J. M. HOOVER
JULY 1973

Final Report
ISU – ERI – AMES – 72316
Iowa Highway Research Board Project HR-151

SURFACE IMPROVEMENT AND DUST PALLIATION UNPAVED SECONDARY ROADS AND STREETS

Submitted to Iowa State Highway Commission

ERI Project 856-S

ENGINEERING RESEARCH INSTITUTE
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SURFACE IMPROVEMENTS AND DUST PALLIATION
OF UNPAVED SECONDARY ROADS AND STREETS

PART I

PROJECT SUMMARY

CONTENTS

INTRODUCTION

As of December 31, 1970 there were 57,270 miles of Local Secondary roads and 32,958 miles of Farm to Market roads in the Iowa secondary road system*. The Local Secondary system carried a traffic load of 2,714,180 daily vehicle miles, accounting for 32% of all traffic in the secondary system. For all Local Secondary roads having some form of surfacing, 98% were surfaced with gravel or crushed stone. During the 1970 construction year 335 miles of surfaced roads were constructed in the Local Secondary system with 78% being surfaced with gravel or crushed stone.

The total maintenance expenditure for all secondary roads in Iowa during 1970 amounted to $40,086,091*. Of this, 42%, or $17,020,332, was spent for aggregate replacement on existing gravel or crushed stone roads with an additional 31% ($12,604,456) being spent on maintenance other than resurfacing. This amounts to 73% of the total maintenance budget and are the largest two maintenance expenditure items out of a list of 10 ranging from bridges to drainage assessments. The next largest item was 7%, for maintenance of existing flexible bases.

Present costs of high type flexible or rigid pavements range from $40,000 per mile up. Because of high cost, budget limitations, and low mileage of high traffic volume on Local Secondary roads, most Iowa counties are severely restricted as to the number of miles of roads that may be paved each year. Present design and construction standards provide

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*Summary of Iowa County Engineers Annual Highway Reports. Part II of 57th annual report, Iowa State Highway Commission, Ames, Iowa (1970).

adequate means for improvement of grade and cross section of secondary roads followed by: (a) immediate road metal surfacing, or (b) eventual high-type surfacing. There are, at present, no formal provisions for an intermediate type surfacing between these two extremes. Therefore, nearly all Iowa counties are in a situation in which they have only a few miles of high-type pavements with the bulk being surfaced with gravel or crushed stone.

From the maintenance expenditures for 1970 it can be seen that the primary method of maintaining aggregate surfaced secondary roads remains, as it has in the past, as aggregate replacement. Roads continue to rut, washboard, ravel, pothole and become the source for billowing clouds of dust. Loose aggregate makes driving hazardous, and results in cases of vehicle damage including cracked windshields, chipped paint and dents, as well as increased fatalities. During the spring thaw and subsequent rains, many roads become extremely soft, slippery and heavily rutted. As aggregate supplies decrease, inferior quality soft limestones and gravels are being used, resulting in faster degradation contributing to the dusting problem. Dust creates a safety hazard to both passing and oncoming traffic. It is also a definite household nuisance in rural areas, especially in heavily populated regions surrounding larger towns and cities where traffic on unpaved roads may range as high as 500 vehicles per day. The severity of these problems continues to increase as: (a) traffic volume increases, (b) more people move to rural areas surrounding larger towns and cities, and (c) as the current concern over air pollution increases. The Iowa Air Pollution Control Commission has drafted guidelines with respect to "fugitive" dust, which ultimately will affect all unpaved state, county and city roads and streets.

Besides the above problems, most counties are faced with (a) rising costs of high type pavement, (b) rising costs of maintenance, and (c) rising costs of replacement aggregates.

The foregoing considerations dictated the need for finding a means to provide for low-cost surface improvement and dust control, using existing in-place materials, for immediate (and intermediate) use as a treated surface course on unpaved secondary roads — the objective of this research project. Three concurrent phases of study were included in the project: (1) laboratory screenings studies of various additives thought to have potential for long-lasting dust palliation, soil-additive strength, durability, and additive retention potential; (2) test road construction using those additives that indicated promise for performance-serviceability usage; and (3) observations and tests of constructed sections for evaluation of the additive's contribution to performance and serviceability as well as the relationship to initial costs.

Criteria for additives considered as potential dust palliatives and surface improvement agents in this project were:

1. Economic — \$5,000 or less per mile, with \$10,000 as absolute maximum for consideration.
2. Water solubility or water dispersibility — for ease of mixing with soil — but becoming water insoluble after incorporation, providing chemical bonding, waterproofing, or other immunities to deterioration by roadway environment and traffic abrasion for an indefinite period of time.
3. No requirement for specialized handling, or construction equipment, other than normally available within a county road unit or through a contractor.

4. Improvement of strength and density of a roadway surface, though advantageous, was not essential so long as adequate dust control and stability versus time was maintained.
5. Must range from a surface penetrant type to easily mixed in-situ up to six inches in depth, utilizing existing roadway soil/ aggregate materials without importation of additional soil or aggregate.
6. Within the range of criteria 1 through 5, quantity of additive must not exceed 4-5% by dry soil weight.

As in any research project of the magnitude incorporated herein, total resulting benefits will still require years of additional study and research before maximum beneficial concepts of design for dust palliation and surface improvement of unpaved secondary roads and streets is achieved. New additives will be marketed and techniques of application, incorporation, and concept will be improved. However, results of this investigation reported here can be, and to a very limited degree already have been, utilized to a large extent by road units seeking low-cost relief for dust control, aggregate replacement, maintenance, and low-traffic volume roadway design.

DUST

Volumetric air-sampling indicated that the concentration of silt-sized particles in the air behind a car moving at 35 mph on a moderately dusty crushed-rock road was about 100 times the pollution concentration in industrial city air.

Average stationary dust data collected from samplers spaced at various intervals on each side of several roadways indicated that each year 28 tons of dust were accumulated per mile for an average daily traffic flow of 100 vehicles between the edges of a 24-ft-wide surface and a 66-ft right-of-way. From the right-of-way to a distance of approximately 150 ft on either side of the road centerline, an additional 28 tons of dust per year were deposited. And from 150 to approximately 500 ft on either side of the centerline, 44 tons were blown off in the form of dust annually. Thus, for every vehicle traveling one mile of unpaved roadway, once a day, every day of the year, one ton of dust is deposited along a 1000-ft-wide corridor centered on the road; i.e., quantity of dust equals 1 ton/mile/year/vehicle of average daily traffic, and 100 tons per mile of road material per year at a traffic rate of 100 vehicles per day. In addition, an unknown amount of degraded soil/aggregate road material annually is washed off the surface by erosion, kicked off by traffic, or bladed off during snow removal.

Of the field projects constructed and/or observed while conducting in-place surface improvement testing, dust contents were reduced from approximately one-third to in excess of 80% that of untreated surfaces with the greatest benefit to control of dust from the MC-800 and cationic emulsified asphalts. Addition of an armor or surface seal coat to the stabilized base contributed to a near zero dust concentration, with an obvious increase in cost. Contracted costs of observed lignosulfonate treated base plus seal coat roads in Cass and Wright Counties were approximately $15,000 per mile.

Aggregate pullout, or loose surface aggregate, was reduced from approximately 25% to in excess of 75% that of the untreated soil aggregate test road surfaces, with the greatest benefits being provided by the MC-800 and cationic emulsified asphalts, and high aggregate content lignosulfonate test sections.

