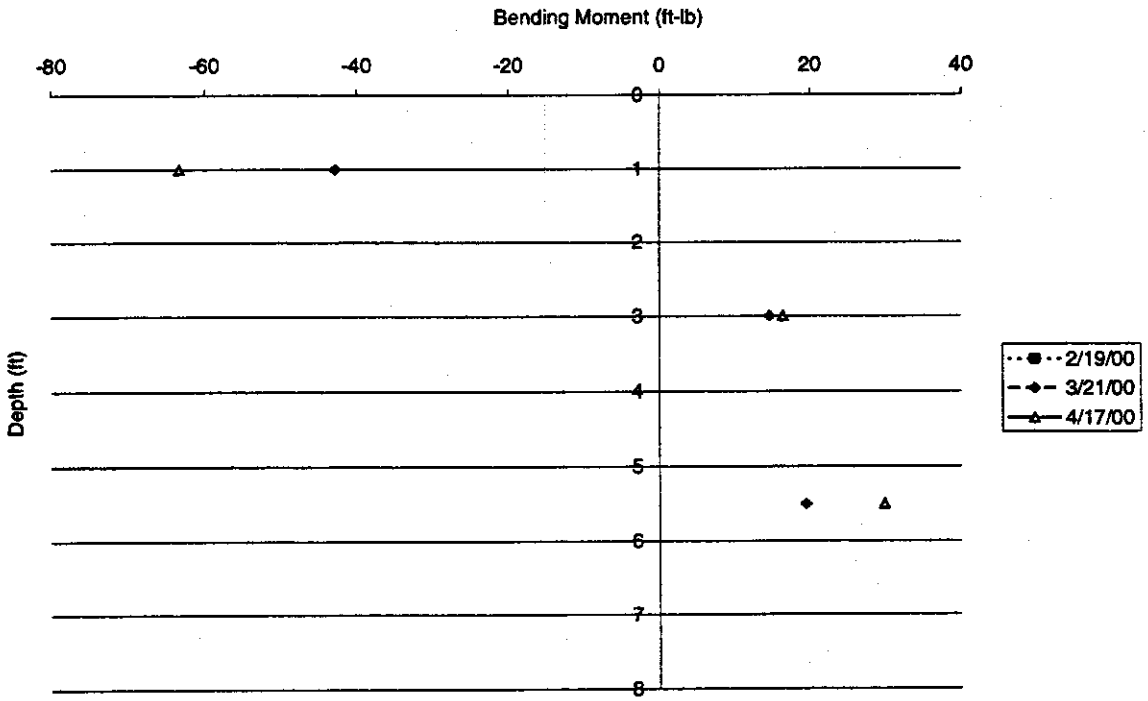


Pin E Bending Moments – Slide S1: Bottom Right

Slide S2 Pin Bending Moments:

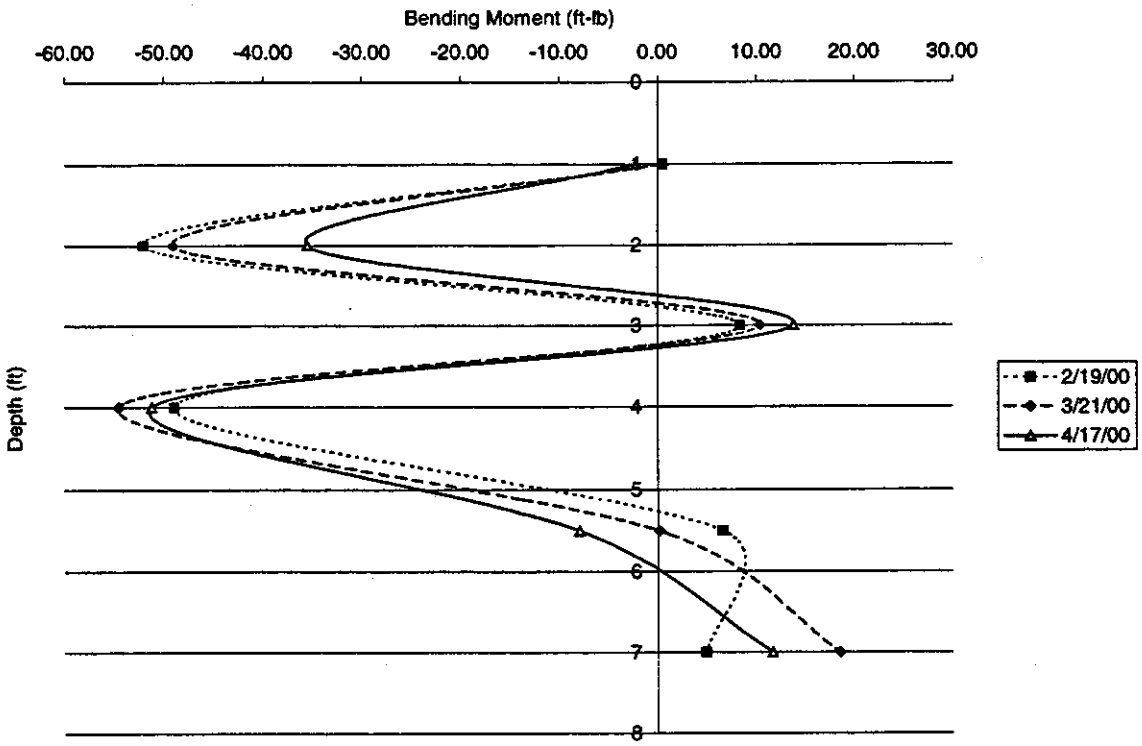


Pin I Bending Moments – Slide S2: Top Center

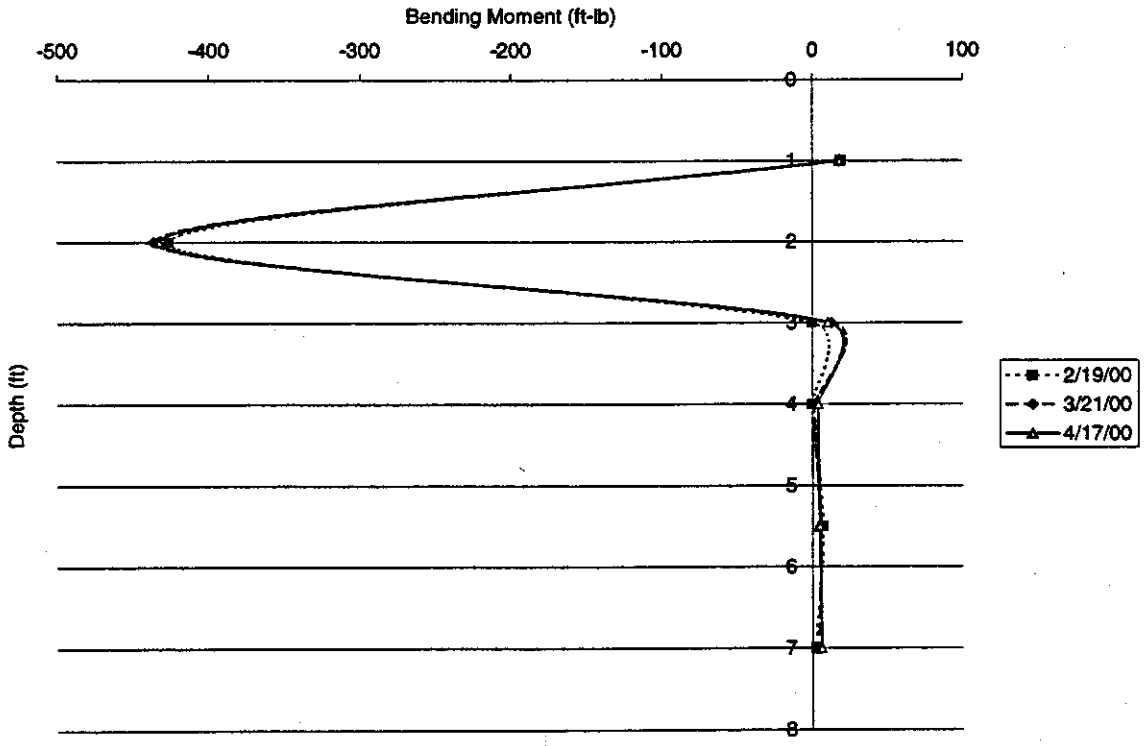


Pin J Bending Moments – Slide S2: Middle Center

Slide S2 Pin Bending Moments (cont'd):

