AN INVESTIGATION OF EMULSION STABILIZED LIMESTONE SCREENINGS

FINAL REPORT
IOWA HIGHWAY RESEARCH BOARD
PROJECT HR-309

FEBRUARY 1994

Highway Division

Iowa Department of Transportation

Final Report
Iowa Highway Research Board
Project HR-309

An Investigation of Emulsion Stabilized Limestone Screenings

By
Jerry D. Nelson, P.E.
Linn County Engineer
319-398-3445
and
Shane Tymkowicz
Secondary Road Research Coordinator
515-239-1382
Office of Materials
Highway Division
Iowa Department of Transportation
Ames, Iowa 50010
and
Mark Callahan
Waterloo Resident Maintenance Engineer
319-235-1326

February 1994

TABLE OF CONTENTS

DISCLAIMER

INTRODUCTION

A significant amount of fine sized waste limestone screenings is produced during aggregate production. This waste material, which is to fine to be used in either asphalt or portland cement concrete paving, is becoming an ever increasing burden of disposal for aggregate producers. Large stockpiles of the material are at most Iowa quarries. Any road construction process which could successfully use this material would be assured of a continuous supply of inexpensive aggregate.

Linn County was interested in developing such a construction process. An Iowa State University laboratory study (see Appendix B, page 42, reference 1) sponsored by Linn County showed that waste limestone screenings could be used as the sole aggregate in an emulsified asphalt mix. Such a mix could be used to replace selected granular surfaced roads and/or provide the base for stage construction of a future asphalt or portland cement concrete pavement.

OBJECTIVE

The objective of this research project was to construct and evaluate an experimental roadway base using a waste limestone screenings/emulsion mix. Specific topics to be investigated included:

1. The development of an efficient roadway construction technique using the waste limestone screenings/emulsion mix.

2. The mix strength, stability and durability properties obtainable in the field.

3. The optimum residual asphalt content and base thickness required to adequately support local traffic.

4. The validity of the anionic/cationic relationship existing between waste limestone aggregate and an asphalt emulsion.

PROJECT LOCATION AND DESCRIPTION

The roadway selected for this research was a 1.27 mile (2.04 km) section of East Main Street beginning at its intersection with Council Street in the town of Robins and running southeast to its intersection with Linn County road W56 (C Avenue NE). A map of this location is shown in Figure 1, page 18.

The field test section layout included sections having compacted thicknesses of 4 and 6 in. (100 and 150 mm) and residual asphalt contents of 2.5%, 3.5% and 4.5% of the dry weight of the waste limestone aggregate. A control section of untreated limestone screenings was also added for comparative purposes.

PRECONSTRUCTION WORK

Work on the existing roadway was performed prior to placing the experimental base. Linn County awarded a contract to Gee Grading and Excavating, Inc. to replace culverts and shape and compact the subgrade. This work was completed early in July 1988.

CONSTRUCTION

Linn County awarded the contract for construction of the experimental base to Vulcan Industries. A copy of the contract is given in Appendix A. The contractor began base production and construction August 1, 1988. The final surface seal coat was placed August 13, 1988.

Base Materials

Base paving materials included waste limestone screenings from Vulcan's quarry in Robins and a CSS-1 emulsion produced by Koch Materials in Dubuque. An average particle size distribution of the limestone screenings is shown in Figure 2, page 19. Included on the graph are dashed boundaries indicating the limits of a well graded soil/aggregate mix (1). The emulsion contained 62% residual asphalt and had a zeta potential ranging from +27.6 millivolts to +34.6 millivolts.

Mix Production

Vulcan Industries produced the mix used on the project. The same limestone screenings were fed from two bins which were metered to feed aggregate to a continuous drum mixer. Emulsion was sprayed into the drum at the rate needed to obtain the desired residual asphalt content in the mix (2.5%, 3.5%, or 4.5%). The mix production rate was low, usually running around 100 tons (907 Mg) per hour.

Several problems were encountered during mix production. First, a considerable amount of balling of the emulsion occurred throughout the time the mix was being produced. Most of these balls were less than 1/2 in. (13 mm) diameter. However, the balling resulted in a slightly uneven distribution of asphalt in the mix. Also, aggregate being fed to the mixer would occasionally clog the bins. Because of this, a worker was required to continuously monitor the bins to ensure aggregate was flowing.

Several attempts were made to reduce the balling problem. It was felt the problem was moisture related, so the contractor began to modify the mix moisture content. First, a drier limestone screenings aggregate, coming immediately from the quarry's rock crushing operation, was fed into the bins. The drier aggregate, however, did not reduce the amount of asphalt balling. Next, a hose was used to apply additional moisture to the surface of the aggregate on the conveyor prior to entering the mixer. This also failed since moisture tests indicated less than desirable mix moisture content, and visual examination indicated layering of moist to relatively dry aggregate on the conveyor. The asphalt balling problem continued throughout the research project.

The asphalt balling was not considered to be a major problem. A majority of the asphalt was well mixed with the aggregate. Also, the method of compaction used on the base, a padsfoot roller and motor grader operation, provided added mixing of the asphalt. The balling simply prevented a more desirable distribution of asphalt throughout the mix, a condition which may have been improved through use of a pugmill, rather than a drum mixer.

Base Construction

Construction data on each test section are presented in Table I.

Table I
Test Section Data

Section No.	Stationing From	Stationing To	Base Depth, Inches	Base Depth, mm	Residual Asphalt Percent
1	108+37	117+83	6	150	4½
2	117+83	127+30	6	150	3½
3	127+30	136+76	6	150	2½
4	136+76	142+22	6	150	0
5	142+22	6+77*	4	100	2½
6	6+77	16+23	4	100	3½
7	16+23	25+70	4	100	4½

*Station Equation 150+02.90 Back = 1+10.00 Ahead

Six-Inch (150 mm) Base

Base construction began on the eastbound lane of Section 1. Mix was hauled to the site in trucks and dumped into a Cedar Rapids BSF-420 asphalt paver. The waste limestone screenings/emulsion mix would not pass through the paver and spread uniformly across the roadway. Construction was discontinued after laying only 470 ft (143 m).

A decision was made to abandon use of the paver. A Jersey type spreader pushed by a Caterpillar D8 was used throughout the remainder of the project to lay the base mix.

The loosely laid mix required from 1 to 3 hours for aeration, depending on the amount of emulsion in the mix. Initially, a steel drum roller was used to compact the base. However, two problems were quickly encountered with its use. First, the mix shoved badly under the roller, resulting in small, tight, shear cracks being created on the surface. Also, the roller created a tight crust which inhibited curing of the mix and reduced compaction in the lower portion of the base.

In order to increase the aeration rate, eliminate shear cracking, and improve depth of compaction, a padsfoot vibratory drum was used to compact and aerate the laid base. The aeration increased the curing rate of the mix and allowed full depth compaction to be completed much sooner than with the smooth drum roller. A motor grader was used to level the surface once the padsfoot had made several passes over the base. Final compaction was done with a pneumatic tired roller providing a smooth, tight surface.

Some shoving of the mix continued to occur under the padsfoot, but to a much lesser extent than had occurred when using the steel drum roller. There were two principle reasons for the shoving. First, the aggregate was lean on limestone screenings larger than a #4 sieve (4.75 mm), resulting in a lack of aggregate interlock being developed. Second, there was no lateral support to confine the mix when compacting the outside edges of the base.

At the start of the second day of construction, a new laydown and compaction procedure was used in order to reduce the amount of shoving encountered the first day. The spreader box was adjusted such that extra material was placed on the outside edge of the eastbound lane. This extra material was spread onto the shoulder and compacted first, thus acting to confine the remaining material being compacted. Although not eliminated, lateral shoving was reduced significantly using this procedure.

The second day, the contractor experienced problems with the mix being too dry. In an attempt to alleviate the asphalt balling problem discussed previously, a drier limestone screening aggregate was used in the eastbound lane of Section 3. The combined effect of using a drier aggregate and reducing the amount of emulsion (2.5% residual asphalt) resulted in a mix too dry to compact. A distributor truck was used to add water to the mix in the field. The mix was then recompacted using the padsfoot roller.

Once the eastbound lane of Section 3 was finished, the contractor returned to begin paving the westbound lane of Section 1. The dry aggregate worked well with the higher emulsion content used on Section 1 (4.5%). However, the asphalt balling problem remained. Use of the dry aggregate was discontinued once it was determined the balling was not being reduced.

After laying the westbound lane of Section 1, the contractor added a second lift on the eastbound lane of Section 1. This was required because the asphalt paver used initially did not place a full 6 in. (150 mm) of base. Once the second lift was completed, the contractor continued paving the westbound lane of Sections 2 and 3, which were completed without further incident.

Four-Inch (100 mm) Base

The paving sequence on the 4 in. (100 mm) base was altered from that finally used on the 6 in. (150 mm) base. Section 7 (4 1/2% A.C.) was paved first, both lanes being paved before beginning Section 6. This pattern of completing one section before beginning another was continued for the remainder of construction.

Placement of each section proceeded without incident. Asphalt balling was the only persistent problem. In a final attempt to resolve the problem, a water hose was placed inside the drum mixer to add moisture to the aggregate during the mixing process. It was hoped this would keep the fines from balling with the asphalt. However, this was not the case. It was determined the balling was not a serious problem and that paving should continue.

The 4 in. (100 mm) sections were compacted more easily than the 6 in. (150 mm) sections. The padsfoot roller penetrated full depth of the lift, confining the material within the roller's pads, resulting in less lateral shoving compared to the 6 in. (150 mm) sections.

Rain fell one night while the 4 in. (100 mm) base sections were being constructed. Fortunately, the contractor had compacted all the mix placed that day and had rolled down all edges. Had this not been done, water would have soaked into the mix and the aeration/curing process would have likely been delayed several days.