DUST CONTROL/SURFACE IMPROVEMENT SOLUTIONS

Historically, engineers have utilized various methods and techniques of dust control. Surface applications of calcium and sodium chlorides assist in laying dust for short periods of time but are easily soluble in rain water and melting snow, occasionally leaching into an open soil/ aggregate surface, but generally eroding into the ditches.

Surface oiling provides longer periods of dust palliation coupled with some bonding action to hold aggregate in place, but ultimately ravels or develops potholes, requiring maintenance.

Over the years a number of Iowa counties have tried, with varying degrees of success, cutback asphalts mixed into the road surfacing to a depth of 2-3 in. Results of this type of surface improvement significantly reduces surface dusting and aggregate loss over a period of about one year, sometimes more, but thereafter involves patching and definite maintenance. Placement of a seal coat over this type of surface improvement adds to dust palliation and improved life, while reducing maintenance costs for an additional period of time. Ultimately however, maintenance is required due to water, softening the aggregated clays in the surface matrix (Fig. 36, Part II Final Report). This "clay ball," at one point of construction manipulation, has been completely coated with the cutback asphalt, but upon final blading is partially sliced, with no asphalt coating on the exposed surface. With water, these balls are liquified, eroded, or knocked from the surface by traffic, and eventually are the source of potholing and ravelling. Pretreatment with lime would reduce this aggregating effect but would also increase cost of materials with an additional construction spreading and mixing operation.

From the above discussion it may be ascertained that the most beneficial dust palliation and surface improvement techniques to date have generally involved the incorporation of an additive(s) to a limited depth within a soil/aggregate road surface. This was a primary concept used while conducting the research project reported herein.

Table 1 presents a brief summation of a portion of the laboratory and field tests, coupled with recommendations for actual field usage including soil types and additive contents to be considered. For purposes of this report summation it was felt that the reader could better analyze and potentially utilize a table, than read additional wordy, though hopefully brief, discussion. With this type of a presentation, <u>it must be fully assumed that the reader will also study each appropriate portion of the Final Report prior to actually designing a surface improvement section</u>.

Comparison of laboratory and field test results in this project, in general, indicate the greatest degree of dust palliation, surface improvement qualities, and stability effectiveness, utilizing the trafficability testing with a realistic observation of freeze-thaw characteristics. Increased strength appears to be of little benefit.

Of the additives recommended for usage in Table I-1, each generally meets the six criteria previously presented. It will be noted that some additives are definitely recommended, some are recommended with reservations. The latter, in general, are due to somewhat lesser degrees of long-term potential as dust palliatives and surface improvement agents as based on laboratory tests, and field tests or observations. If coupled with surface seal coats however, these products appear to have excellent low-cost base course qualities.

Soil additives	Soil or aggregate used in the study	AASHO classification	Range of additive content studied, %	Effective additive content(s), %	Effect on density as compared to untreated soil	Effectiveness during (a) immersion (b) capillary saturation (c) erosion	Trafficability effectiveness in providing waterproofing, fines retention, and stability	Freeze-thaw effectiveness	Recommendation for project field trials	Soil classification and range of additive content recommended for project field trials
A. Asphaltic products and elastomers										
1. Peneprime[a]	Gravelly sandy loam	A-1-b	0-6	2-> 4	Slight increase	Low (a)	Very low	N/D[b]	Yes – with reservations	A-1 to A-2, 3-5%
2. Semi-pave[a]	Gravelly sandy loam	A-1-b	0-6	None	Slight increase	None (a)	N/D[b]	N/D[b]	No	–
3. MC-800 cutback Asphalt	Gravelly sandy loam	A-1-b	0-6	1-6	Slight increase to slight decrease	Good (a)	Good	Good	Yes	A-1 to A-4, 3-5% residual
	–	A-1-b to A-4	0-4	3-4	Slight decrease	Good (a)	Good	Good	Yes	
4. Redicote E-36[a] Cationic asphalt Emulsion	Silty loam	A-4(8)	0-8	3-6	Slight decrease	N/D[b]	N/D	Good	Yes	A-1 to A-4, 3-5% residual
	Gravelly sandy loam	A-1-b	0-6	2-5	Slight decrease	Good (a)	Good	Good	Yes	
	–	A-1-b to A-4	0-4	3-4	Slight decrease	Good (a)	Good	Good	Yes	
5. Petroset SB[a] Cationic latex Emulsion	Gravelly sandy loam	A-1-b	0-6	2-6	Slight decrease	Excellent (a)	Good	N/D	No – due to cost	–
6. Petroset RB[a] Cationic latex Emulsion	Gravelly sandy loam	A-1-b	0-6	2-6	Slight decrease	Good (a)	N/D	N/D	No – due to cost	–
B. Lignosulfonates and secondary additives										
1. Lignin	Gravelly loam	A-2-6(0)	0-2	1-2	None to slight increase	Very poor (b)	N/D	Good	Yes – with reservations	A-2, 1-2% solids
	Clay loam to silty clay loam	N/D[b]	0-2	1-2	None to slight increase	Very poor (b)	N/D	Good	Yes – with reservations	A-4, 1-2% solids
	Gravelly sandy loam	A-1-b	0-2	1-2	None to slight increase	Very poor (a)	N/D	Good	Yes	A-1-b, 1-2% solids
	Sandy loam	N/D	0-2	1/2-2	Slight increase	Very poor (a)	N/D	Fair to good	Yes – with reservations	A-2 to A-4, 1-1/2% solids
2. Lignin + lime	Glacial till	N/D	1.5 to 3 + 0 to 2	1.5 to 2.5 + 0 to 1	None to decrease	Very poor (a)	N/D	Good	Yes	A-2 to A-6, 1 to 2 + 1/2 to 1%
	Silty clay alluvium	N/D	0 to 2 + 0 to 1/2	1.5 to 2 + 0 to 1/2	Slight decrease to slight increase	Very poor (a)	N/D	Good	Yes – with reservations	A-4 to A-6, 2 + 1/2 to 1%
	Sandy to silty loam	N/D	0.8 to 2.0 + 0 to 1	1 to 2 + 0 to 1	Increase to decrease	Very poor (a)	N/D	Fair to good	Yes – with reservations	A-2 to A-4, 1 to 2 + 1/2%
	Clay loam to silty clay loam	N/D	0 to 1 + 1/2	1 + 1/2	None	Very poor (b)	N/D	Good	Yes – with reservations	A-4, 1 + 1/2%
	Gravelly sandy loam	A-1-b	1 + 1/2	1 + 1/2	None	Very poor (b)	N/D	Good	Yes – with reservations	A-1-b, 1 + 1/2%
3. Lignin + alum	Silty clay loam to clay loam	N/D	0 to 1 + 1/2	1 + 1/2	None	Very poor (a)	N/D	Good	No	–
	Gravelly sandy loam	A-1-b	1 + 1/2	1 + 1/2	None	Very poor (a)	N/D	Good	Yes – with reservations	A-1-b, 1 + 1/2%
4. Lignin + sodium silicate	Gravelly loam	A-2-6(0)	1 + 0 to 1	1 + 0	Slight decrease	Poor (b)	N/D	N/D	See additive B.1.	–
			2 + 0 to 2	2 + 1-1/2 to 2	Slight decrease	Poor (b)	N/D	N/D	Doubtful	–
5. Lignin + polymer JB[a]	Gravelly loam	A-2-6(0)	1 + 0 to 1-1/2	1 + 0	None to decrease	Poor (b)	N/D	N/D	See additive B.1.	–
			2 + 0 to 1-1/2	2 + 1/2 to 1	None to decrease	Good (b)	N/D	N/D	No – due to cost	–
C. Soil-chemical additives[c]										
1. Sodium silicate	Gravelly loam	A-2-6(0)	0-1-1/2	1	Slight decrease	Poor (b)	N/D	N/D	No	–
2. Polymer JB[a]	Gravelly loam	A-2-6(0)	0-2	None	Increase to decrease	Poor (b)	N/D	N/D	No	–
3. Residual[d]	Glacial till	N/D	0-5	4	Decrease	Fair (b)	N/D	Good	Yes	A-1-b to A-4, 3-5%
4. Petro D Dust[a]	Sandy loam	A-2-4(0)	0-2	0.1-0.25	Decrease	Excellent (c)	Good	Good	Yes	A-2 to A-4, 0.1-0.25%
	Sandy loam	A-2-4(0)	Seal coat – 0.2 to 0.25 gal/sq yd	0.2-0.25 gal/sq yd	None	Fair to poor (c)	Fair	Poor to fair	Yes – but coupled w/mixed in place	– , 0.2-0.25 gal/sq yd
5. SA-1[a]	Sandy loam	A-2-4(0)	0-0.2	0.1	None	Good (c)	N/D	N/D	With further testing – yes	A-2 to A-4, 0.1-0.2%
6. Kalpak[a]	Sandy loam	A-2-4(0)	0-0.2	0.1-0.2	Slight decrease to increase	Excellent (c)	N/D	N/D	With further testing – yes	A-2 to A-4, 0.1-0.2%
7. Clapak[a]	Sandy loam	A-2-4(0)	0-0.2	0.1	None	Fair (c)	N/D	N/D	Doubtful even with further testing	–
8. Claset[a]	Sandy loam	A-2-4(0)	0-0.2	0.1-0.2	None	Very poor (c)	N/D	N/D	No	–
9. Elvanol 71-30[a]	Sandy loam	A-2-4(0)	0-1	0.1-0.5	Decrease	Excellent (c)	Poor	Good	No – difficulty in mixing	–
10. Elvanol 52-22[a]	Sandy loam	A-2-4(0)	0-1	0.1-0.5	Decrease	Excellent (c)	None	None	No	–
11. Stypol 40-5020[a]	Sandy loam	A-2-4(0)	0-1	0.5	Decrease	Good to excellent (c)	Fair to good	Fair	Yes	A-2 to A-4, 0.5%