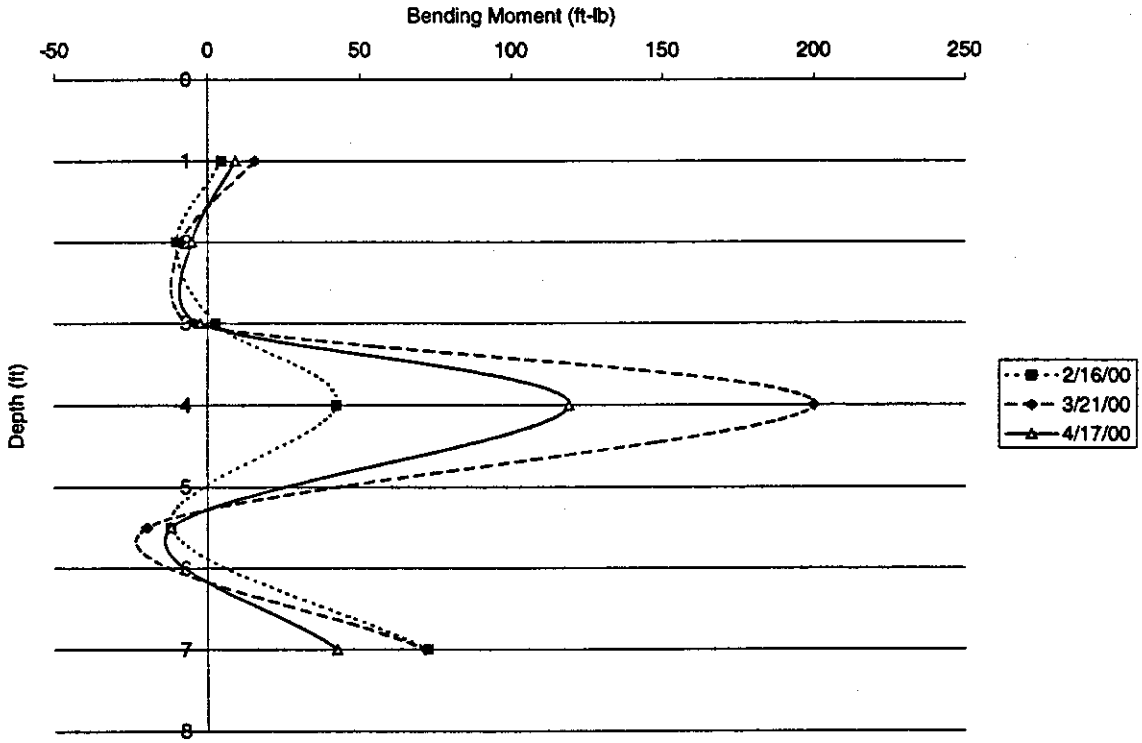


Pin A Bending Moments – Slide S2: Middle Left

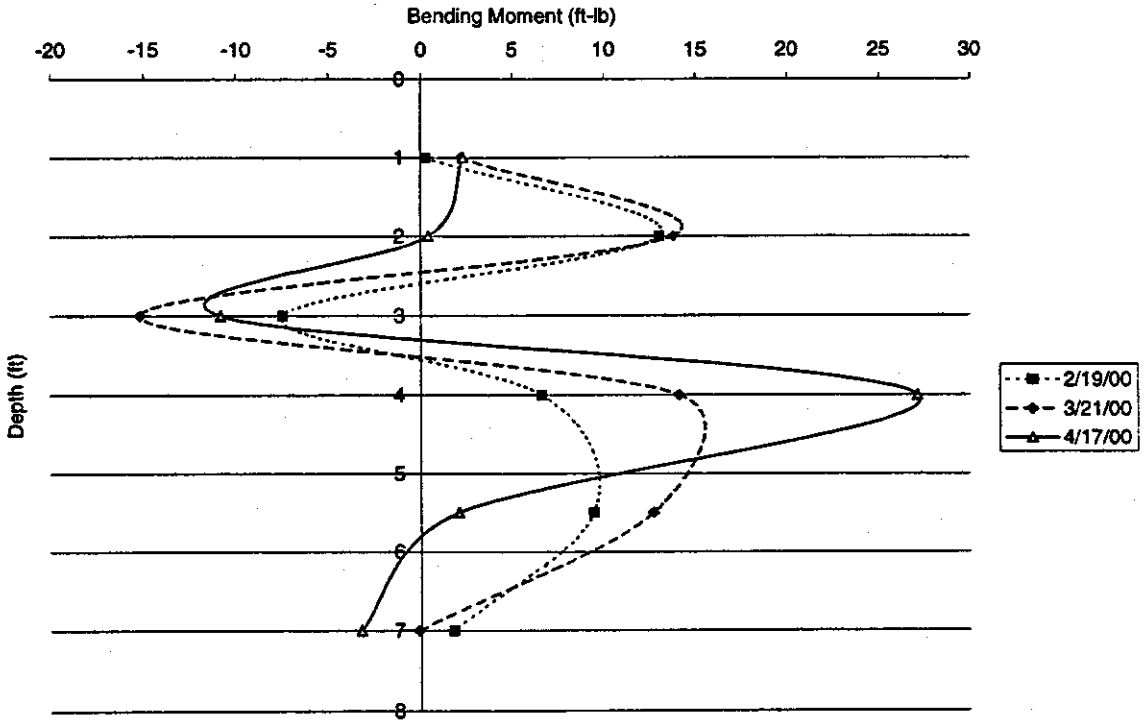


Pin C Bending Moments – Slide S2: Bottom Center

Slide S3 Pin Bending Moments:



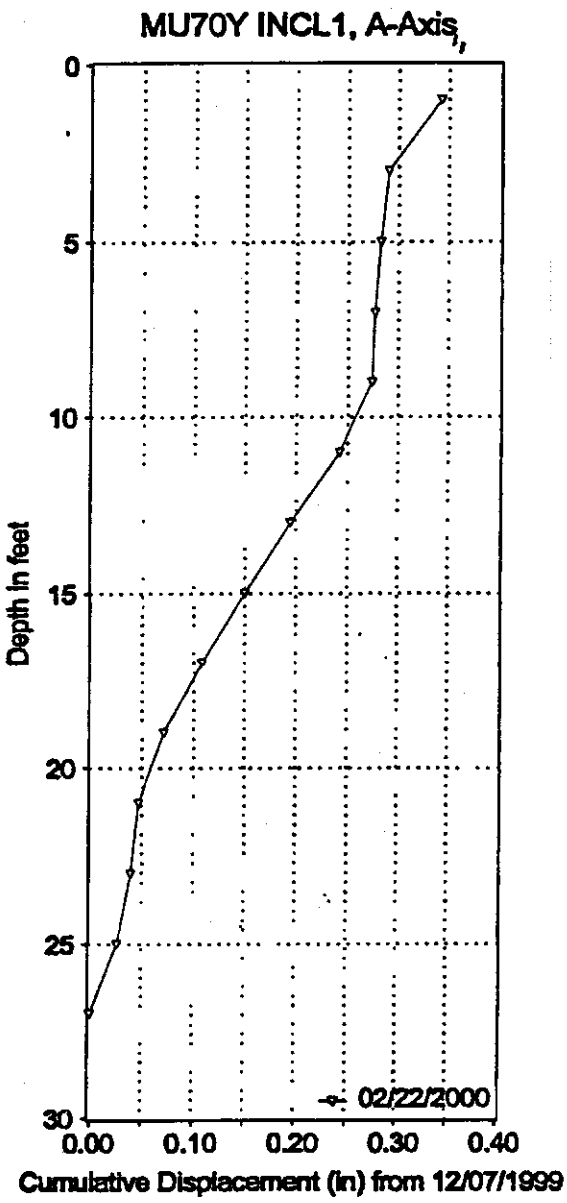
Pin H Bending Moments – Slide S3: Top Center