The control section, consisting of untreated limestone screenings, was placed using the same technique used in placing the other sections. Finally, a double seal coat was placed over the entire project to keep down limestone fines and to provide a water tight riding surface.

TESTING

Testing on the project was conducted jointly by Iowa State University and the Iowa DOT. Iowa State University personnel ran moisture and density tests during construction and prepared field mixed samples of the waste limestone screenings/emulsion mix for laboratory testing. A report prepared for Linn County by Iowa State University describing the test results is given in Appendix B.

Iowa DOT testing included Road Rater structural rating, 25-Foot California Profilometer, and BPR Roughometer testing. Results of these tests are given in Appendix C.

Testing was continued for a period of five years. Annual testing performed by the Iowa DOT include the Road Rater and crack surveys.

Road Rater Summary

Road Rater testing has been conducted annually on the entire project (Table 2 and Table 3). The Road Rater is a dynamic deflection measuring device used to determine the structural adequacy of pavements. The differences in pavement structural ratings from year to year may be explained by the fact that annual testing is performed on the outside wheel track during the months of April and May when the roadway exhibits the poorest structural support. The structural rating can vary from one year to the next depending upon the moisture content of the soil at the time of testing.

Table 2
Average Structural Ratings

Section	Station	1989	1990	1991	1992	1993
1	108+37 to 117+83	2.44	2.47	2.34	1.98	1.89
2	117+83 to 127+30	2.53	2.66	2.44	2.21	1.89
3	127+30 to 136+76	2.02	2.20	2.13	1.79	1.60
4	136+76 to 142+22	1.80	2.06	1.53	2.07	1.67
5	142+22 to 6+77	1.71	1.83	*	1.75	1.68
6	6+77 to 16+23	2.16	2.47	2.40	2.00	1.85
7	16+23 to 25+70	1.97	2.14	1.98	1.70	1.65

Note: Station Equation 150+02.90 = 1+10.00
*Error in data collection

Table 3
Average Soil K Values
(pci)

Section	Station	1989	1990	1991	1992	1993
1	108+37 to 117+83	195	210	189	192	205
2	117+83 to 127+30	200	214	214	206	223
3	127+30 to 136+76	171	186	177	176	180
4	136+76 to 142+22	164	204	103	155	181
5	142+22 TO 6+77	129	153	155	141	149
6	6+77 to 16+23	193	218	214	202	225
7	16+23 to 25+70	148	164	159	156	181

Note: Station Equation 150+02.90 = 1+10.00

The annual average structural ratings for the project are given graphically in Figure 3, page 20. For any given asphalt content, the annual structural ratings of the 6 in. (150 mm) base was higher than the 4 in. (100 mm) base except for the 2.5% asphalt sections in 1993. The 3.5% asphalt test sections had the highest structural rating for the 6 in. (150 mm) and the 4 in. (100 mm) depths. The 6 in. (150 mm) base with no asphalt cement showed a large variation in its structural rating. The general trend for all the test sections that were treated with the asphalt emulsion was an increase in the structural rating from 1989 to 1990. From 1990 to 1993 these sections experienced a steady decrease in their structural rating. Note that no data was available for Section 4 in 1991.

The annual average soil K values are shown in Figure 4, page 21. The same general trends that were observed in the annual average structural ratings are also evident in the graph of the annual average soil K values. The 6 in. (150 mm) bases had higher soil K values than the 4 in. (100 mm) bases. The 3.5% asphalt content test section had higher soil K values than the 2.5% and 4.5% asphalt content test sections for each depth. The 6 in. (150 mm) base with no asphalt had a wide variation in soil K value.

Table 4 lists areas that have received A.C. strengthening mats. These A.C. strengthening areas seemed to have only minor effects on the Road Rater tests. The general trends of the Road Rater data appeared to be unaffected by the A.C. strengthening mats.

Crack Survey

Crack surveys were conducted annually since the completion of the project in 1988. However, during the duration of the project strengthening asphalt mats and new chip seal layers were required in some test sections. The asphalt strengthening areas are listed in Table 4. These maintenance operations have prevented a detailed comparison of cracking between the sections. However, several trends were noted during the crack surveys.

Table 4
A.C. Strengthening Areas
Robins - East Main Street

Date	Location	Test Section	Length	Depth	Side	Width
10-17-91	*Sta. 147+00 to 1+72	V	365'	2"	F.W.	24'
6-5-92	Sta. 4+00 to 5+50	V	150'	2"	F.W.	24'
6-5-92	Sta. 144+50 to 147+35	V	285'	2"	F.W.	24'
6-5-92	Sta. 140+78 to 142+28	IV & V	150'	2"	F.W.	24'
7-6-93	Sta. 134+28 to 144+78	III, IV, V	1050'	1½"	LT.	12'
7-6-93	Sta. 136+03 to 144+78	III, IV, V	875'	1½"	LT.	12'

*Equation Sta. 150+02.90 Back = Sta. 1+10.00 Ahead.

1 inch = 25 mm
1 ft. = 0.305 m

The sections having a 0% or 2.5% residual asphalt content (Sections 3, 4 and 5) had the most severe cracking. These sections also required the asphalt strengthening mats. The 6 in. (150 mm) bases having a 3.5% or 4.5% residual asphalt content had fewer cracks than the 4 in. (100 mm) bases of the same residual asphalt content. The section with the fewest cracks was Section 1 (6 in. (150 mm) base, 4.5% asphalt).

PROJECT COST

The project cost $141,355.13. The contract can be found in Appendix A. The final construction costs can be seen in

Appendix D. A large portion of the contract price was the $70,297.08 for the bituminous treated aggregate (limestone screenings). The contract price for the bituminous treated aggregate was $14.84/ton. If the price for bituminous treated aggregate could be reduced, the economic benefit of using limestone screenings would greatly increase. This may be possible as these screenings continue to stockpile.

CONSTRUCTION RECOMMENDATIONS

After the project was completed, a meeting was held to discuss possible improvements to the procedures used. Some suggestions made included the following:

1. The mixing process will need to be improved on future projects. Although adequate for this project, the drum mixer used did not completely mix the emulsion and limestone screenings. The asphalt balling problem persisted throughout the project. It is recommended a traveling plant or road mixer be used on future projects. If a central plant is required, a pugmill type would be more suitable.

2. A padsfoot roller and motor grader worked well to compact and shape the roadway. This procedure should be continued due to fineness of the aggregate and lack of interlocking granular particles. Steel drum and pneumatic tired rollers should only be used in the final stages to obtain a tight base surface.

3. Base lifts should be limited to a maximum compacted thickness of 4 in. (100 mm). This depth worked well with the compaction technique used on this project. Excessive shoving of the mix is likely to occur when compacting lifts of greater thickness.

4. Precautions should be taken to prevent rain water from soaking into the material after it is placed. All material placed in a day should be compacted and rolled to provide a tight surface seal. Also, all edges should be rolled down to allow easy drainage of rainwater.

DISCUSSION

The use of limestone screenings mixed with an asphalt emulsion as a base is a viable technique. If the base had at least a 3.5% residual asphalt content and a depth of 4 in. (100 mm) an acceptable base was produced. A 6 in. (150 mm) thickness will produce a base that will yield fewer cracks and a higher structural strength. Figure 5, page 22 shows the optimum residual asphalt content to most likely reside near 3.5% for maximum structural strength. These results may vary as the gradation of the limestone screenings change, especially as the percentage of clay and silt particles increase.

CONCLUSIONS

This research on emulsion stabilized limestone screenings support the following conclusions:

1. A low maintenance roadway can be produced using a seal coat surface on 6 inches (150 mm) of stabilized limestone screenings with 4.5% asphalt cement.

2. A 6 inch (150 mm) emulsion stabilized base with less than 3.5% asphalt cement does not produce a satisfactory low cost maintenance roadway.

3. A 4 inch (100 mm) emulsion stabilized base does not produce a satisfactory low cost maintenance roadway.

4. A 2 inch (50 mm) asphalt concrete surface would be necessary on many roads to provide a low maintenance roadway using emulsion stabilized limestone screenings.

ACKNOWLEDGEMENT

Research project HR-309 was sponsored by the Iowa Highway Research Board and the Iowa Department of Transportation. Partial funding for this project was from the Secondary Road Research Fund in the amount of $78,760.

The authors would like to extend their thanks to the Linn County Board of Supervisors, Iowa State University and the Iowa DOT for their support in developing and conducting this research. Professor James Hoover of Iowa State University authored Appendix B contained in this report. We also wish to thank Vulcan Industries for their participation and cooperation in the project.

REFERENCES

1. Hoover, J. M., Hunter, J. G., "Asphalt Emulsion as Roadway Stabilizer for Waste Limestone Aggregate," ERI Project 1899, Engineering Research Institute, Iowa State University, Ames, Iowa, 1987, pp 1-4.