Table I-1. Footnotes.

[a] Commercial, proprietary, or trade name.

[b] N/D. Not determined.

[c] In addition to the soil-chemicals noted, a grouping of chemicals were examined but initially rejected. See Table 3 and the appropriate discussion, Part IV Final Report.

[d] A residual waste product from the Chemplex Plastics plant, Clinton, Iowa.

Recommended field usage of certain soil-chemical additives (C.4 through C.11, Table 1) are based only on laboratory testing, although criteria and comparison of laboratory test results with products recommended under items A and B indicate their readiness for field trials. Items C.5 and C.6 (i.e. SA-1 and Kelpak) need further trafficability and freeze-thaw analyses, though other laboratory test data indicate an excellent potential for field trials.

PART II

ASPHALTIC PRODUCTS AND ELASTOMERS AS DUST PALLIATIVES AND SURFACE IMPROVEMENT AGENTS FOR UNPAVED SECONDARY ROADS

by

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and

J. M. Hoover

CONTENTS

INTRODUCTION

Bituminous materials have long been used, with varying degrees of success, as soil stabilizers and waterproofing agents. With advancing technological developments many new asphaltic products, whose properties have been altered and supposedly enhanced by the addition of chemicals and selected emulsifying agents, are being marketed. Elastomers, which contain rubber in a modified form, have also been introduced. The beneficial effects of these products, as soil stabilizers and dust palliatives, can only be determined by laboratory evaluation and field trials.

OBJECTIVE

This investigation was conducted essentially as a three phase project consisting of (a) laboratory screenings of various asphaltic products and elastomers to evaluate their effectiveness as soil stabilizers and dust palliatives, (b) construction of a test road, based on the results of the laboratory screening phase, using those additives that appeared to be the most effective and economical, and (c) observation and tests of the various sections of the test road for evaluation of the additives performance and serviceability with respect to dust palliation and surface improvement.

The primary purpose of this study is to present the results of each of the above phases. The test road was constructed in September 1971 and had been in service only one year and three months at the time of termination of research. Therefore, phase 3 analyses are somewhat limited due to the period of time covered.

REVIEW OF LITERATURE

Bituminous materials have long been used with varying degrees of success, as soil stabilizers and waterproofing agents. With advancing technological developments many new asphaltic products, whose properties have been altered and supposedly enhanced by the addition of chemicals and selected emulsifying agents, are being marketed. Elastomers, which contain rubber in a modified form, have also recently been introduced. The various additives used in this study were primarily of three classes: (a) cutback asphalts, (b) cationic asphalt emulsions, and (c) cationic latex emulsions.

The bulk of the asphalt presently being used comes from the petroleum refining process and is a highly complex material composed primarily of various hydrocarbon compounds which at present are not completely defined or well understood[2-4]. Asphalt cement is asphalt that has been refined to meet a variety of particular specifications depending upon its intended use and the using agency. For highway construction many of these specifications are set by the Asphalt Institute, ASTM and AASHO standard testing methods[3,5]. Asphalt cement is semi-solid to solid in consistency, must be heated to high temperatures for use, and in order to be used with unheated aggregates, must be put into a liquid form. For highway work this is generally in the form of cutback asphalt or emulsified asphalt, which allows usage at normal or slightly elevated temperatures.

Cutback asphalts are formed by adding various amounts and types of solvents to asphalt cement, the type of solvent determining the type of cutback produced. Highly volatile solvents (gasoline or naptha) produce rapid curing (RC) cutbacks. Kerosene and light volatile oils produce medium curing (MC) and slow curing (SC) cutbacks, respectively. The process of using a solvent with asphalt cement provides the following advantages with regard to soil-aggregate mixtures: (a) use at normal or slightly elevated temperatures, (b) facilitates dispersion of the asphalt cement, (c) aids in ease of mixing and, (d) aids in compaction. Water is needed in soil-aggregate mixtures to aid in dispersion of the cutback and compaction[6-8] with curing accomplished by evaporation of the solvent and water. Rapid curing cutbacks are normally used for sandy soils containing a minimum amount of silt and clay. Medium curing and slow curing cutbacks are normally used for soils containing a considerable amount of silt and clay, primarily because they will incorporate better, and provide a more homogeneous mixture than an RC cutback[4].

Asphalt emulsions are relatively new in the highway construction field in the U.S. with anionic emulsions first being introduced in 1930 and cationic emulsions in 1958[9]. Emulsion formation and properties are much different than those for the widely used cutbacks and therefore will be discussed in more detail.

Emulsions contain asphalt dispersed as small droplets in a water medium. Droplet sizes[10] range from 1-5 microns in diameter. In the preparation of emulsions, dispersion of the asphalt is usually accomplished by a colloid mill or other mechanical means which shears the asphalt into small droplets, as water is simultaneously introduced. The system of water and dispersed asphalt produced is thermodynamically unstable[10-12] and the asphalt will coalesce unless an emulsifying agent is present. The emulsifying agent must be compatible with both asphalt and water. Water has polar molecules and asphalt has nonpolar molecules thus requiring the emulsifier molecules to be of the mixed polar and nonpolar type. Physical and chemical properties of the emulsion are, therefore, largely dependent on the chemical type and molecular structure of the emulsifying agent. There is no single theory of emulsion formation and stability, and what has been found true for one emulsion system is not necessarily applicable to other systems. Becher[10] and Sumner[11] present a review of emulsion theories and a technical treatment of the physical-chemical properties of emulsions.