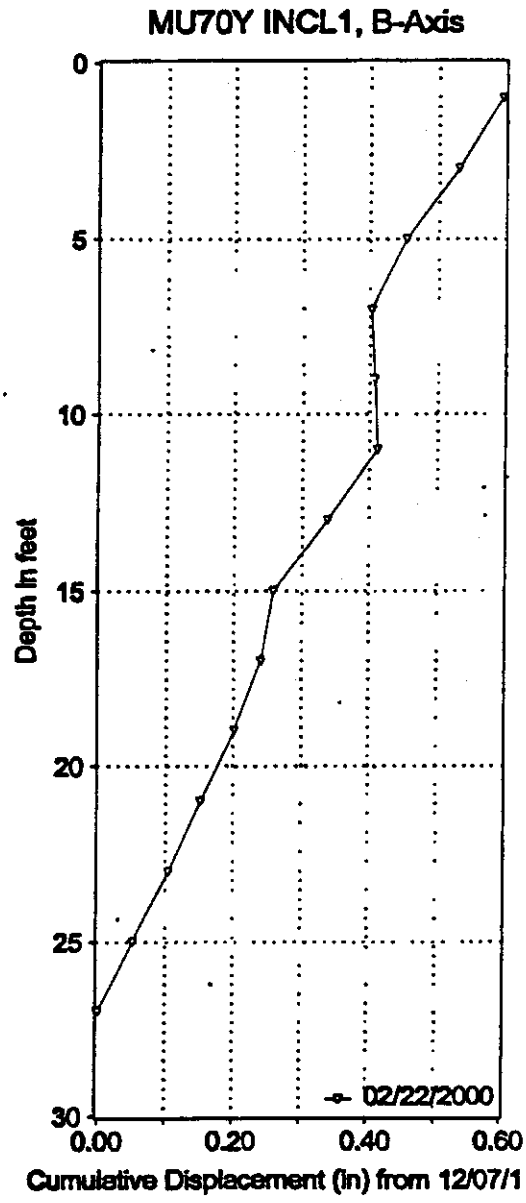


Pin B Bending Moments – Slide S3: Bottom Center

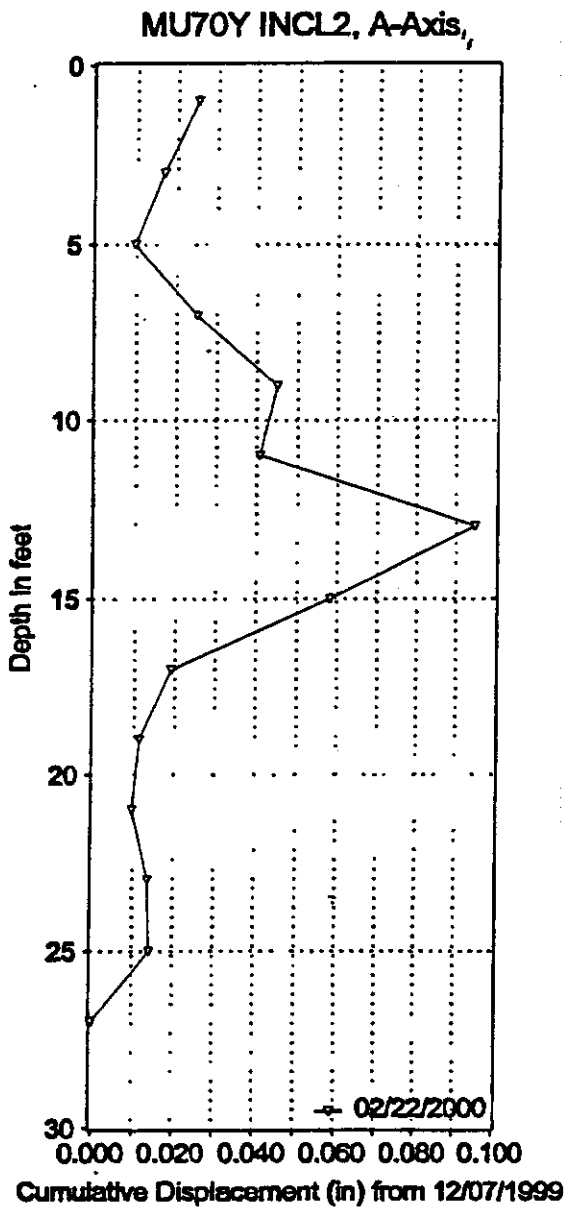
Appendix D –

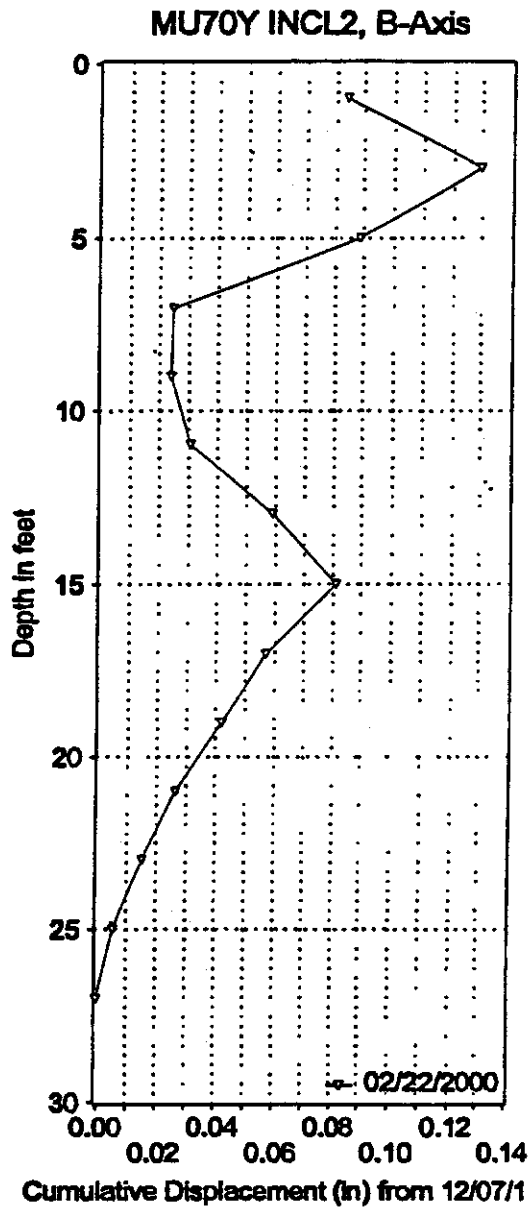
Slope Inclinometer Data



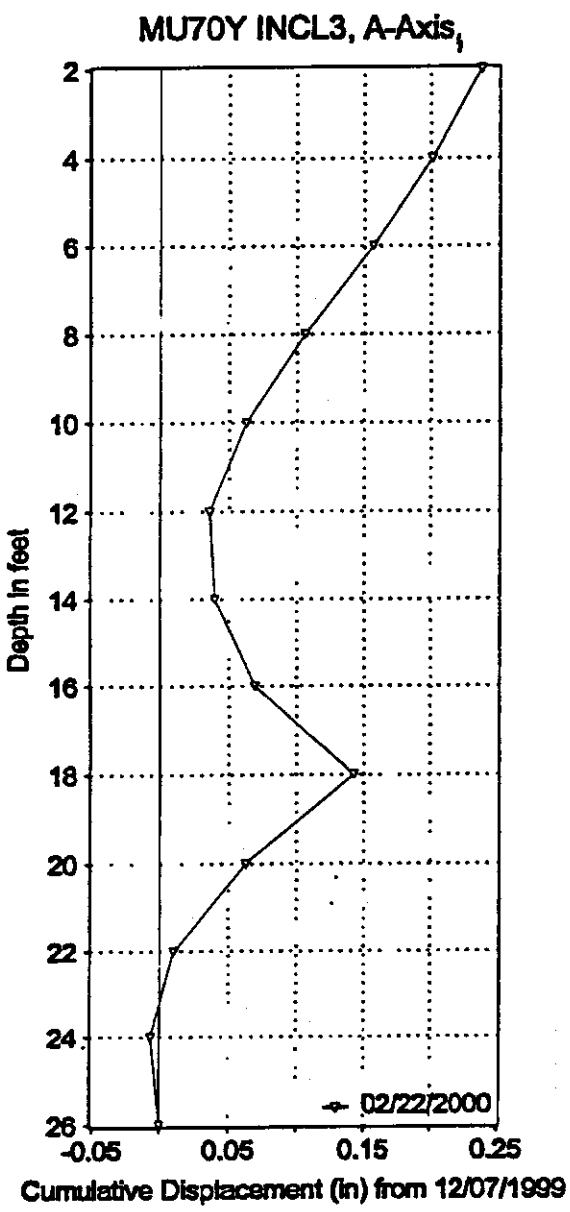


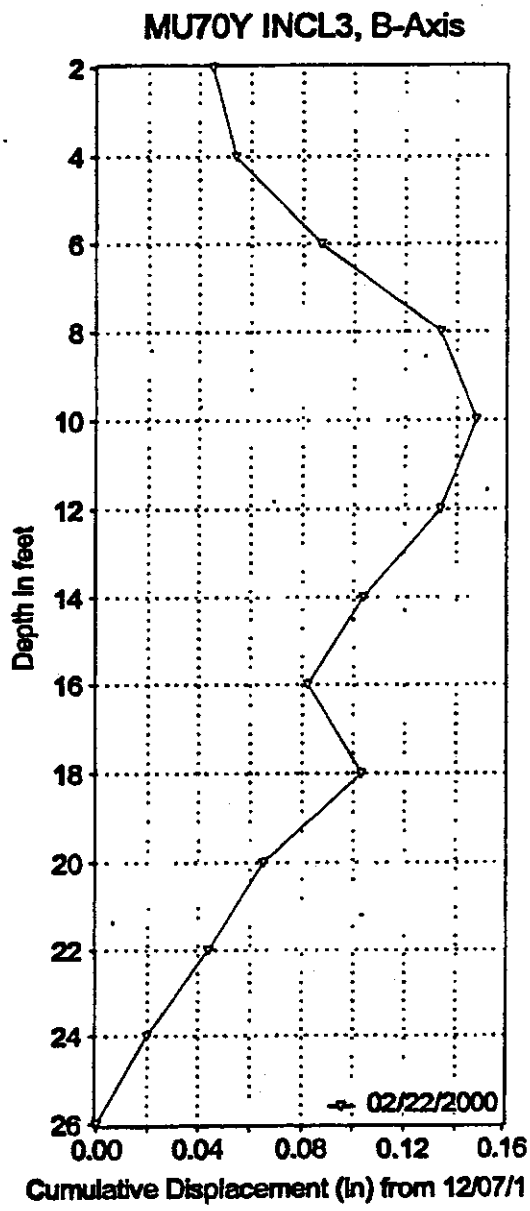
Inclinometer 1 – Slide S1: Middle Left



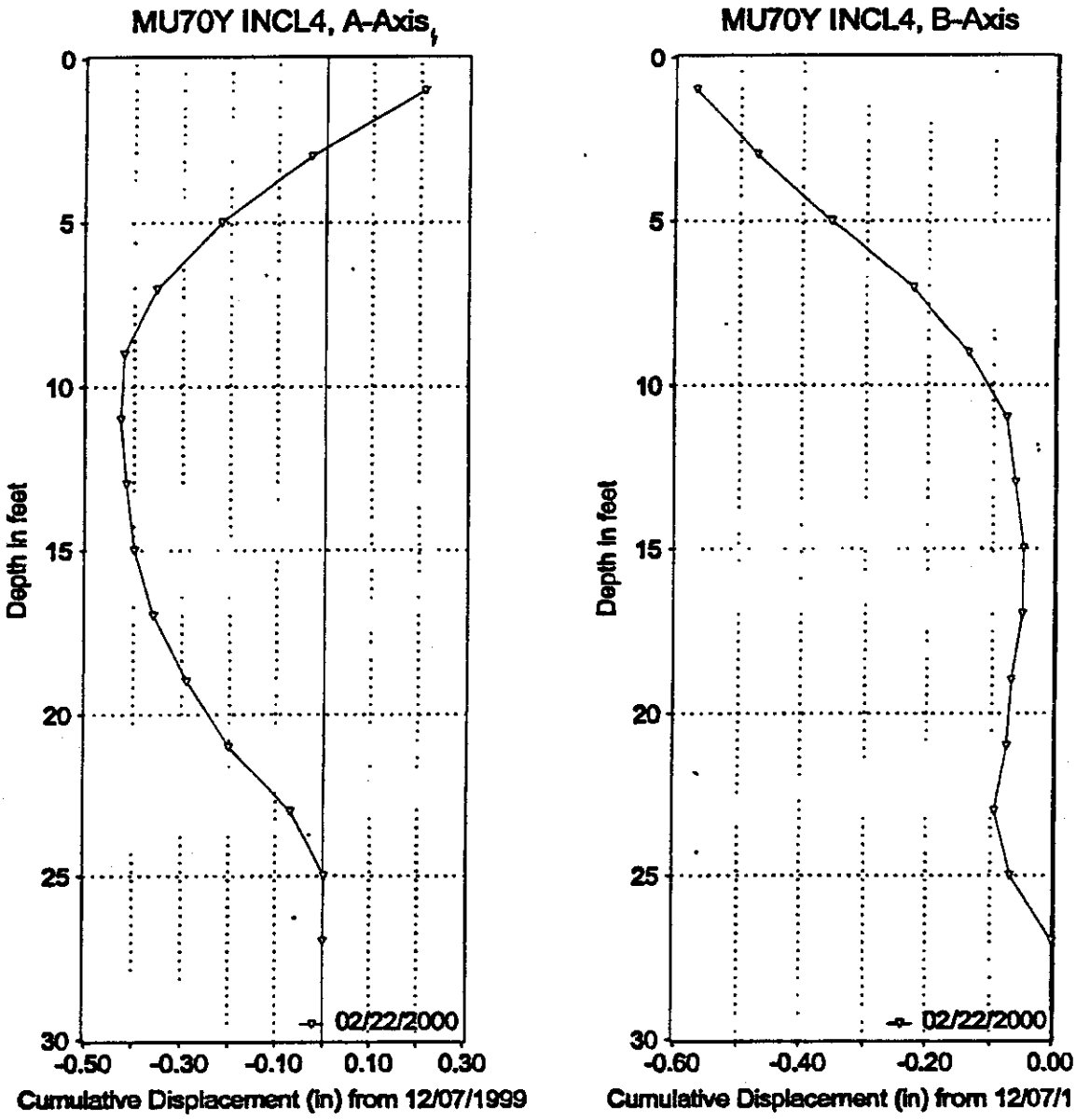


Inclinometer 2 – Slide S1: Top Right

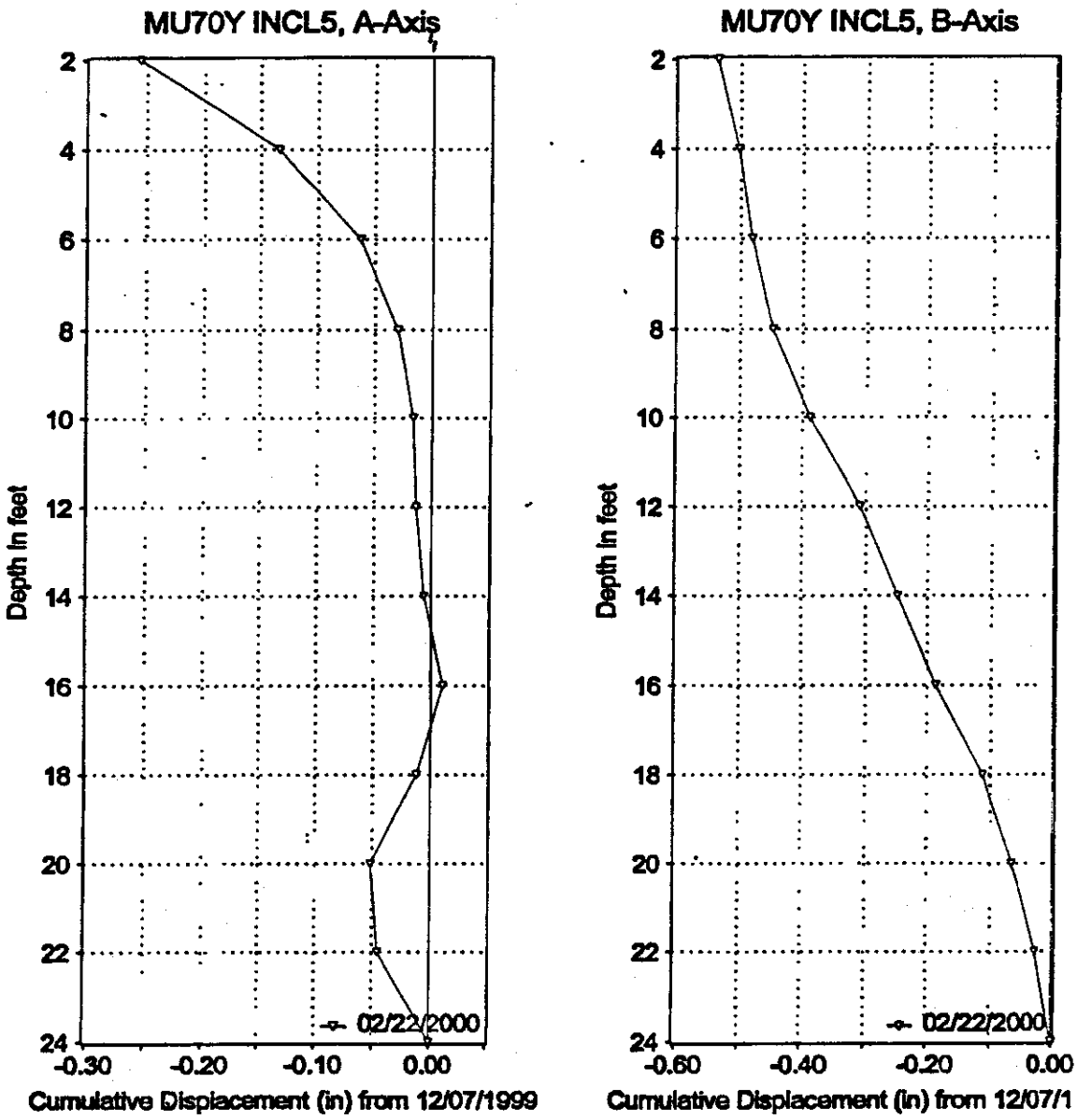