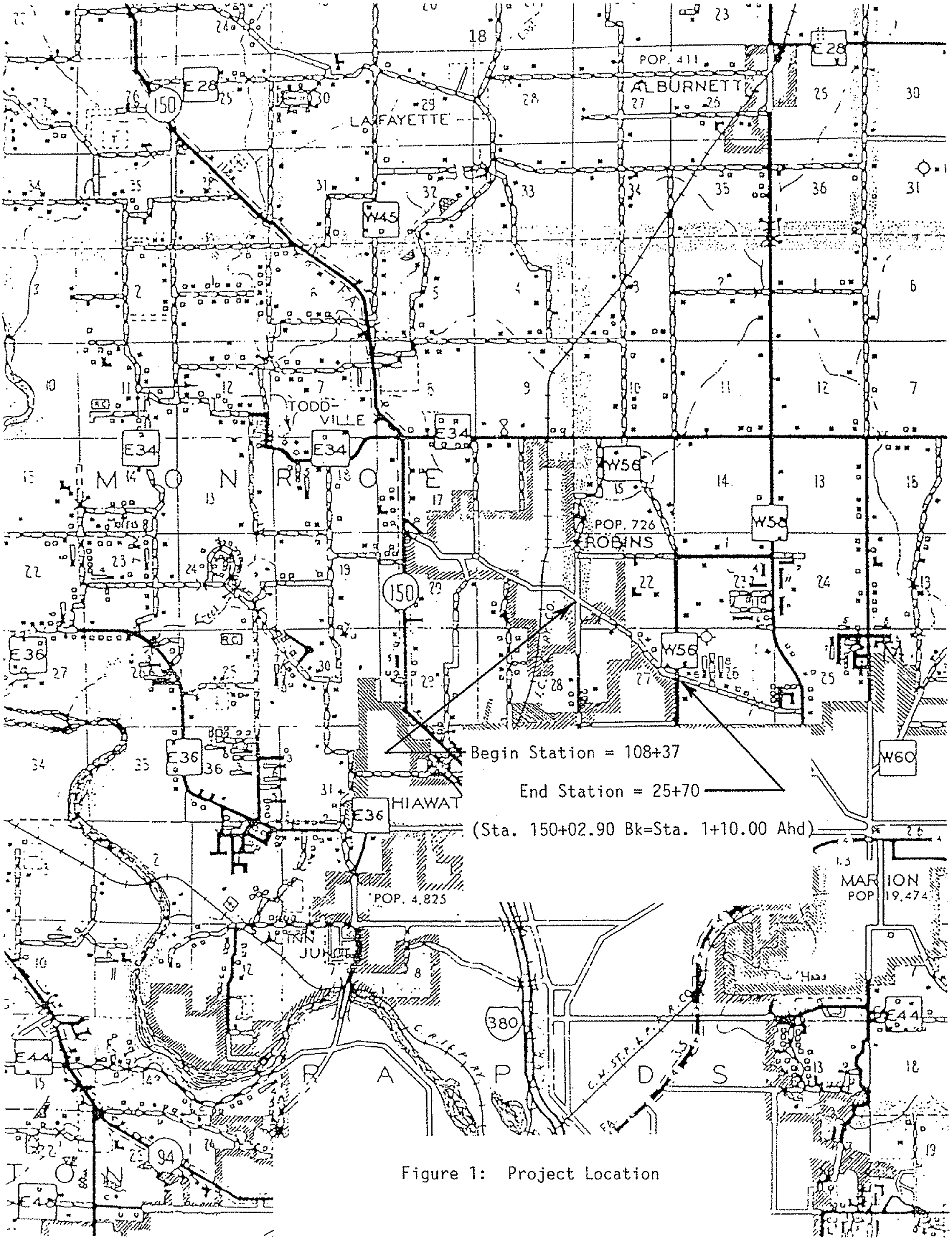


Figure 1: Project Location

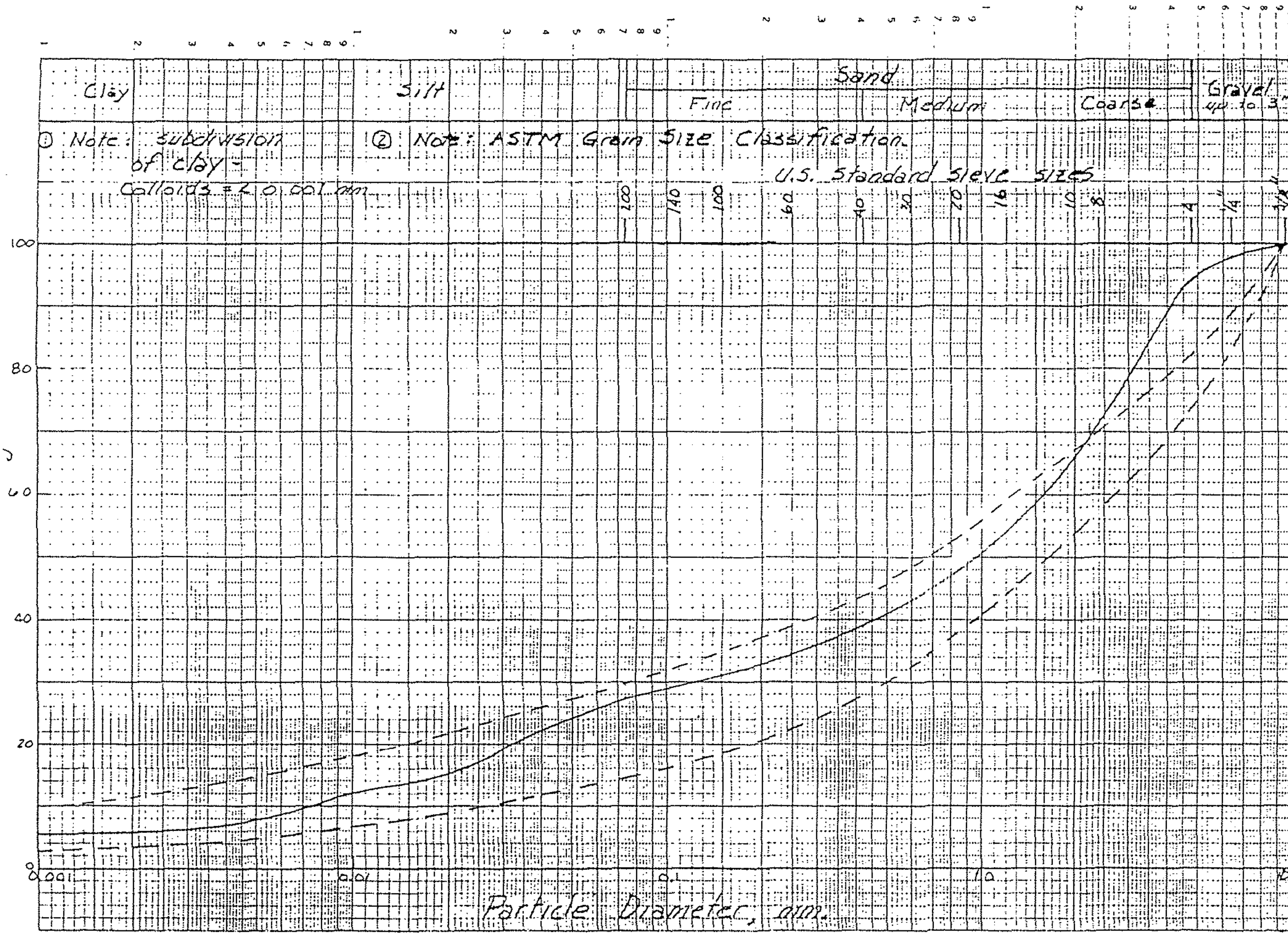
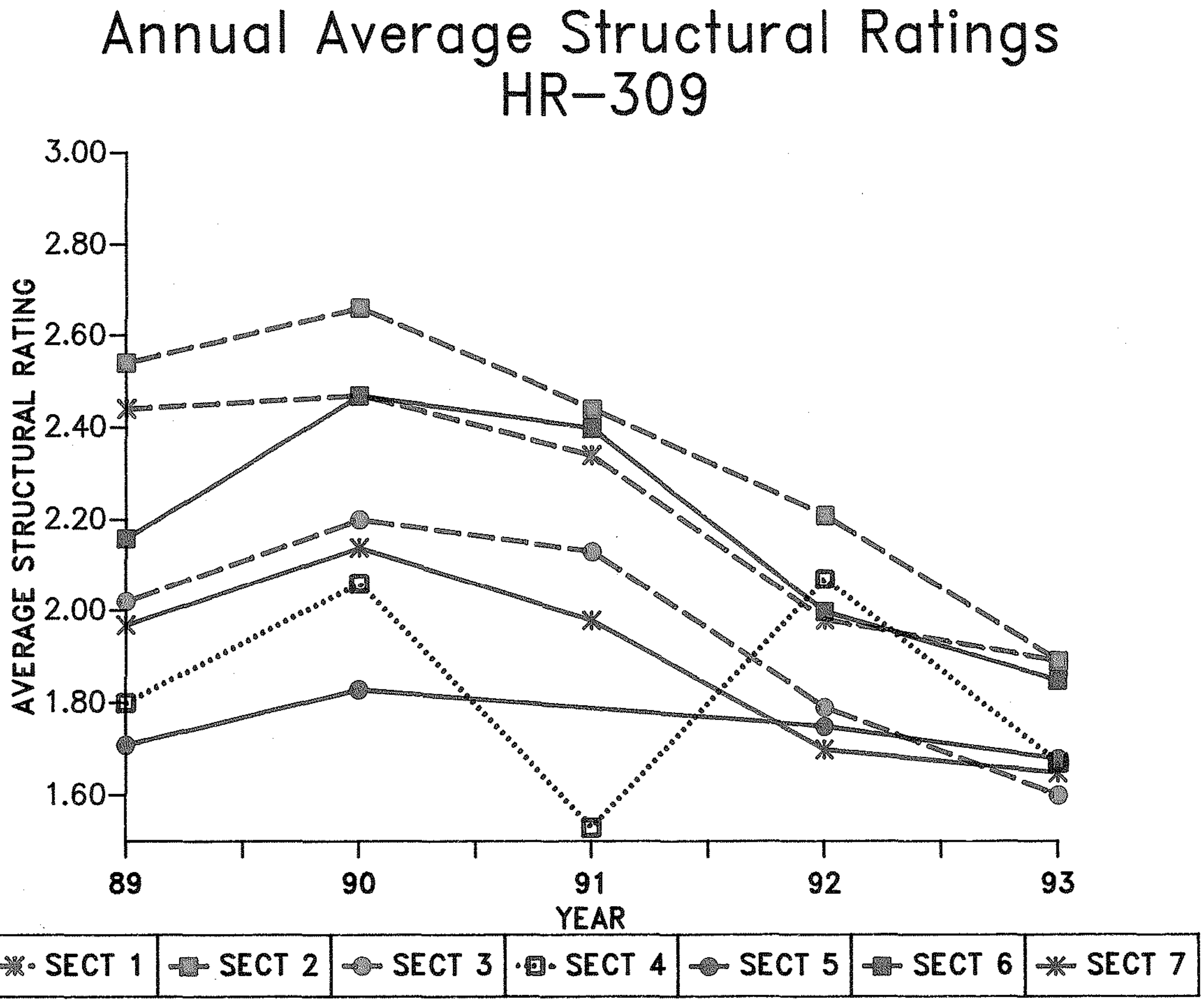