There are three chemical types of emulsifying agents: anionic, cationic and nonionic. Each is distinguishable by its properties upon ionization but all share one common property of being adsorbed at an interface between a liquid and air, a solid, or another liquid.

The polar organic portion of an ionized emulsifying agent's molecule governs its properties[9,10,13]. The organic polar portion of the molecule is hydrophillic (water loving) and is soluble in water. The hydrocarbon nonpolar portion of the molecule is hydrophobic (water hating) and nonsoluble in water but soluble in asphalt. Therefore if the emulsifying agent is present in sufficient quantity, when the emulsion system of asphalt is dispersed as droplets in water, the emulsifier molecules will preferentially attach themselves to the droplet interface with the hydrocarbon nonpolar portion solubilized by the asphalt and the organic polar portion solubilized by the water. This creates a protective charged film of emulsifier molecules around each droplet of asphalt. The charge of the emulsifier film is dependent on the charge carried by the polar organic portion of the emulsifier molecule. If positive, the emulsion is termed cationic, if negative, anionic. This charged film repels other droplets, also charged, and gives rise to the stability of the emulsion. Without the charged film the emulsion is unstable and would coalesce into two distinct phases of asphalt and water.

There are numerous emulsifying agents producing both cationic and anionic emulsions. In making cationic emulsions, a suitable acid[9,13] is added to the water phase. A base is added to the water phase for anionic emulsions[9,14]. This makes a cationic emulsion positively charged and acidic, while anionic emulsions are negatively charged and basic.

In contrast to cutbacks, the type of emulsifier determines the type of emulsion produced as well as the speed with which the emulsion breaks on contact with mineral surfaces. Three classes are generally available for highway purposes, rapid-setting (RS), medium-setting (MS) and slow-setting (SS).

The process of emulsification provides the same advantages as listed for the cutbacks with regard to soil-aggregate mixtures. Two primary advantages are obtained in using asphalt emulsions, one being that they may be applied to very wet soils much easier than cutbacks, for reasons to be mentioned later. The second advantage is that curing is accomplished through the evaporation of water from the emulsion mixtures, as opposed to solvent evaporation from the cutback mixtures. With the current concern over air pollution the use of cutbacks in any type of construction could be severely limited in the future.

Structure and composition of aggregates is one of the most important factors[9,13-16] that affect adhesion in an aggregate-asphalt system. Conventionally aggregates also are defined as either hydrophobic or hydrophillic. Silica, quartz, and gravel are generally accepted as examples of hydrophillic aggregates and limestone is regarded as hydrophobic. Mertens and Wright[9] point out that the terminology is incorrect, for the term "hydrophobic" aggregate implies that it cannot be readily wetted with water. Actually all limestones and similar aggregates are as readily wetted by water as "hydrophillic" aggregates. They suggest that these two aggregate types be described as "electronegative" and "electropositive" rather than "hydrophillic" and "hydrophobic," respectively. In this terminology the surface properties of aggregates range from extremely electropositive, to electronegative[9].

If the aggregates are dry, the electrical charges are practically nonexistent because adsorbed ions neutralize the surface charge. In the presence of water, these adsorbed ions disassociate from the surface, being dissolved in water, and the aggregate surface bears an electrical charge. Silaceous aggregate surfaces become negatively charged in the presence of water and are termed electronegative. Limestone and other calcareous material surfaces bear a positive charge in the presence of water[9,13] and are termed electropositive. Some investigators are of the opinion that, with few exceptions, calcareous materials are also electronegative[17]. Mertens and Wright[9] suggest that the surface charges are the result of fracturing and hydration. Each aggregate particle may possess a great many unsatisfied surface charges resulting from broken electrostatic chemical bonds[9,15]. In many aggregates both positive and negative charges may exist. The bulk of aggregates are of the mixed to electronegative charge type, with limestone being electropositive.

The above discussion on surface charge of aggregates has little affect with respect to cutback asphalts. Asphalt, as well as the hydrocarbon solvents, are nonpolar in nature and consequently have no affinity for the aggregate surface due to its charge. Emulsions, however, because of the charged nature of the asphalt droplets are affected considerably. For a given aggregate-emulsion system the degree of adhesion and the coating properties are, theoretically, largely determined by the charge on the emulsified droplet relative to the surface charge of the aggregate. If the charges are different, good coating and adhesion are obtained. The difference in surface charge between the emulsified droplet of asphalt and the aggregate surface promotes adhesion by functioning as a bonding agent. The charged organic portion of the emulsifier molecule actually displaces water on the aggregate surface to satisfy the charge deficiency while the asphalt is still held by the inorganic portion of the emulsifier molecule. This results in the asphalt being bonded to the surface of the aggregate preferentially and the emulsifier molecule functioning as a bonding agent and built in anti-stripping agent.

Of the many early theories regarding the mechanism of bituminous stabilization the "intimate mix" and "plug" theories of Endersby[18] appeared to have gained the widest recognition. These theories are general in nature with the "intimate mix" theory implying that nearly all particles are coated with asphalt and stuck together. This is probably applicable to noncohesive soils. The "plug" theory applies, in part, to cohesive soils in which aggregates of soil particles are coated with asphalt which acts as a waterproofer by plugging the soil voids. In general then, the mechanism by which bituminous materials stabilize soil is primarily physical in character by imparting cohesion to the treated soil mass (most important for noncohesive soils) and by waterproofing the soil mass (most important for cohesive soils).

Michaels and Puzinauskas[7,19] expanded these general theories to cutback asphalt and asphalt emulsion stabilization of fine-grained soils specifically, indicating that the effectiveness of asphalt as a soil stabilizer is limited by certain physical and physiochemical properties of the asphalt and the soil to which it is applied. They believe that the three most important factors are (a) the virtual impossibility of distributing asphalt uniformly through a fine-textured granular solid such as soil, (b) the inability of asphalt to adhere to (and thus coat) wet soil particles, and (c) the sensitivity of the asphalt-soil bond (when developed) to destruction by water. The first factor arises from the high viscosity and water-immiscibility of asphalt (both of these properties being necessary for stabilization); the second and third arise from the typical hydrophillic character of most soil minerals, and the strongly hydrophobic properties of asphalt. They contend that with gravel and sands, these factors are of relatively minor importance because (a) the surface area (per unit mass) to be asphalt-coated is small (and thus uniformity of distribution of asphalt is not critical), (b) the quantity of water needed or present to permit proper handling of aggregate is rather small, and can be easily removed by evaporation, and (c) dependence of the properties of the aggregate on moisture content is relatively low. As one proceeds to soils of finer particle size, these factors become of rapidly increasing significance until, with plastic soils containing large amounts of colloidal minerals, incorporation of asphalt is virtually ineffective as a means of stabilization or waterproofing.

Michaels and Puzinauskas[7,19] propose the following mechanisms for stabilization of fine-grained soils.