Inclinometer 3 – Slide S2: Middle Center



Inclinometer 4 – Slide S2: Bottom Right



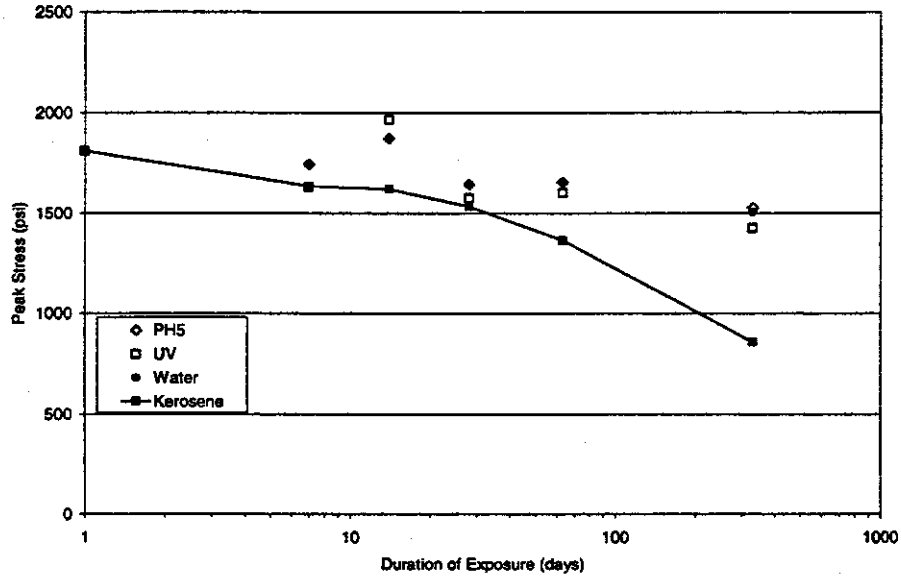
Inclinometer 5 – Slide S3: Middle Center

Appendix E –

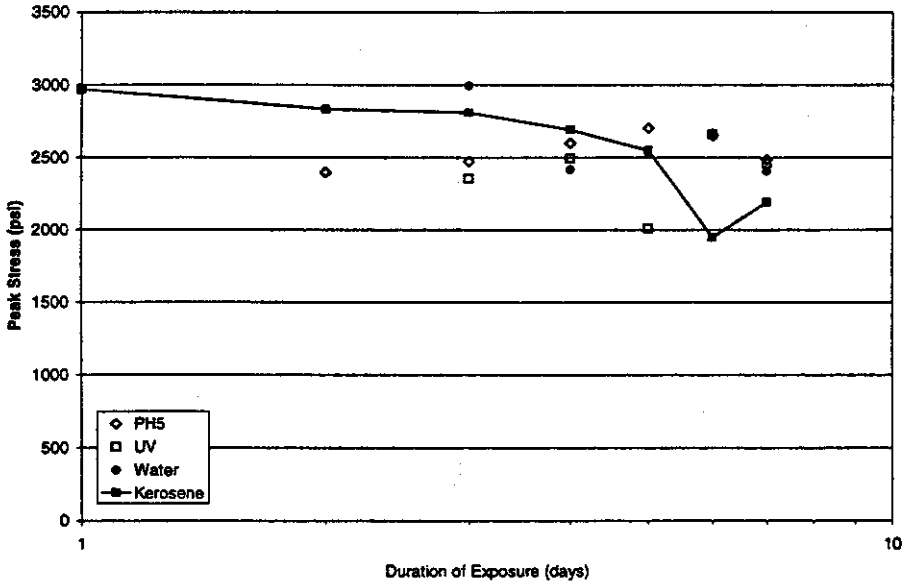
Recycled Plastic Material Properties from Exposure Tests

Ultimate Stress (psi):

	Tension				Compression			
days	PH5	UV	Water	Kerosene	PH5	UV	Water	Kerosene
1	1810	1810	1810	1810	2970	2970	2970	2970