Figure 2: Waste Limestone Aggregate
Gradation Curve

FIGURE 3



Annual Average Structural Ratings
HR-309
AVERAGE STRUCTURAL RATING
3.00
2.80
2.60
2.40
2.20
2.00
1.80
1.60
89
90
91
92
93
YEAR
SECT 1
SECT 2
SECT 3
SECT 4
SECT 5
SECT 6
SECT 7

FIGURE 4

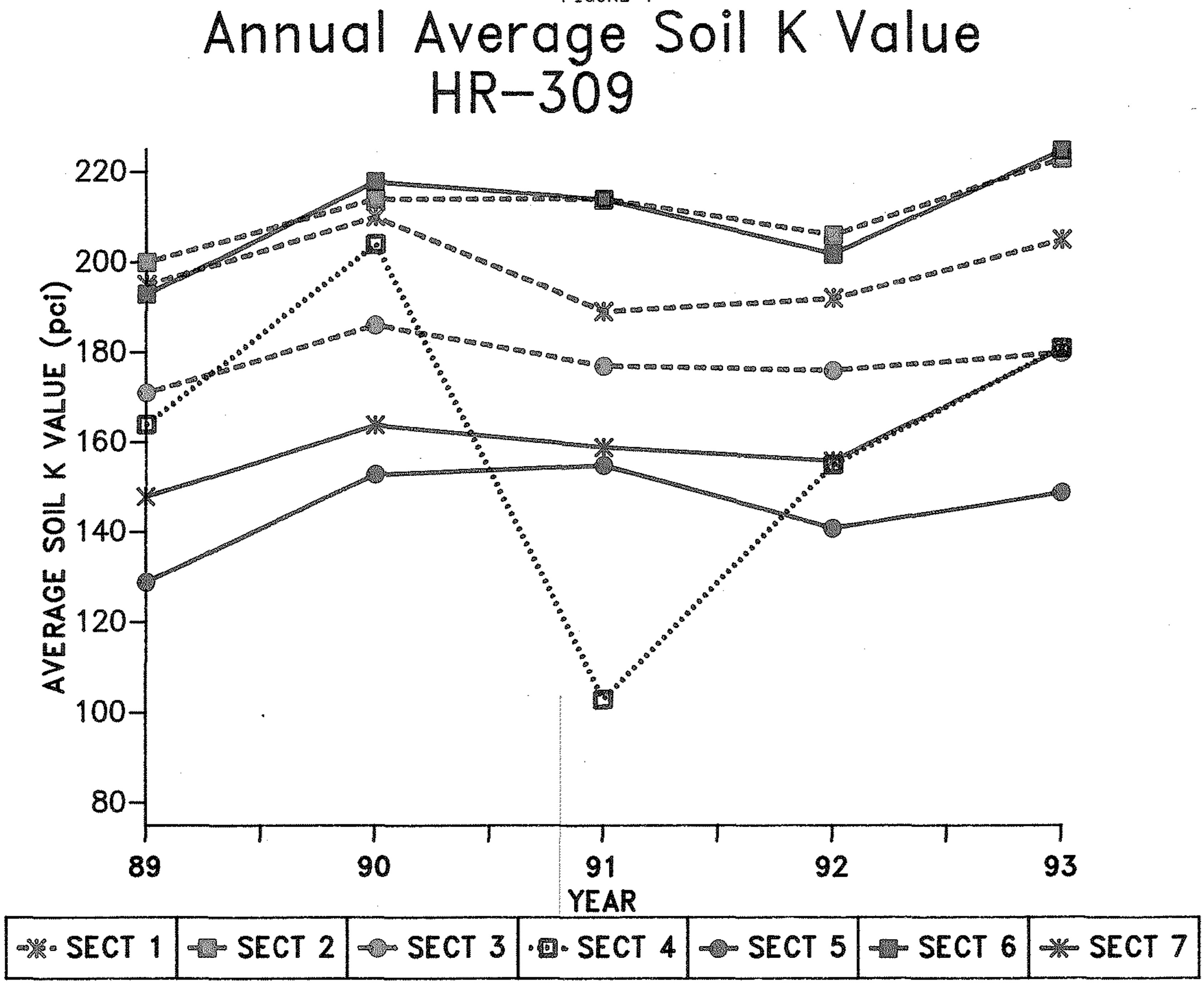
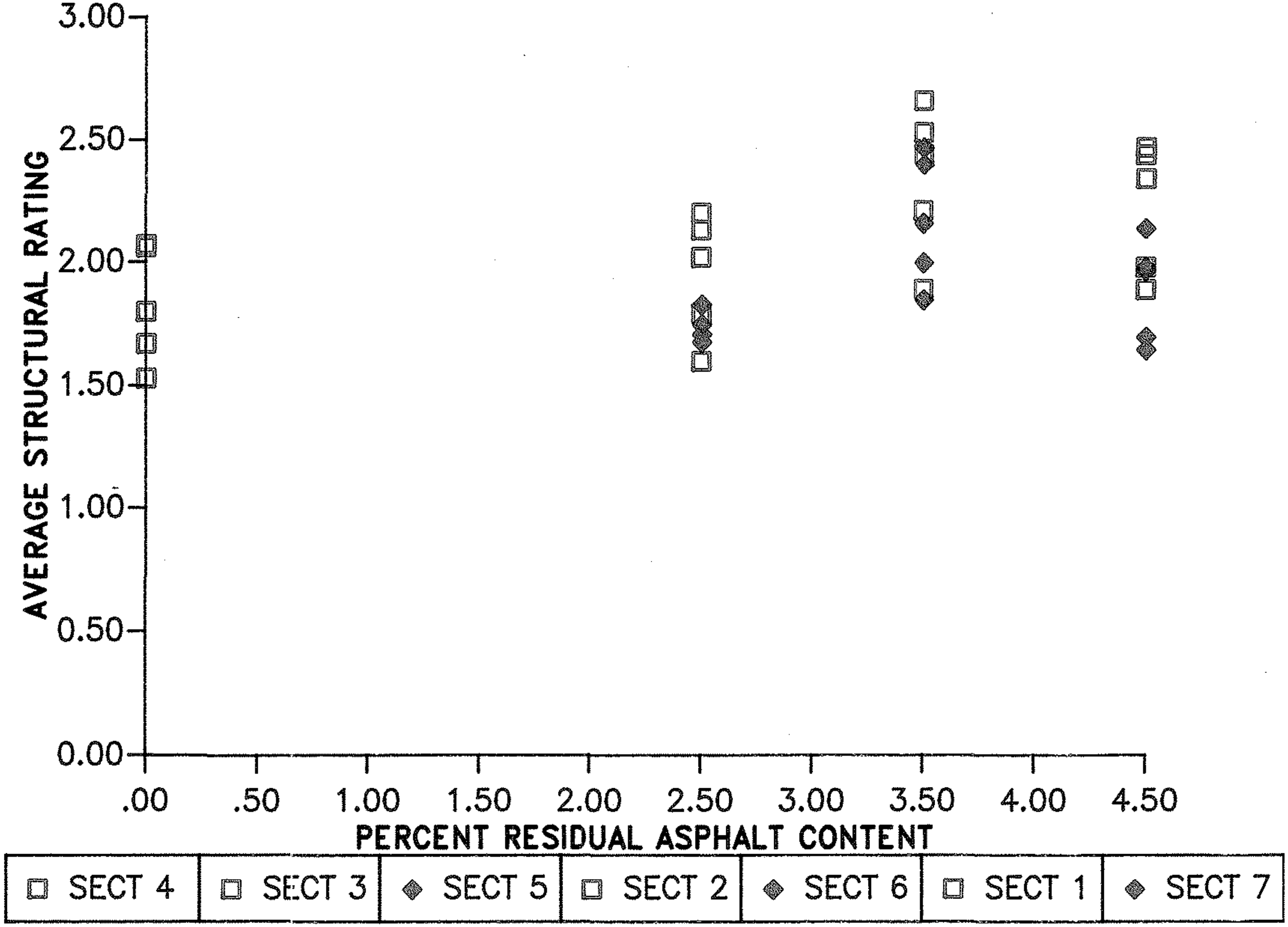

Annual Average Soil K Value
HR-309
AVERAGE SOIL K VALUE (pci)
220
200
180
160
140
120
100
80
89
90
91
92
93
YEAR
SECT 1
SECT 2
SECT 3
SECT 4
SECT 5
SECT 6
SECT 7

FIGURE 5

ANNUAL AVERAGE STRUCTURAL RATING RELATED TO ASPHALT PERCENTAGE HR-309



Appendix A
Contract

Form 740323 6-75

CONTRACT

Kind of Work Bituminous Base Miles 1.255
Project No. LFAC-910-88 County Linn

THIS AGREEMENT made and entered by and between Linn County, Iowa, by its Board of Supervisors consisting of the following members: B. Joseph Rinas, Kenneth A. Schriner and Jean E. Oxley, party of the first part, and Vulcan Materials Company of Cedar Rapids, Iowa, party of the second part.