Mechanism of Cutback Stabilization

Stabilization of soils with cutback asphalt is normally carried out by (a) blending the soil (at or near optimum water content for compaction) with the cutback, (b) compacting the mixture, and (c) curing the compacted mixture by allowing water and cutback solvents to evaporate over a period of time. During mixing, the nonwetting cutback phase is dispersed as globular or filamentous masses between soil particles or soil aggregates. The degree of subdivision of the cutback and breakdown of soil-aggregates, is apparently dependent on the duration of the mixing process. Upon compaction, the free void space is reduced, part of the volatile fluids (water and cutback solvent) are lost by evaporation, and the dispersed cutback particles are squeezed out into filaments and laminae which occupy a rather large fraction of the void space. Because of capillary forces, the asphalt phase will be confined to the larger pores and/or channels, while water will occupy the fine textured porosity of the mass. The resulting compacted mixture may then be considered as a close packed mass of water wet particles, and soil aggregates surrounded by a partially continuous network of asphalt. As water evaporates from the mixture, it becomes possible for the asphalt cutback to wet out and adhere to a larger and larger fraction of the soil particles and soil aggregate surfaces. As the wetting-out process proceeds, the asphalt would be expected to distribute itself more and more uniformly through the soil, provided it possesses sufficient fluidity to do so. This fluidity depends primarily upon the rate of evaporation of the cutback solvent.

If the soil itself is of low cohesiveness, the adhesion of the asphalt to the soil on evaporation of water will result in the development of cohesion of the mass. If, on the other hand, the soil is a highly cohesive one, then the presence of asphalt in the voids will interfere with the formation, on drying, of soil-soil bonds which are far higher in strength than soil-asphalt bonds. Hence a reduction in strength may occur.

After cutback solvent evaporation has occurred in a fine-grained soil, the asphalt itself occupies a relatively small fraction (typically about one third) of the void space. In order to develop significant water-resistance, the asphalt must be distributed in a fashion that will block, to the greatest possible degree, water uptake by the soil. This can be most satisfactorily accomplished if the asphalt is consumed in surrounding (i.e. protective sheaths), relatively large agglomerates of (essentially asphalt-free) soil particles. This condition would be much better than that which would be obtained if the asphalt were perfectly distributed, wherein one of every three pore spaces in the soil were filled with asphalt.

Mechanism of Emulsion Stabilization

Stabilization of soils with asphalt emulsions (assumed nonionic) is typically performed by (a) blending the emulsion with wet soil (with the total water content at or near optimum), (b) compacting, and (c) curing as with cutbacks. During the mixing process (in contrast to treatment with cutbacks), since the asphalt is initially dispersed as microscopic droplets in water, uniform distribution of the asphalt through the soil should be far simpler to accomplish, provided the asphalt particles retain their identity until mixing is complete. If coagulation or agglomeration of the emulsified particles takes place on mixing, this will result in the formation of globs of asphalt which will be virtually impossible to break down and redistribute. The efficiency of distribution therefore depends on the stability of the emulsion in contact with the soil, which in turn is determined by the type and amount of emulsifying agent used. Hence asphalt emulsions that "break" easily on agitation, change in temperature, or contact with electrolytes present in the soil water are likely to yield very inhomogeneous mixtures with the soil.

Assuming that the emulsion remains stable during mixing, the compacted mixture can be envisioned as an assemblage of soil particles and/or soil aggregates with void space partially filled by small globules of asphalt. During curing, water evaporates, exposing dry soil surfaces to which asphalt particles can adhere. However, since there is no asphalt solvent present, there can only be very slow redistribution of asphalt by wet-out and flow (as postulated for cutbacks). The formation of continuous films of asphalt around the soil particles takes place very slowly, if at all. The final cured product might be imagined as a mass of soil particles whose surfaces are "peppered" with small droplets of asphalt. If, however, the asphalt emulsion is cationic or anionic in nature, or has been stabilized with an anti-stripping agent as an emulsifier, the effect would be to render the soil surfaces oil-wettable. This would, theoretically, cause the asphalt droplets to promptly deposit on and adhere to the soil particles and soil aggregate surfaces permitting the development of cohesive strength before curing. This again would be dependent on the emulsion remaining stable during mixing.

The foregoing theories appear to adequately explain the mechanisms of bituminous stabilization of fine-grained soils using cutback asphalt and asphalt emulsion.

Borgfeldt and Ferm[20] (in field and laboratory testing of a cationic emulsion, anionic emulsion and cutback asphalt field mixes) concluded that cationic emulsions were superior in their ability to coat and adhere to a wide variety of dry or moist aggregates, resistance to mechanical stripping action and moisture, as well as rapid setting tendencies.

Comparisons of cationic versus anionic asphalt emulsions have indicated that cationic emulsions are superior to anionic emulsions in ability to adhere to aggregate surfaces in either a wet or dry condition. Mertens and Borgfeldt[13] present several theories for this behavior.

Cationic emulsions, however, were only introduced into the United States in 1958 and although usage is increasing, very little published information is available regarding field performance or laboratory evaluation.

Bituminous soil stabilization, although used extensively for many years, is still an art more than a science. This is evidenced by the large variety of methods of mix design, testing, design criteria and guidelines presently in use by state highway departments, Asphalt Institute, governmental agencies and various branches of the armed forces[21]. Some methods roughly agree while others vary widely, primarily because of the complex number of variables generated from the various combinations of soils, bituminous materials, mixing conditions, and curing and compaction methods.

Elastomers are elastic or rubberlike substances similar to natural or synthetic rubber. The elastomers investigated in this study are very new, and little, if any, published information (other than manufacturer literature) is available, especially with respect to incorporation into the soil. They are recommended for use as a surface treatment for dust palliation. They do, however, possess high tensile strength as opposed to asphalt, and considering that they are available as a cationic emulsion they may also be preferentially adsorbed to a negatively charged surface. If this is true, they may be capable of developing considerable additional strength, as compared to asphalt, and may function to provide not only waterproofing but considerable cohesion and elasticity due to their tensile strength.

LABORATORY EVALUATION

Many past studies investigating various additives as stabilizing agents used unconfined compressive strength as a means of evaluation. Specimens were tested under various conditions of curing, molding, moisture content, saturation and immersion to name a few. The unconfined compression test has been used to evaluate large numbers of additives primarily because it is fast, simple and economical. Results, however, can be used for comparison only since they are primarily an indicator of strength and may or may not be an indicator of total stability; stability being a function of many variables of which strength is only one. It was desired in this study to find some means of obtaining a valid indication of stability, as well as determine if unconfined compression results could be used as indicators of total stability.

Initial evaluation was conducted using unconfined compression tests at various curing conditions. Additives that appeared most promising were then tested using a traffic simulator apparatus that was capable of imposing maximum expected field loading and adverse environmental conditions. Results of each method were then compared for final evaluation.

Materials

Soils

The major portion of this study was conducted using a moderately hard limestone designated as Bedford limestone, obtained from near Bedford, Iowa in Taylor County. This limestone is of the Pennsylvanian system and outcrops in nearly half the state. Formations in this system are generally quite soft and contain relatively high amounts of clay. Performance records of Bedford limestone as a base material has not been as satisfactory as other Iowa limestones[22].

Secondary roads handling higher volumes of traffic are generally surfaced with crushed limestone material rather than gravel. Due, primarily, to its angularity and amount of filler material, crushed limestone compacts to form a relatively hard crust as opposed to gravel which is rounded and generally low on filler material. It can be assumed that if counties begin a program of surface stabilization and dust palliation, the list of priorities will be based on a combination of traffic volume as well as the number of residences that the road serves. More often than not this would be a crushed-limestone-surfaced road. For these reasons it was felt that the use of the Bedford limestone, being of rather poor quality and fairly abundant would be an appropriate material to use. Results could then be compared with test road soil-aggregate materials using the same additives.

A very limited study was also conducted using a loess material, designated as 20-2, obtained from Harrison County, Iowa. Loess is an abundant surficial deposit found extensively in southern and southwestern Iowa, and it is highly likely that many secondary roads have subgrades composed partially of loess materials. Considerable previous work has been done with loess and cutback asphalts[6]. It was desired, however, to evaluate the effect of the Redicote E-36 cationic emulsion on a fine-grained material. There was insufficient 20-2 loess to use with the latex emulsions.