7	1743	1629	1743	1635	2399			2835
14	1871	1965	1871	1621	2475	2354	2995	2811
28	1644	1575	1644	1533	2601	2495	2418	2694
63	1654	1600	1654	1365	2705	2012	2526	2550
330	1527	1425	1506	859	2654	2658	2669	1949
375	N/D	N/D	N/D	N/D	2487	2460	2408	2192



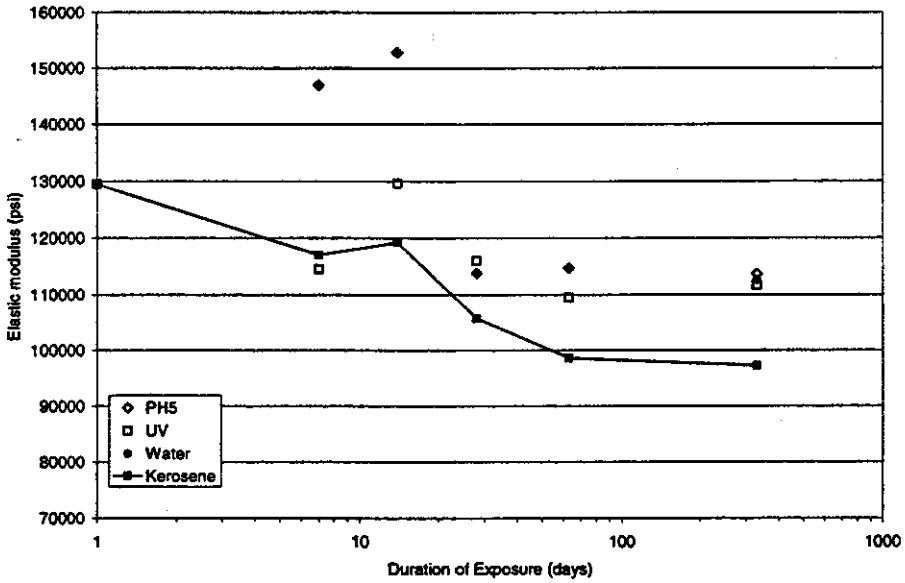
Peak Stress for Tension Tests After Exposure



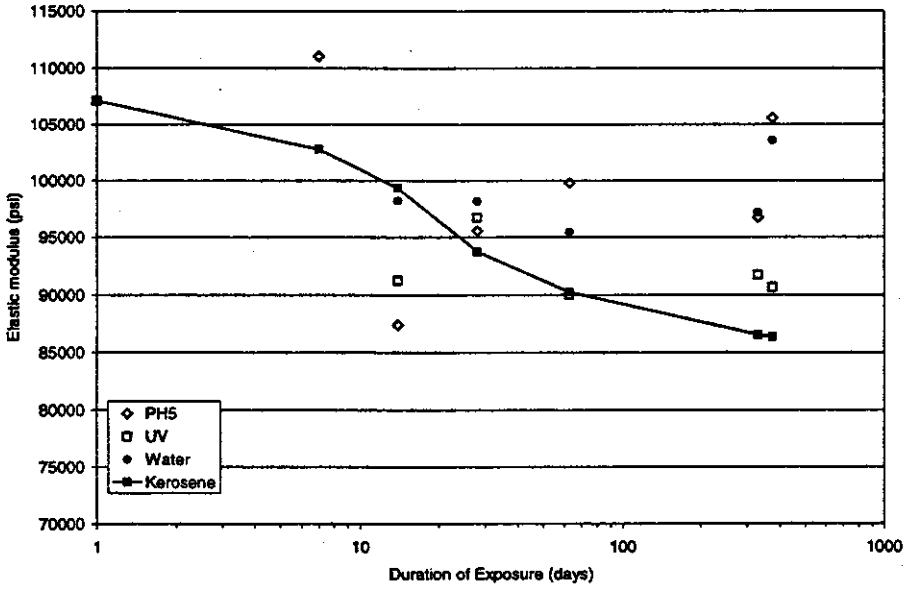
Peak Stress for Compression Tests After Exposure

Elastic Modulus (psi)