WITNESSETH: That the party of the second part, for and in consideration of One hundred thirty eight thousand four hundred seventy-two & 87/100 Dollars ($ 138,472.87) payable as set forth in the specifications constituting a part of this contract, hereby agrees to construct in accordance with the plans and specifications therefore, and in the locations designated in the notice to bidders, the various items of work as follows:

Item No.	Item	Quantity	Unit Price	Amount
	Linn County project LFAC-910-88, bituminous base on East Main Street from Council Street to C Avenue.			
1	Base Bituminous Treated Aggregate	4,498 Ton	14.84	66,750.32
2	Base Untreated	875 Ton	11.81	10,333.75
3	Asphalt Emulsion CSS-1	58,840 Gal.	0.65	38,246.00
4	Primer or Tack Coat Bitumen	3,976 Gal.	1.10	4,373.60
5	Binder Bitumen, Furnish and Apply MC-3000	5,522 Gal.	1.10	6,074.20
6	Aggregate, Cover Furnish & Apply ½" Size	230 Ton	17.50	4,025.00
7	Shoulders, Type B Granular	1,156 Ton	7.50	8,670.00
	Total			$ 138,472.87

The Standard Specifications Series 1984 of the Highway Division of the Iowa Department of Transportation and current supplemental specifications shall apply to construction work on this project.

Special Provision - Linn County Ordinance #1-1-1987 and Resolution 1987-1-5 covering minimum wage scale shall apply to this project provided the contractor's bid and subsequent award of contract for the work is more than $75,000.00.

Linn County Supplemental Specification for Asphalt Emulsion/Waste Limestone Aggregate Construction shall apply to this project.

Said specifications and plans are hereby made a part of and the basis of this agreement, and a true copy of said plans and specifications are now on file in the office of the County Auditor under date of May 24, 19 88.

That in consideration of the foregoing, the party of the first part hereby agrees to pay to the party of the second part, promptly and according to the requirements of the specifications the amounts set forth, subject to the conditions as set forth in the specifications.

That it is mutually understood and agreed by the parties hereto that the notice to bidders, proposal, the specifications for Bituminous Base Project No. LFAC-910-88 County, Iowa, the within contract, the contractor's bond, and the general and detailed plans are and constitute the basis of contract between the parties hereto.

That it is further understood and agreed by the parties of this contract that the above work shall be commenced on or before, and shall be completed on or before;

Approx. or Specified Starting Date or Number of Working Days	Specified Completion Date or Number of Working Days
15 Working Days	9-1-88

That time is the essence of this contract and that said contract contains all of the terms and conditions agreed upon by the parties hereto.

It is further understood that the second party consents to the jurisdiction of the courts of Iowa to hear, determine and render judgement as to any controversy arising hereunder.

IN WITNESS WHEREOF the parties hereto have set their hands for the purposes herein expressed to this and three other instruments of like tenor, as of the 29 day of June, 19 88.

Approved: Department of Transportation

By Harvey H. Olson
Contracts Engineer

Date JUL 13 1988

Linn County, Iowa
Party of the first part

By Jean E. Oxley
Chairman

Vulcan Materials Company

By [signature]
Party of the second part

Appendix B
Iowa State University
Test Report

SOIL/AGGREGATE PROPERTIES AND CLASSIFICATION

The soil/aggregate material used for construction was a waste limestone screenings provided from the Vulcan Materials quarry near Robbins, Iowa.

Figure 1 shows the average particle size distribution curve for several soil/aggregate samples removed from the stock piling operations during construction of the test sections. Included on the graph are dashed boundaries indicating the general limits of a well graded soil/aggregate mix. The term "well graded" refers to that gradation needed to achieve maximum densification under a given compactive effort. As noted in the plot, the soil/aggregate shows a larger quantity of gravel and coarse sand than that considered to be well graded. The uniformity coefficient of 165, Table 1, would indicate a moderately well graded material, whereas a well graded material would have a uniformity coefficient in excess of 200, and a poorly graded material would exhibit a uniformity coefficient of 10 or less. Table 1 presents additional average physical properties and classifications of the soil/aggregate used during construction.

Table 1. Physical Properties and Classification.

Particle Size	
Gravel (> 4.76 mm), %	5.7
Sand (4.76-0.074 mm), %	66.6
Coarse sand (4.76-2.00 mm), %	28.4
Medium sand (2.00-0.42 mm), %	26.9
Fine sand (0.42-0.074 mm), %	11.3
Silt (0.074-0.005 mm), %	19.8
Clay (< 0.005 mm), %	8.0
Colloids (< 0.001 mm), %	5.6

46 6013

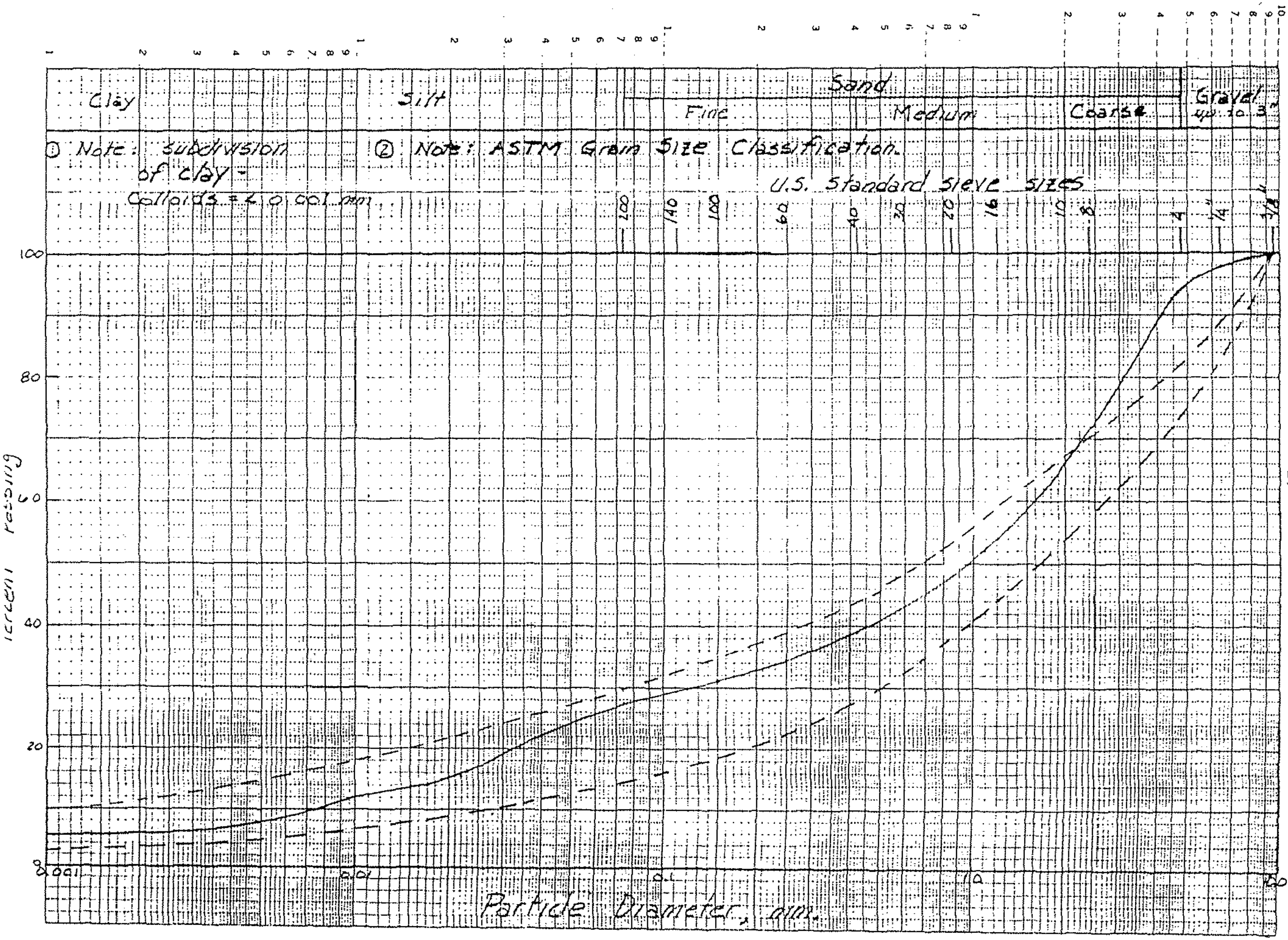

Clay
Silt
Sand
Fine
Medium
Coarse
Gravel
4φ to 3"
① Note: subdivision of clay -
Colloids = < 0.001 mm
② Note: ASTM Grain Size Classification.
U.S. Standard sieve sizes
200
140
100
60
40
30
20
16
10
8
4
1/4"
3/8"
100
80
60
40
20
0
Percent Passing
0.001
0.01
0.1
1.0
10
Particle Diameter, mm.

Table 1. Physical Properties and Classification. (CONTINUED)

Effective size, mm	0.0095 mm
Uniformity coefficient	165
Atterberg Limits	Non-plastic
AASHTO classification	A-2-4(0)
Unified classification	SM
Specific gravity	2.72
Zeta potential, mv	-17
pH	9.4

EMULSIFIED ASPHALT

The 1987 Linn County study,[1] on the use of emulsified asphalts in conjunction with waste limestone screenings, revealed that best results were achieved with a CSS-1 emulsion having a zeta potential of +18 mv; a value almost equal, but opposite in charge to the soil/aggregate used during the study.[1] Based on these initial results, and the fact that the soil/aggregate used for construction had a zeta potential of -17 mv, a CSS-1 emulsion having a zeta potential of about +18 mv, was recommended for use in construction of the test sections. Analysis of emulsion samples removed from two tankers during construction, showed zeta potential values of +34.6 mv and +27.6 mv, respectively.