Engineering properties of the Bedford limestone and 20-2 loess are given in Table II-1.

Table II-1. Properties of soils.

	Loess (20-2)	Bedford limestone
Physical properties		
Liquid limit (%)	30.8	20.0
Plastic limit (%)	24.6	18.0
Plasticity index (%)	6.2	2.0
C.M.E. (%)	19.6	n.d.
Shrinkage limit	22.3	n.d.
Specific gravity	2.71	2.73
Chemical properties		
Cation exchange capacity	13.4	10.88
pH	8.7	9.4
Textural composition		
Gravel	0.0	73.2
Sand	0.4	12.9
Silt	79.8	8.4
Clay	19.8	5.5
Colloidal clay	14.5	1.7
Textural classification	Silty loam	Gravelly sandy loam
Engineering classification	A-4(8)	A-1-b

Additives

The various additives used in this study were primarily of three classes: (a) cutback asphalts, (b) cationic asphalt emulsions, and (c) cationic latex emulsions. The additives are as follows:

1. Redicote E-36 cationic asphalt emulsion
2. MC-800 cutback asphalt
3. Peneprime, a specially processed cutback
4. Semi-Pave, a specially processed cutback
5. Petroset SB cationic latex emulsion
6. Petroset RB cationic latex emulsion.

Nearly all are relatively new products with the exception of the MC-800 cutback asphalt which has been used extensively in road construction and is presently being used as a shallow-depth (less than 4 in.) stabilizer and dust palliative for unpaved secondary roads in Pottawattamie and other counties with reasonably encouraging results. It was desired to use it in this study for comparison with the newer products.

Unconfined Compression Tests

Specimen Preparation and Curing

All test specimens used in this portion of the study were 2 in. in diameter by approximately 2 in. in height. Molding was accomplished using the drop hammer technique previously reported by Chu and Davidson[23]. This method yields densities comparable to standard Proctor densities.

For the Bedford limestone and 20-2 loess, material passing the 3/8 in. and #10 sieve respectively, was used to establish optimum moisture-maximum density, given in Table II-2.

For the Bedford limestone, specimens were molded maintaining the total water content at 9% for the emulsion treated specimens (i.e. added water plus water in emulsion equals 9%). For the cutbacks, added

Table II-2. Optimum moisture-maximum density relationships using 2 in. by 2 in. specimens and drop hammer technique.

Material	Optimum moisture (%)	Dry density (pcf)
Bedford limestone	11.5	126.1
Loess	15.4	113.6

water was maintained at 9%. It should be noted that 9% is not optimum moisture as given in Table II-2. Specimens molded at optimum moisture (11.5%) were very wet, difficult to handle and deformed with a convex top on extrusion. Trial specimens were molded at different moisture contents less than optimum, and 9% selected on this basis. The 20-2 loess specimens, treated with emulsion, were molded maintaining the total of emulsion plus water at 16%.

All additive percentages used and referred to herein are on the basis of total amount of additive, irrespective of the proportions of solids or liquids.

Peneprime, Semi-Pave and Redicote cationic emulsion were heated to 150 °F prior to sample preparation. Redicote cationic emulsion was also used in an unheated condition, where noted. MC-800 cutback asphalt was heated to 275 °F prior to specimen preparation. Petroset SB and Petroset RB latex emulsions were used in an unheated condition.

Specimen preparation was accomplished in the following manner. A 2000-gram air dry-soil sample was placed in a Kitchen-Aid mixer and a predetermined amount of distilled water was slowly added as the mixer operated at low speed. The following process was then used: machine mixed 1 min, hand mixed 1 min, machine mixed 1 min and again hand mixed. Upon completion of this process an appropriate amount of additive was then slowly incorporated, as the mixer operated at low speed. Upon addition of the additive, the sample was machine mixed 1 min, hand mixed 1 min, machine mixed 1 min, again hand mixed and then placed in a humid atmosphere for a minimum of 5 min prior to molding. Specimens were then molded with height, weight and diameter measurements being determined.

Curing of all specimens was accomplished at room temperature upon perforated racks. At least two specimens were tested at each of the following conditions:

1. Immediately upon molding
2. Air cured 24 hours
3. Air cured 24 hours then immersed in water 24 hours
4. Air cured 72 hours.

The purpose of immersion of specimens, after 24 hours of air curing, was to evaluate the waterproofing characteristics of each additive as quickly as practically possible following compaction.

Method of Testing

All testing in this portion of the study was accomplished using a Soiltest AP-170 unconfined testing unit, delivering load through a calibrated proving ring. This unit was equipped with a motor which provided a uniform and constant deflection rate of 0.1 in./min. A ball and socket arrangement was used between the proving ring and specimen to minimize nonparallel end effects. Weight, height, and diameter measurements were taken prior to testing.

Results

Results of the combination of Redicote emulsion and loess are presented in Fig. II-1. Unconfined compressive strength increased slightly with the addition of Redicote up to approximately 1-2% emulsion content, for all cure times and is probably indicative of increased cohesion. Strength decreased with increasing amount of Redicote past the 1-2% emulsion content range. This decrease became much more pronounced for the specimens air cured 72 hours, and was probably the result of (a) asphalt films and "plugs" of asphalt decreasing frictional resistance through lubrication and (b) asphalt replacing, interfering with, and reducing the number of high strength bonds created by evaporating water. An additional factor probably contributing to the strength decrease is that dry density decreased with increasing additive content as noted in Table II-3.

Results of the immersion tests were qualitative only. All specimens failed in some manner and were too soft to test. Untreated specimens slaked immediately. In the approximate range of 1-4% additive content, specimens retained their form, in varying degrees, but were too soft to test. Above 5% emulsion content, specimens again slaked, but more slowly, to a very poor form or an indistinguishable pile. It was initially assumed that as emulsion content increased, the waterproofing characteristics would increase. This did not appear to be the case for this particular cationic emulsion and loess. It was noted on the compacted loess specimens that there was very little observable evidence of the presence of asphalt up to about 6% emulsion content. Above about 6% emulsion small globules of asphalt were visible on the sides of specimens,