	Tension				Compression			
days	PH5	UV	Water	Kerosene	PH5	UV	Water	Kerosene
1	129456	129456	129456	129456	107106	107106	107106	107106
7	146936	114477	146936	117023	111028			102764
14	152807	129596	152807	119239	87384	91246	98203	99325
28	113795	115995	113795	105707	95564	96724	98176	93751
63	114704	109416	114704	98667	99769	89995	95426	90237
330	113675	111596	112576	97279	96720	91716	97183	86543
375	N/D	N/D	N/D	N/D	105549	90654	103556	86336



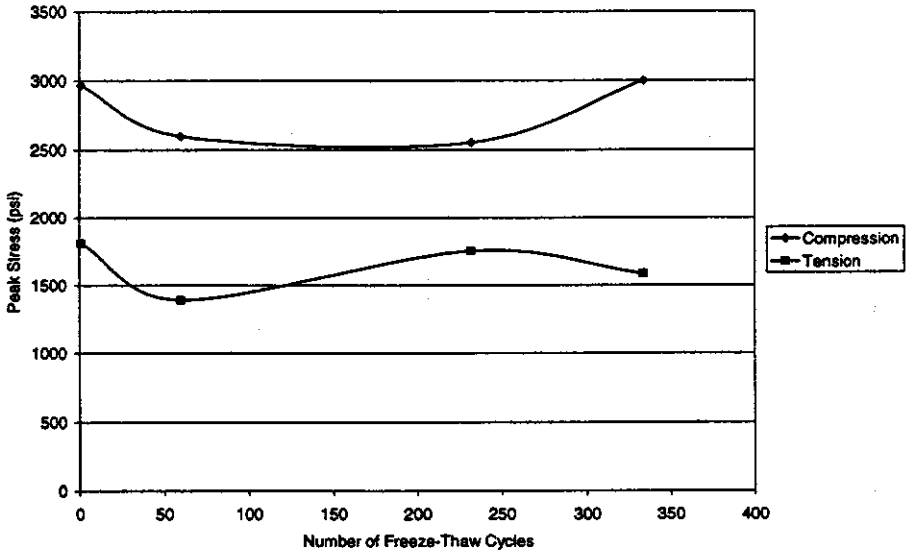
Elastic Modulus for Tension Tests After Exposure



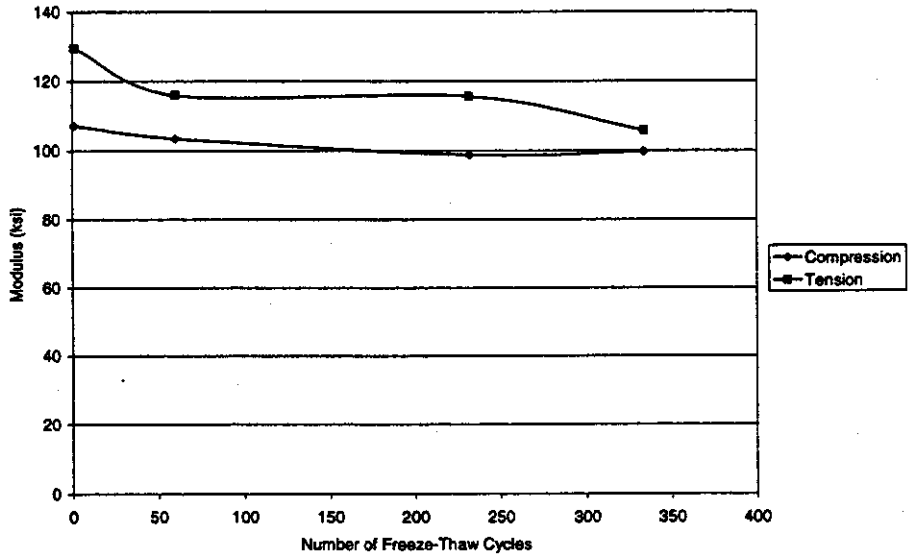
Elastic Modulus for Compression Tests After Exposure

Freeze-Thaw Evaluation

	Tension		Compression	
Cycles	Modulus (psi)	Peak Stress (psi)	Modulus (ksi)	Peak Stress (psi)
1	129.456	1810	107.106	2970
60	115.89	1392	103.398	2599
232	115.573	1754	98.747	2552
334	105.77	1585	99.672	3000