Following is a listing of test results for the emulsion produced for the Linn County project, as supplied by Koch Materials Company, Asphalt Division, Dubuque, Iowa:

Weight per gallon @ 60°F	8.53
Viscosity @ 77°F	23S
Sieve test, %	0
Pen of residue from distillation	86
Residue from distillation, %	61.5
Oil from distillation	0

LABORATORY INVESTIGATION

As previously noted, several soil/aggregate samples were removed from stockpiling operations during construction, in order to provide a large composite sample for future laboratory tests when combined with asphalt emulsion samples removed from selected emulsion tank trucks. These future tests are for the purpose of providing correlations with the 1987 study,[1] as well as studies performed on field mixed materials noted below.

During construction, a series of samples were randomly removed from each test section mix immediately after spreader laydown of the respective treated bases, and prior to field compaction. Each sample series was then divided, one portion being placed in sealed containers for return to Spangler Geotechnical Laboratory (SGL) for molding and testing, the second portion being compacted on site in Proctor molds at AASHTO T-99, ASTM D 698, compactive energy; the latter specimens then being wrapped and sealed for transport to SGL for testing. The following laboratory tests were then performed on (1) plant mixed field laboratory compacted specimens, and/or (2) plant mixed SGL compacted specimens.

Indirect Tensile Strength

Indirect tensile strength (ITS) tests were performed on Proctor size specimens field molded during construction, from uncompacted mixes removed from the roadway. All specimens were wrapped in plastic and foil immediately following molding in order to maintain the molded moisture content until tests could be performed. Prior to testing, the specimens were air cured for 72 hrs.

The indirect tensile test is a method for evaluating the tensile or flexural capabilities of a stabilized mix. Testing is accomplished by compressing each sample laterally between two diametrically opposing strip loads. Under this condition, a fairly uniform stress is developed internally, acting perpendicular to and along the diametral plane of the applied load resulting in a splitting of the specimen. Tensile strength, S_t, is calculated from the equation:

$$S_t = 2P/\pi DL$$

where: P = maximum load

D = specimen diameter

L = specimen length

Table 2 presents the average indirect tensile strength values calculated from duplicate specimens.

Table 2. Indirect Tensile Strength.

Nominal Treatment	Field Molded M.C.,%	Dry Density, pcf	S_t, psi	Test M.C.,%
Untreated	6.0	124.0	21.8	0.99
2.5% CSS-1h	5.0	120.6	9.8	1.19
3.5% CSS-1h	6.6	123.4	16.1	1.32
4.5% CSS-1h	6.5	122.3	13.5	1.26

Addition of the emulsified asphalt decreased density and tensile strength values from those of the untreated limestone screenings, though maximum treated values of each appeared at the 3.5% residual asphalt content level. In general, S_t values of these field mixes were somewhat less than attained in the 1987 laboratory study.[1]

Freeze-Thaw

A major problem affecting pavement courses in any climate where freezing occurs is caused by frost action. Frost heave occurs when water, primarily absorbed through capillary action, freezes and expands, causing a breakdown of the particle to particle matrix structure. Frost boils occur during thawing resulting in high moisture retention causing a loss of a base material's load bearing capability. Continuous freeze-thaw cycles can reduce a soil structure to a loose collection of soil and aggregate particles providing little or no load support. A stabilizing agent must control the effects of heaving, while maintaining the soil structure, in order to provide load support during severe freeze-thaw cycling.

Freeze-thaw deterioration was analyzed using Proctor size field mixed and field molded specimens. The test duplicates normal field conditions of freezing from the surface while free water is available at the specimen base for capillary absorption. As temperature drops, absorption increases, moving water to the freezing front, allowing development of ice lensing.

Prior to testing, all specimens were air cured for 72 hrs. Following F-T testing, all specimens were subjected to Iowa K-Tests (described in a later section) to evaluate strength and stability retention.

The volumetric F-T test is accomplished by placing specimens in plexiglass holders having perforated base plates. The holder and specimens are then placed in Dewar flasks containing water in contact with the specimen base, thus allowing capillary saturation. To keep the water in the flask from freezing, a 6 watt bulb maintains a water temperature of approximately 35°F. Once set up, initial height measurements are taken so that volumetric changes can be monitored. The test apparatus and specimens are then placed in a freezer maintained at approximately 20°F for 16 hrs. After the freeze cycle, the apparatus and specimens are removed from the freezer, and maintained at room temperature for 8 hrs. Height measurements are taken after each freeze and thaw cycle. Upon completion of ten cycles, the specimens were removed from the plexiglass holders and K-tested for strength and stability.

Effect of volumetric changes during F-T may be viewed through two criteria. First, residual elongation may be described as that quantity of heaving which occurs in a material as the difference between zero change, and either freeze or thaw volumetric change, during any number of cycles; i.e., the departure of the freeze-thaw curve from the abscissa of the plot. In addition, residual change often indicates water absorption and expansion characteristics of the material being tested, which does not dissipate through gravitational drainage during thawing. Second, cyclic change is the difference between freeze and thaw volumetric changes during any single cycle, and represents a volumetric expansion due to ice lense formation during freezing, or a volumetric shrinkage due to thawing coupled with downward gravitational flow. Development of a sudden cyclic elongation is most often attributable to a stabilized soil-product matrix (structure) breakdown with accompanying loss of overall stability. Large combinations of both residual and cyclic

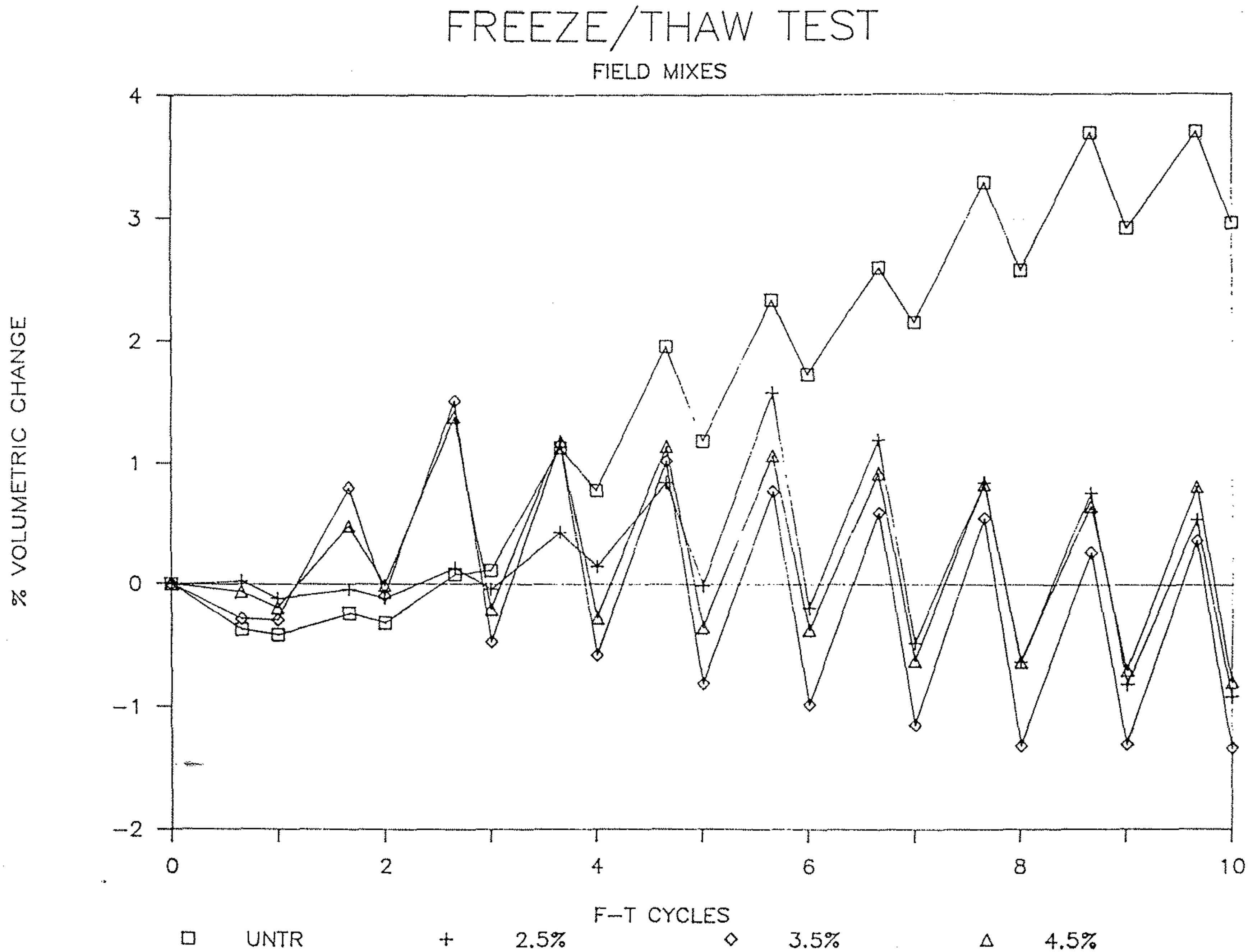


Figure 2. Percent Volumetric Change versus Number of Freeze-Thaw Cycles, Field Mixed-Field Molded Specimens.

change represent a definite lack of freeze-thaw stability, and accompanying loss of strength. Very low combinations of each, would show a soil or soil-additive composite having little or no frost heave susceptibility with an accompanying retention of strength.

Figure 2 presents the average volumetric freeze-thaw results for the field mixed and molded specimens. As noted, the untreated specimens produced considerable residual expansion during the ten cycles, indicating water absorption with accompanying expansion. Cyclic variation was relatively minimal with the untreated until about the third cycle, suggesting structural deterioration thereafter.