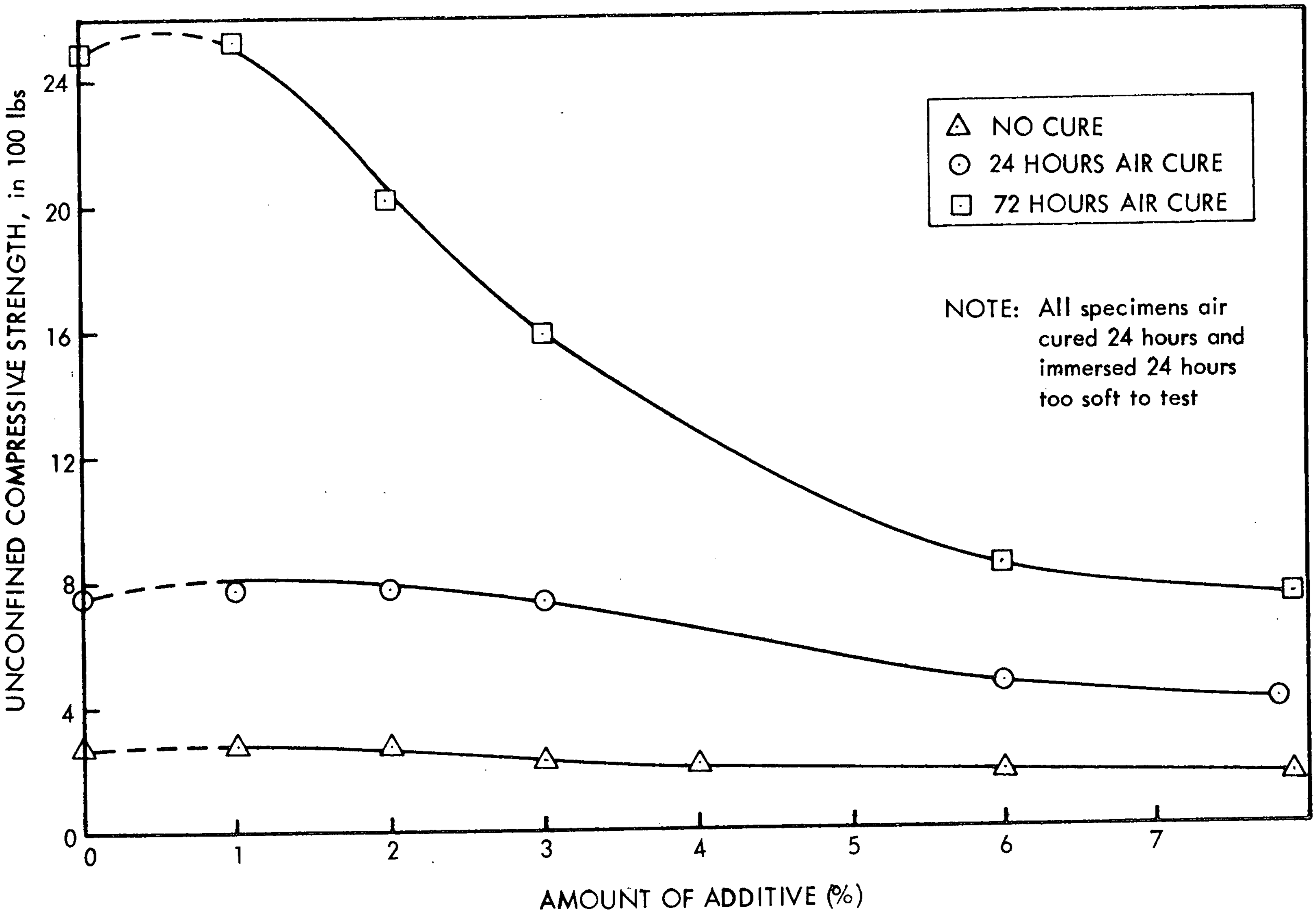


Fig. II-1. Unconfined compression test results, Redicote emulsion and 20-2 loess.

Table II-3. Density variations of unconfined compression specimens Bedford limestone and 20-2 loess material.

Material	Additive	Additive amount (%)	Approx.[a] liquid cont. (%)	Wet density (pcf)	Dry[b] density (pcf)
Bedford limestone	Untreated	9.0	9.0	130.7	119.9
	Redicote (unheated)	0.5	9.0	127.7	117.1
		1.0	9.0	125.8	115.2
		2.0	9.0	123.4	113.2
		4.0	9.0	120.2	110.3
		6.0	9.0	117.9	108.2
	Redicote	1.0	9.0	128.8	118.1
		2.1	9.1	124.8	114.4
		4.0	9.0	122.9	112.8
		6.6	9.2	122.9	112.8
		6.6	9.2	119.8	109.7
	MC-800	1.0	9.3	132.2	121.2
		2.0	9.5	133.9	122.6
		3.0	9.7	134.8	123.5
		3.7	9.9	135.6	124.1
		6.2	10.6	133.2	122.2
	Peneprime	1.0	9.5	135.5	124.1
		2.0	10.0	137.3	126.1
		3.0	10.6	138.6	127.1
		3.9	11.0	138.0	126.9
	Semi-Pave	1.0	9.5	138.1	126.6
		2.0	10.2	140.7	126.2
		4.0	11.4	137.3	124.1
	Petroset RB	0.5	9.0	131.8	120.9
		1.0	9.0	128.7	118.0
		2.0	9.0	126.0	115.0
		4.0	9.0	123.3	113.0
		6.0	9.0	130.5	119.7
	Petroset SB	0.5	9.0	129.0	118.7
		1.0	9.0	128.4	117.8
		2.0	9.0	125.0	114.7
		3.0	9.1	123.3	113.0
		4.0	9.1	123.9	113.6
		6.0	9.1	120.9	110.8

Table II-3. Continued.

Material	Additive	Additive amount (%)	Approx.[a] liquid cont. (%)	Wet density (pcf)	Dry[b] density (pcf)
Loess (20-2)	Untreated	16.0	16.0	131.0	113.0
	Redicote	1.0	15.4	129.5	112.5
		2.0	14.7	126.8	110.5
		3.0	14.1	126.8	111.2
		4.0	13.4	126.1	112.0
		6.0	12.1	120.6	107.6
		8.0	10.8	116.4	105.8

[a] Dry density and percent liquids based on the following: MC-800 @ 25% volatiles; Redicote E-36 @ 35% liquids; Petroset SB @ 52% liquids; Petroset RB @ 65% liquids; Peneprime @ 52% volatiles; Semi-Pave @ 50% volatiles.

[b] Dry densities are the average of all specimens.

surrounded by an apparently untreated (or asphalt free) matrix of soil particles. From Table II-3, for loess specimens, the amount of available water at the time of mixing for 6% and 8% emulsion contents was below optimum. It will be shown later that this emulsion is highly dependent on adequate water being available to facilitate dispersion. This is thought to be one of the causes of the asphalt globule formation. The cation exchange capacity of the loess (13.4 me/100 grams, whole material) may also be affecting, in some manner, the cationic emulsifier molecules present in the emulsion. This coupled with the high surface area for loess may be additional explanation for the formation of globules of asphalt and the apparent failure of the emulsion to coat soil aggregate particles with asphalt and provide complete waterproofing.

It was thought this same effect might be observed with Bedford limestone material, which has a cation exchange capacity of 10.9 me/100 grams (whole sample) and the Redicote cationic emulsion. As will be seen later, however, poor waterproofing characteristics did not occur for the Bedford material but instead improved substantially. The reason for this behavior is subject to question but may be related to the fact that the loess has a somewhat higher cation exchange capacity and much larger surface area than Bedford limestone and hence greater opportunity for cationic activity to take place. This is coupled with the fact that moisture content of the Bedford specimens was maintained near optimum for all additive contents (Table II-3). The above considerations indicate that surface area, moisture content, and possibly cation exchange capacity of a soil may be important factors in cationic emulsion stabilization.

Figures II-2-8 present the results of the unconfined compression tests with Bedford limestone material. Several general observations will be discussed.

For untreated material there was a significant increase in strength of about 500 lb from the no cure to the 24-hour air-cured tests. There was also an increase in strength of about 250 lb from the 24-hour to the 72-hour cures. The increase was, however, much less than the initial 24-hour strength increase and can be expected to increase at a decreasing rate with time. Much of this strength increase can probably be attributed to high strength bonds created by evaporation of water.