Peak Strength versus Number of Freeze-Thaw Cycles

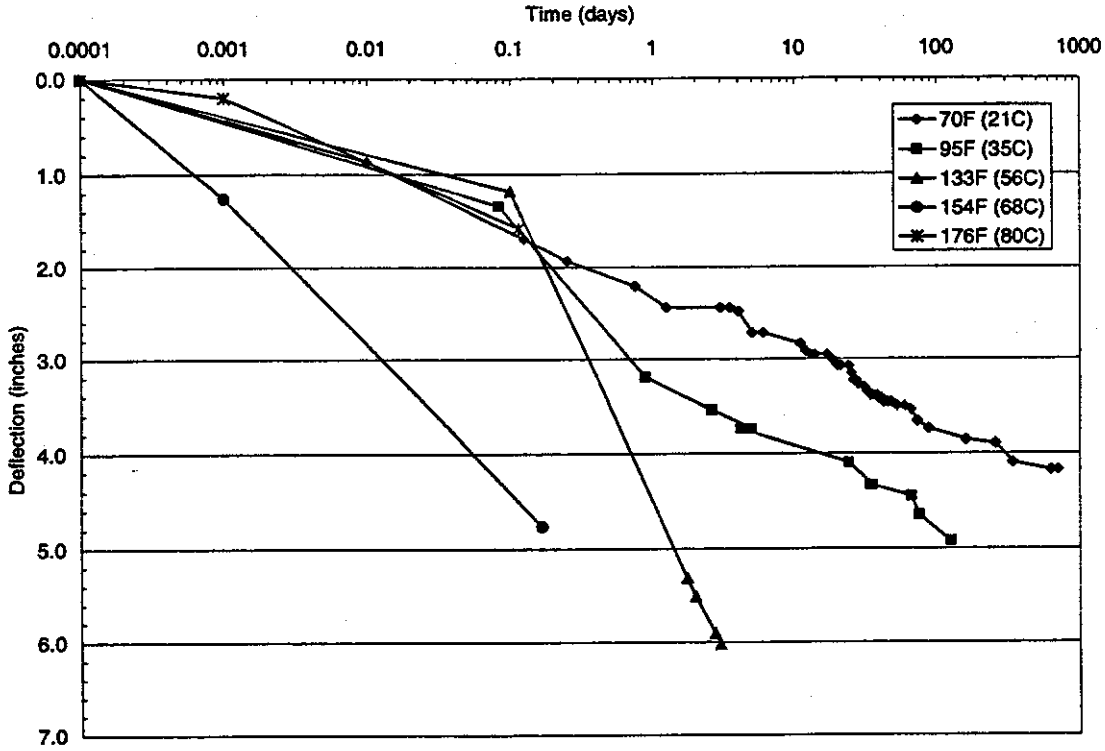


Elastic Modulus versus Number of Freeze-Thaw Cycles

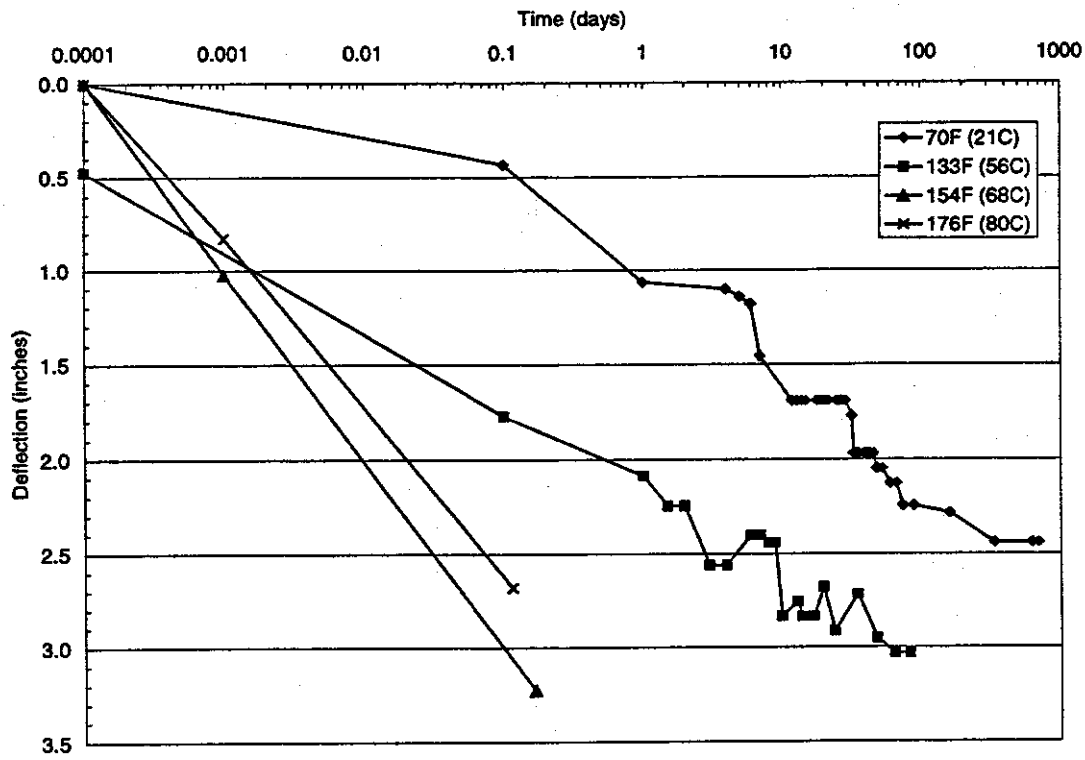
Appendix F –

Bending Creep Test Data and Arrhenius Modeling Calculations

Bending Creep Deflection Data:

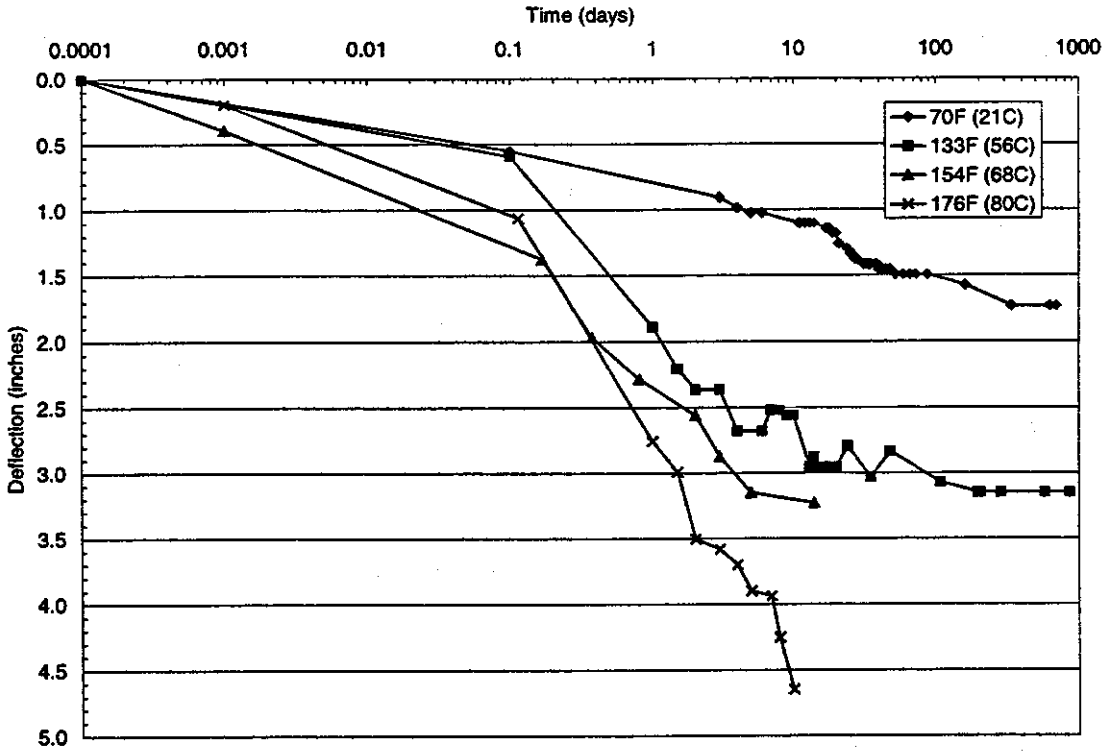


Deflection versus Time for Single 50-lb. Loading

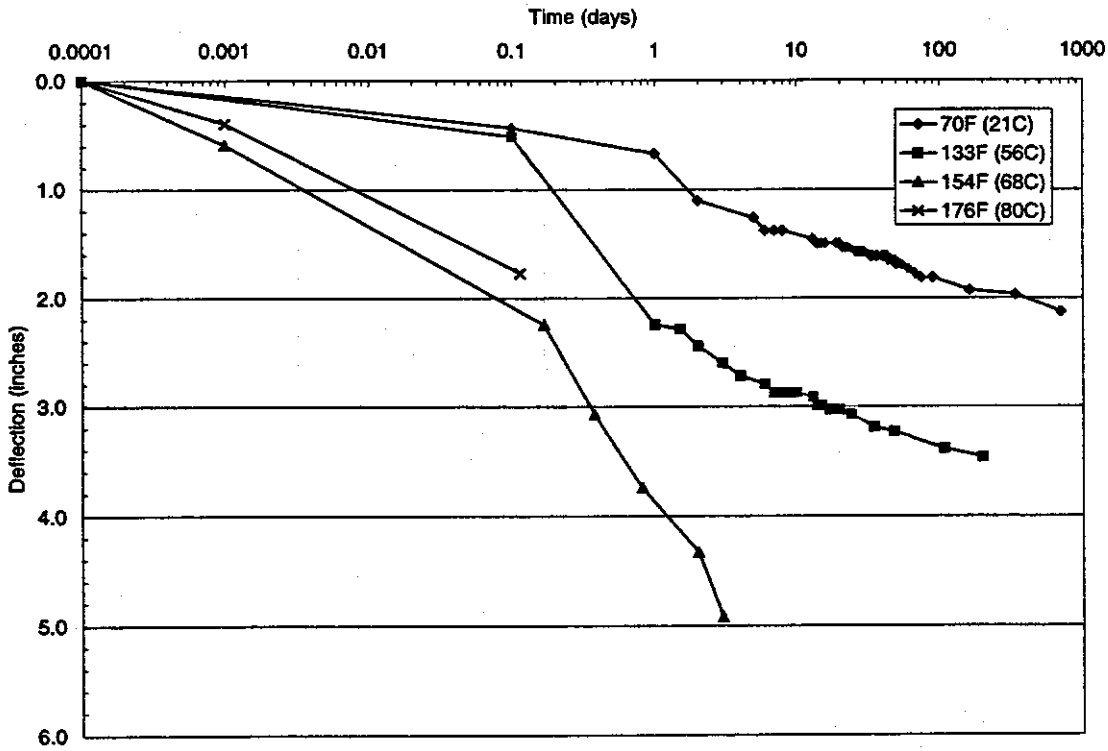


Deflection versus Time for Single 35-lb. Loading

Bending Creep Deflection Data (cont'd):



Deflection versus Time for Single 21-lb. Loading



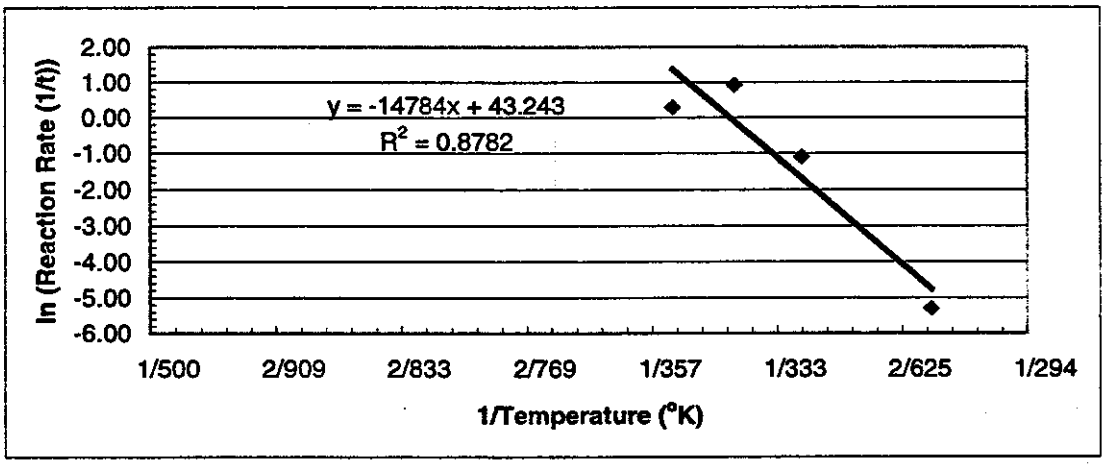
Deflection versus Time for 5 Equally Spaced 10-lb. Loading