All emulsion treated specimens performed in a similar fashion with little variation between concentrations. Residual change was quite small for each of the emulsion treated mixes, and definitely less than the untreated, suggesting relatively good control of heaving effects. As noted in Table 3 however, emulsion treatment did not prevent capillary moisture intrusion during F-T testing, since average moisture contents following 10 cycles were similar to that of the untreated soil/aggregate. Cyclic volumetric changes of the treated specimens were somewhat larger than the untreated, becoming noticeable at about cycles 2 and 3. While the cyclic changes suggest some potential for matrix breakdown, K-tests after 10 cycles of F-T showed good stability; the cyclic changes thus potentially indicating some elastic abilities of the soil/aggregate matrix when treated with the emulsion.

Table 3. Average Moisture and Density Summary of F-T Specimens.

Nominal Treatment	Field Molded M.C., %	Dry Density, pcf	Test M.C. After 10 F-T Cycles, %
Untreated	6.03	124.3	7.82
2.5% CSS-1h	7.09	122.2	7.31
3.5% CSS-1h	6.59	117.7	8.88
4.5% CSS-1h	5.94	117.6	7.58

Iowa K-Test

The K-Test simulates an undrained, relatively rapid static field loading stress state. Essentially, the test is a variable restraint stress-path triaxial shear test.[2] The test provides qualitative values of cohesion (c) and angle of friction (ϕ); parameters which are not unlike those produced from triaxial shear tests, but are not quantitative duplicates thereof. Values of c-ϕ may be used in variations of the classic Terzaghi analysis to obtain the bearing capacity (q_o). When coupled with vertical loading, axial deformations converted to axial strains, provide determination of a pseudo-elastic modulus (E).[3] A brief explanation of each parameter is as follows:

1. Stress Ratio (K). A nominal uncorrected ratio of horizontal to vertical stress induced in a loaded specimen. May be viewed as a qualitative indicator of lateral stability. Values of K should never exceed 1.00. The smaller the K value, the greater the improvement in lateral stability; an asset in control of movements in a compacted earth fill, or control of rutting in a pavement course.

2. Angle of Internal Friction (ϕ). Refers to the sum of sliding friction plus interlocking forces within the soil/aggregate matrix. Related to stability and bearing capacity of a compacted material.

3. Cohesion (c). A parameter indicative of the amount of attractive (electro-static) and adhesive forces between particles in a soil matrix. Related to stability and bearing capacity of a compacted material.

4. Psuedo-Elastic Modulus (E). An approximate relationship between stress and strain of a soil during vertical loading. Thus E is indirectly related to compressibility. Since soil is an elastic-plastic material, values of E should be viewed only from a qualitative standpoint.

5. Ultimate Bearing Capacity (q_o). Calculated from the classic Terzaghi bearing capacity equation for soil under a surficially applied circular footing. In its determination, q_o utilizes c-ϕ values, as well as soil wet unit weight.

Parameters obtained from the K-Test must be considered in a developmental stage, and should not be used for design purposes. They are viewed herein from a qualitative context of comparison of the untreated and treated mixes.

Table 3 shows the average molded moisture content and dry density at time of field molding, and moisture content of the specimens following freeze-thaw as utilized in the K-Test. All specimens had similar cured moisture contents of approximately 1.2% prior to freeze-thaw testing. Following F-T testing, all of the treatments exhibited similar moisture contents.

Table 4 presents results of the K-Test performed on the F-T specimens. While friction angles tended to decrease with increasing residual asphalt contents, cohesion of the treated mixes was considerably higher than the untreated (0). The slight variation in cohesion of the 3.5% mix may be attributed to the slight variation in moisture content thereof noted in Table 3.

Stress ratios increased slightly with residual asphalt content. The very small increase in K-ratios suggest a slight loss of lateral stability, and increase in rutting potential, though the increases are so small as to suggest no loss in either mode. The latter concept is also validated in that none of the K-ratios were greater than those produced by an A-7-0(12), CL soil, stabilized with 4% of a CSS-1 emulsion and constructed in Pottawattamie County, Iowa, in 1979;[4] a base stabilization project still in service with double chip coat surfacing.

Table 4. Iowa K-Test Summary.

Nominal Treatment	ϕ^{o}	c, psi	E, psi	K	q_o, psi
Untreated	40.2	0	5889	0.236	31.4
2.5% CSS-1h	36.6	2.5	3272	0.245	179.0
3.5% CSS-1h	37.3	1.8	2953	0.243	144.1
4.5% CSS-1h	34.6	2.9	2612	0.268	157.9

Increased residual asphalt content produced decreases in the pseudo-elastic moduli (E) indicating some potential for compressibility and rutting, if the base materials were ever subjected to capillary saturation during freezing and thawing cycles, and illustrating the need for adequate external drainage.

Cohesion and friction angle (c-ϕ) values were used to compute the ultimate bearing capacity (q_o) against shear. For this purpose, a surface load applied to a 12 inch diameter plate was assumed; this assumption corresponding to the approximate contact area of a truck tire. If it is assumed that tire contact pressure ranges from 75-125 psi, the q_o value obtained from the untreated mix, Table 4, would suggest an early failure if used as a base course under a thin chip and seal surface and allowed to reach saturation. However, each of the treated mixes, Table 4, indicated more than adequate load bearing support under similar conditions.

While each of the K-Test parameters were affected by frost action and saturation, the combined F-T and K-Test data suggest that the addition of the asphalt emulsion as a stabilizing agent may provide significant control of the effects of frost heave, while maintaining sufficient stability and load bearing support following a spring thaw.

Marshall Test

The Marshall test is one form of mix design testing used to ascertain optimum residual asphalt content. Results can also be applied to thickness design of the various courses of a flexible pavement system.

Quadruplicate four-inch diameter by 2.5-inch high cylindrical specimens were molded in the laboratory using mixes obtained from the field, while maintaining moisture contents achieved during construction. Compaction consisted of 75 blows per side with a 10-lb. hammer, dropped 18 inches. Following molding, all specimens were air cured for 72 hrs, after which two specimens of each mix were Marshall tested, the remaining two allowed to capillary saturate for 96 hrs. prior to testing.

In general, a mix should meet or exceed the following criteria:[5]

a. Minimum stability of 500 lbs.

b. Maximum stability loss of 50% after 96 hr. saturation.

c. Maximum of 4% absorbed moisture after 96 hr. saturation.

d. Flow values between 0.80 and 0.180 inch.

While limitations are not generally established for percent air voids in materials of predominant sand size, flow values are important in preventing distress of a pavement system. Mixes having flow values below the noted range tend to be brittle, causing premature cracking. Above the range noted, mixes tend to be soft, increasing rutting potential. High flow values are also usually accompanied by low stability values.[6] The optimum residual asphalt content is generally chosen as that which provides maximum saturated stability, but may be adjusted + or - depending on moisture absorption, percent loss of stability, voids, and coating of particles. If one or more of the criteria are not met, the mix may be considered inadequate.

Table 5 presents the average Marshall test data for specimens which were laboratory molded from the field mixes. Densities tended to vary between the different concentrations of residual asphalt instead of decreasing with increasing asphalt contents, due to the varying moisture contents encountered during construction. Optimum moisture content for maximum densification of the treated mixes should have been 7.0% or slightly greater.

As stability is dependent on density, the variations mentioned above are reflected in both the cured and saturated stability values for the different mixes. Both cured and soaked stability values were well above the minimum criteria, with the exception of the untreated mix, which failed during saturation. It should be noted that while stabilities exceeded minimum specifications, percent stability losses due to saturation exceeded maximum criteria.

Flow values of the cured and saturated mixes were all within the

0.80-0.180 inch range. Random variability of the flow values, however, appeared related to density variations.

Absorbed moisture data, Table 5, is the numerical difference between moisture contents following saturation and curing. Little variation in absorption was evident between the different residual asphalt contents. However, a drastic reduction in moisture absorption was apparent between the untreated and treated mixes. Quantity of absorbed moisture for each of the treated mixes exceeded the 4% maximum by about 1.0%.

In terms of Marshall test criteria, each of the mixes might be questionable for use as a pavement course. However, due to the experimental nature of these mixes, only actual in-situ performance with time will determine the effectiveness of the emulsion and waste limestone base course materials.

Table 5. Marshall Test Summary.

Nominal Treatment	Molded Moisture, %	Dry Density, pcf	Cured Stability, lbs	Soaked Stability, lbs	Stability Loss, %
Untreated	6.17	136.4	6257	---	100
2.5% CSS-1h	5.46	132.0	4365	1410	67.70
3.5% CSS-1h	7.02	135.0	6497	1895	70.83
4.5% CSS-1h	6.92	131.1	5245	1262	75.94

	Cured Flow,	Soaked Flow,	Cured Test MC,	Soaked Test MC	Absorbed Moisture
Untreated	0.117	---	0.92	12.19	11.27
2.5% CSS-1h	0.095	0.137	0.86	5.84	4.98
3.5% CSS-1h	0.128	0.123	0.83	5.65	4.82
4.5% CSS-1h	0.123	0.145	1.11	6.40	5.29

	Cured Voids	Soaked Voids
Untreated	19.5	---
2.5% CSS-1h	19.8	17.2
3.5% CSS-1h	16.4	15.2
4.5% CSS-1h	17.8	16.0

Residual Asphalt Contents

Asphalt contents of each emulsion treated mix were determined in accordance with ASTM Designation D2172, Method B, Quantitative Extraction of Bitumen from Bituminous Paving Mixtures. Samples used for this test were randomly selected from the field mixed materials obtained prior to compaction. Results indicated 2.35, 3.15, and 4.05% residual asphalt for the nominal contents of 2.5, 3.5, and 4.5%. While the extracted values were less than the nominal mix design values, it must be noted that a period of time elapsed between construction mixing and extraction testing, a condition often yielding somewhat lower than targeted bitumen contents.