Arrhenius Modeling Calculations:

RPP Bending Creep Data Summary
Arrehnius Modeling for Longterm Bending Behavior

Data for single 50 lb weight - Time to reach failure (breaking)

	Time (day)	Temp (C)	Temp (K)	1/time s-1	Ln(1/t)	1/temp c-1	1/temp K-1
not failed	697	21	294	0.001435	-6.546785	0.047619	0.003401
failed	200	35	308	0.005000	-5.298317	0.028571	0.003247
failed	3	56	329	0.333333	-1.098612	0.017857	0.00304
failed	0.4	68	341	2.500000	0.916291	0.014706	0.002933
failed	0.75	80	353	1.333333	0.287682	0.012500	0.002833



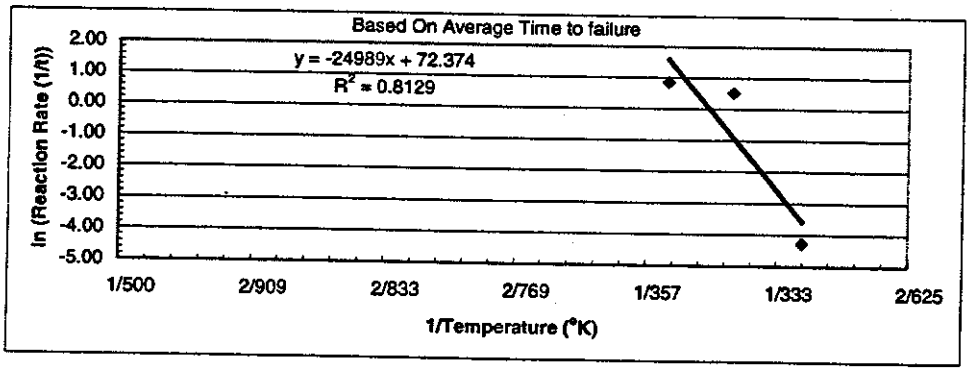
ln (1/t) = -14784(1/T) + 43.243
t = time to reach failure (defined here as breaking)
T = temperature (°K) at which RPP will be in the field (Assumed = 21C=294K)

t = 1145 days (under the single 50 lb cantilever stress)

RPP Bending Creep Data Summary
Arrehnius Modeling for Longterm Bending Behavior

Data for single 35 lb weight - Time to reach failure (breaking)

	Time (day)	Time (day)	Avg. Time (day)	Temp (C)	Temp (K)	1/avg time d^{-1}	Ln(1/avg t) d^{-1}	1/temp C^{-1}	1/temp K^{-1}	Ln(1/Lg t) d^{-1}	Ln(1/Min t) d^{-1}
not failed		697	697	21	294	0.001435	-6.546785	0.047619	0.003401		
				35							
failed	35	108	71.5	56	329	0.013986	-4.269697	0.017857	0.00304	-4.682131	-3.555348
failed	0.375	0.81	0.5925	68	341	1.687764	0.523404	0.014706	0.002933	0.210721	0.980829
failed	0.11	0.75	0.43	80	353	2.325581	0.843970	0.012500	0.002833	0.287682	2.207275



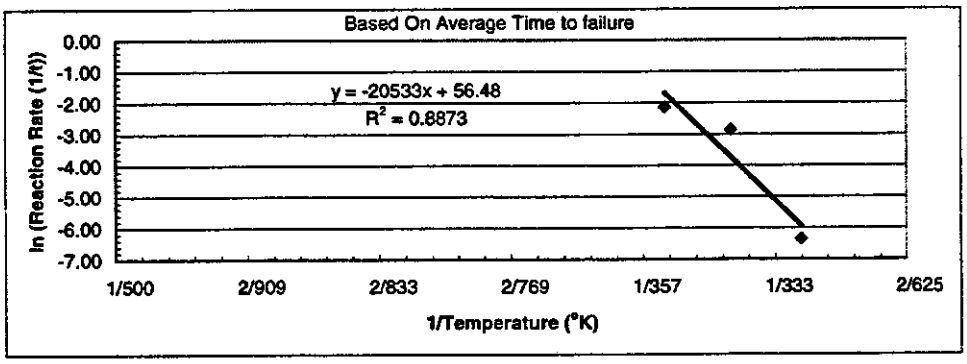
ln (1/t) = -24989(1/T) + 72.374
t = time to reach failure (defined here as breaking)
T = temperature (°K) at which RPP will be in the field (Assumed = 21C=294K)

t = 303336 days (830 years) (under the single 35 lb cantilever stress)

RPP Bending Creep Data Summary
Arrehnius Modeling for Longterm Bending Behavior

Data for single 21 lb weight - Time to reach failure (breaking)

	Time (day)	Time (day)	Avg. Time (day)	Temp (C)	Temp (K)	1/avg time d^{-1}	Ln(1/avg t) d^{-1}	1/temp C^{-1}	1/temp K^{-1}	Ln(1/Lg t) d^{-1}	Ln(1/Min t) d^{-1}
not failed		697	697	21	294	0.001435	-6.546785	0.047619	0.003401		
				35							
not failed	574	574	574	56	329	0.001742	-6.352629	0.017857	0.00304	-6.352629	-6.352629
failed	15	20	17.5	68	341	0.057143	-2.862201	0.014706	0.002933	-2.995732	-2.70805
failed	6	11	8.5	80	353	0.117647	-2.140066	0.012500	0.002833	-2.397895	-1.791759



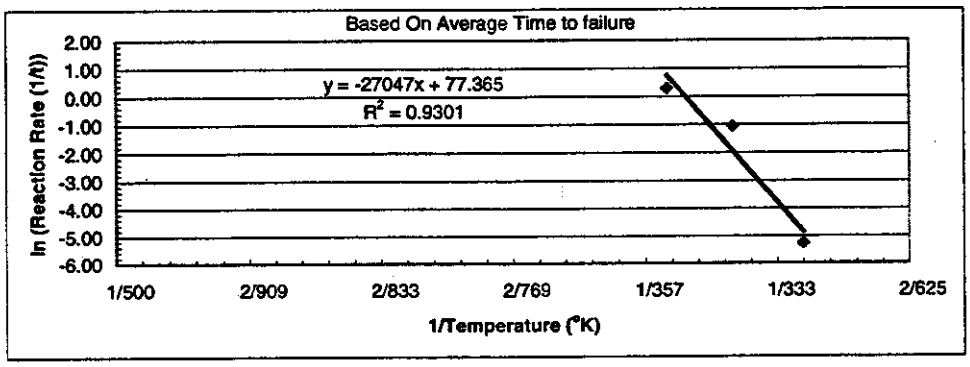
ln (1/t) = -20533(1/T) + 56.48
t = time to reach failure (defined here as breaking)
T = temperature (°K) at which RPP will be in the field (Assumed = 21C=294K)

t = 634210 days (1700 years) (under the single 21 lb cantilever stress)

RPP Bending Creep Data Summary
Arrehnius Modeling for Longterm Bending Behavior

Data for 10-Lb weights at 5 locations - Time to reach failure (breaking)

	Min Time (day)	Max Time (day)	Avg. Time (day)	Temp (C)	Temp (K)	1/avg time d^{-1}	Ln(1/avg t) d^{-1}	1/temp C^{-1}	1/temp K^{-1}	Ln(1/Lg t) d^{-1}	Ln(1/Min t) d^{-1}
not failed		697	697	21	294	0.001435	-6.546785	0.047619	0.003401		
				35							
failed	189	200	194.5	56	329	0.005141	-5.270432	0.017857	0.00304	-5.298317	-5.241747
failed	0.81	5	2.905	68	341	0.344234	-1.066433	0.014706	0.002933	-1.609438	0.210721
failed	0.75	0.75	0.75	80	353	1.333333	0.287682	0.012500	0.002833	0.287682	0.287682



$\ln(1/t) = -27047(1/T) + 77.365$
t = time to reach failure (defined here as breaking)
T = temperature (°K) at which RPP will be in the field (Assumed = 21C=294K)

$t = 2.262 \times 10^6$ days (6000 years) (under five 10-Lb evenly distributed weights on cantilever)

The data (s) below show the ratio of tensile stress in the creep specimens to max tensile stress (1800 psi) versus the estimated time for the specimens to fail in creep bending under the stress that they are loaded.

Ratio of σ_T creep/ σ_{TRPP}	Time to Failure (days)	Time to Failure (years)
70	1144	3.1
50	300000	822
30	600000	1644

See Figure 18 in body of report for graph of data.