SUMMARY

Laboratory tests conducted on the field mixed materials will ultimately be included in correlations with additional laboratory tests, the 1987 laboratory feasibility investigation,[1] and periodic in-situ performance evaluations. Additional laboratory studies presently being conducted include trafficability, CBR, and Resilient Modulus testing. Field tests being performed in-situ include moisture-density, Clegg Impact Values, and Benkelman Beam deflection tests. Such laboratory and field tests will be presented in subsequent reports. While inclusion of major performance and laboratory conclusions herein would be premature, and particularly without benefit of at least one full year of field climatic conditions, as of the date of this report, all test sections appear in excellent condition.

REFERENCES

1. Hoover, J.M., and Hunter, J.G. "Asphalt Emulsion as Roadway Stabilizer for Waste Limestone Aggregate." Final Report, ERI Project 1899, County Engineer, Linn County, Iowa. 1987.

2. Spangler, M.G., and Handy, R.L. Soil Engineering, 4th edition, New York, Harper & Row, Publishers. 1982.

3. Handy, R.L., Lutenegger, A.J., and Hoover, J.M. "The Iowa K-Test." Transportation Research Record 827, Transportation Research Board, Washington, D.C. 1981.

4. Hoover, J.M., et al. "Mission Oriented Dust Control and Surface Improvement Processes for Unpaved Roads." Final Report, Project HR-194, Highway Division, Iowa Department of Transportation. 1981.

5. The Asphalt Institute. A Basic Asphalt Emulsion Manual. Manual Series No. 19 (MS-19), March, 1979.

6. Krebs, Robert D., and Walker, Richard D. Highway Materials, New York, McGraw-Hill, Inc. 1971.

Appendix C
Post Construction Test Results

HR-309
An Investigation of Emulsion Stabilized
Limestone Screenings
Field Test Results

Road Rater Results

Section	Description	80% Structural Rating	Soil K Value
1	6", 4 1/2% A.C.	4.25	218
2	6", 3 1/2% A.C.	4.75	210
3	6", 2 1/2% A.C.	3.25	208
4	6", Untreated	3.55	223
5	4", 2 1/2% A.C.	3.55	235
6	4", 3 1/2% A.C.	3.85	235
7	4", 4 1/2% A.C.	2.90	197

Smoothness Test Results

Section	BPR Roughometer Roughness, In./Mi.		25 Ft. California Profilometer Roughness, In./Mi.	
	EB	WB	EB	WB
1	144	131	19.3	15.9
2	133	146	12.6	14.2
3	148	146	22.3	19.5
4	161	169	19.3	34.3
5	152	146	31.5	25.7
6	125	123	27.6	16.7
7	117	132	17.6	24.6

1 in./mi. - 15.8 mm/km
1 in. = 25 mm

Appendix D
Construction Materials and Costs

ITEM	UNIT	RATE	QUANTITIES CONTRACT	ACTUAL	OVERRUN/ UNDERRUN	AMOUNTS CONTRACT	ACTUAL	OVERRUN/ UNDERRUN
Bituminous Treated Aggregate	Ton	14.84	4,498	4,737	+239	66,750.32	70,297.08	+3,546.76
Base, Untreated	Ton	11.81	875	541.77	-333.23	10,333.75	6,398.30	-3,935.45
Asphalt Emulsion CSS-1	Gal.	0.65	58,840	66,049	+7,209	38,246.00	42,931.85	+4,685.85
Primer or Tack Coat Bitumen	Gal.	1.10	3,976	2,607	-1,369	4,373.60	2,867.70	-1,505.90
Binder Bitumen, Furnish and Apply MC-3000	Gal.	1.10	5,522	5,052	-470	6,074.20	5,557.20	-517.00
Aggregate Cover, Furnish and Apply 0.5 inch Size	Ton	17.50	230	243.67	+13.67	4,025.00	4,264.23	+239.23
Shoulders, Type B Granular	Ton	7.50	1,156	756.56	-399.44	8,670.00	5,674.20	-2,995.80
Prime Subgrade			Extra Work Order				3,364.57	+3,364.57
Total						138,472.87	141,355.13	+2,882.26

1 ton = 907 kg
1 gal. = 3.78 L

Appendix E
Construction Photographs

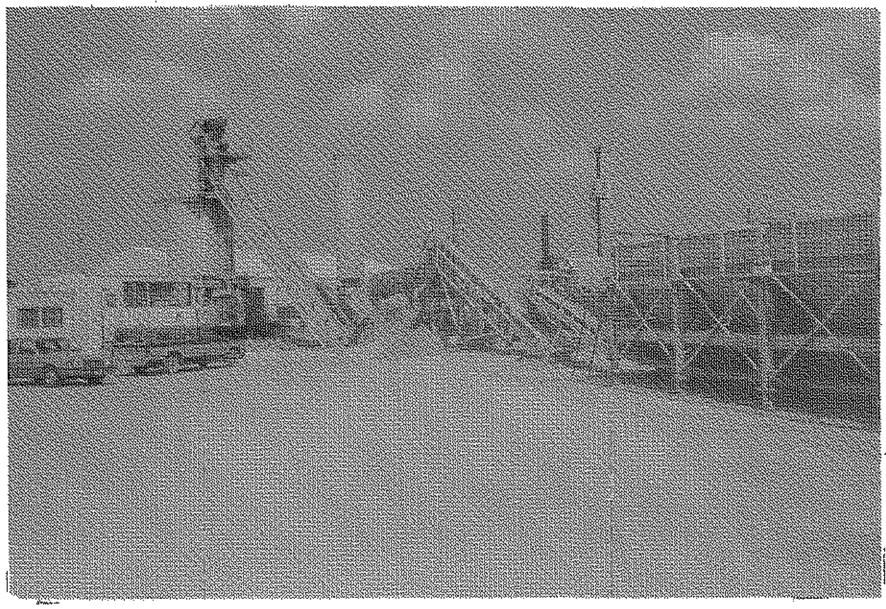


Photo 1: Contractor's drum mixer plant

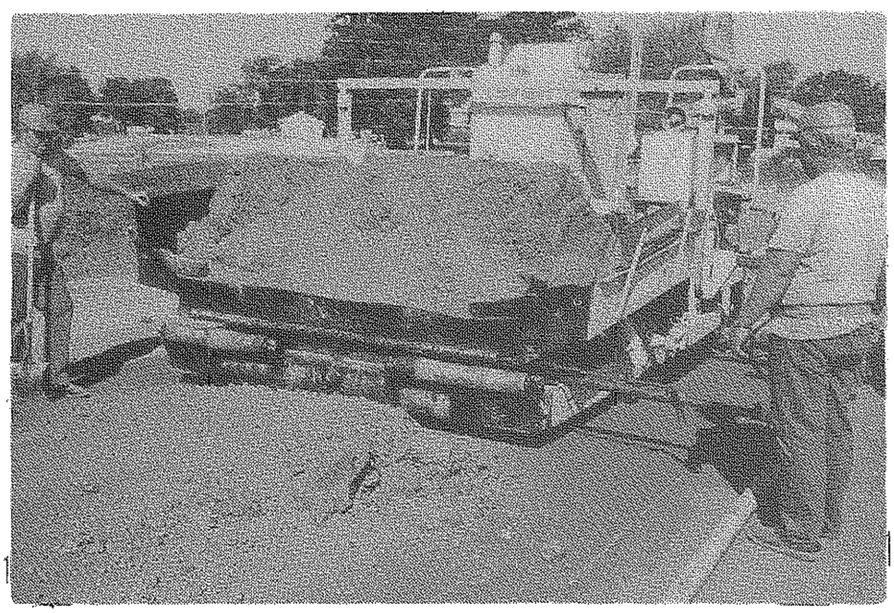


Photo 2: Stiffened mix in asphalt paver

Photo 3: Torn base mat placed using asphalt paver

Photo 4: Padsfoot roller compacting base laid with spreader box

Photo 5: Compacted base prior to final shaping and compaction

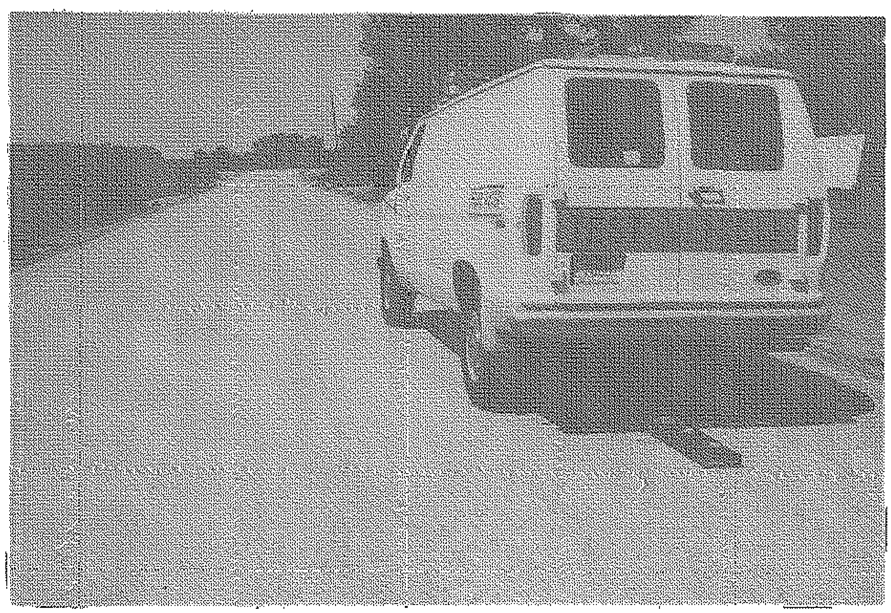


Photo 6: DOT Road Rater testing being conducted on finished roadway