For specimens treated with asphalt (i.e. Peneprime, Semi-Pave, MC-800 and Redicote emulsion) the following observations can be made with respect to the untreated condition: (a) For specimens tested with no curing there was a slight decrease in strength with increasing amount

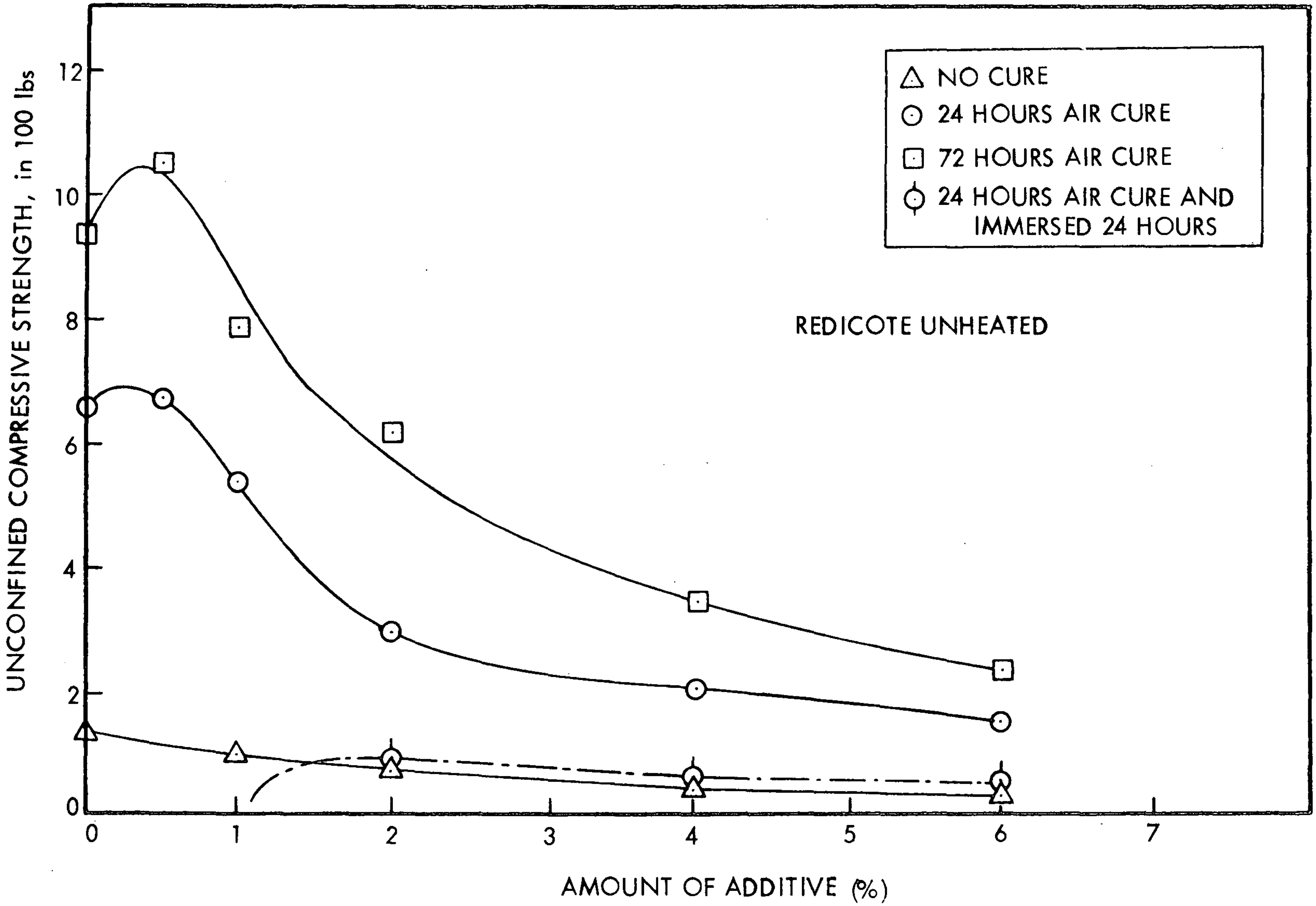


Fig. II-2. Unconfined compression test results, Redicote emulsion and Bedford limestone.

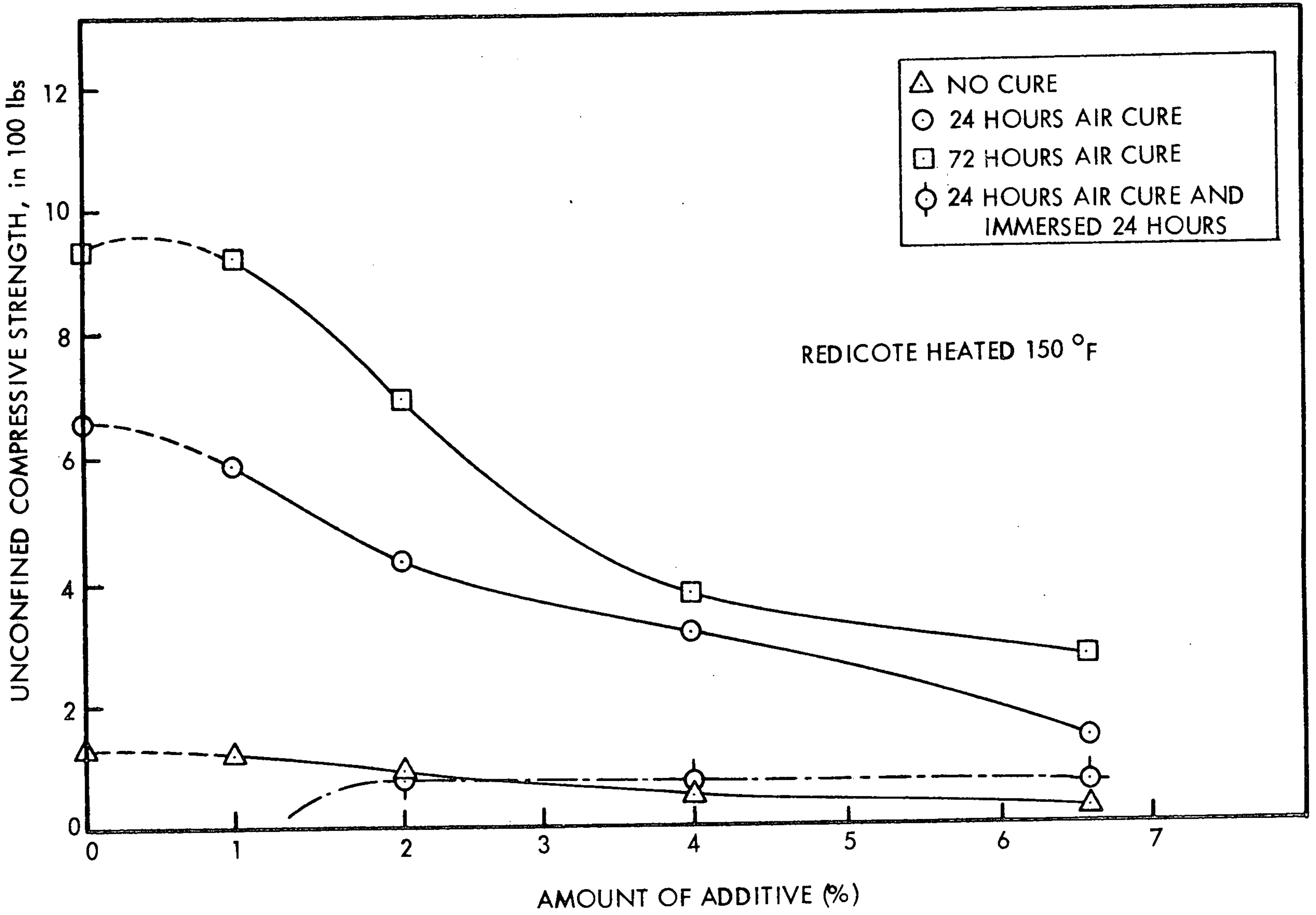


Fig. II-3. Unconfined compression test results, Redicote emulsion and Bedford limestone.

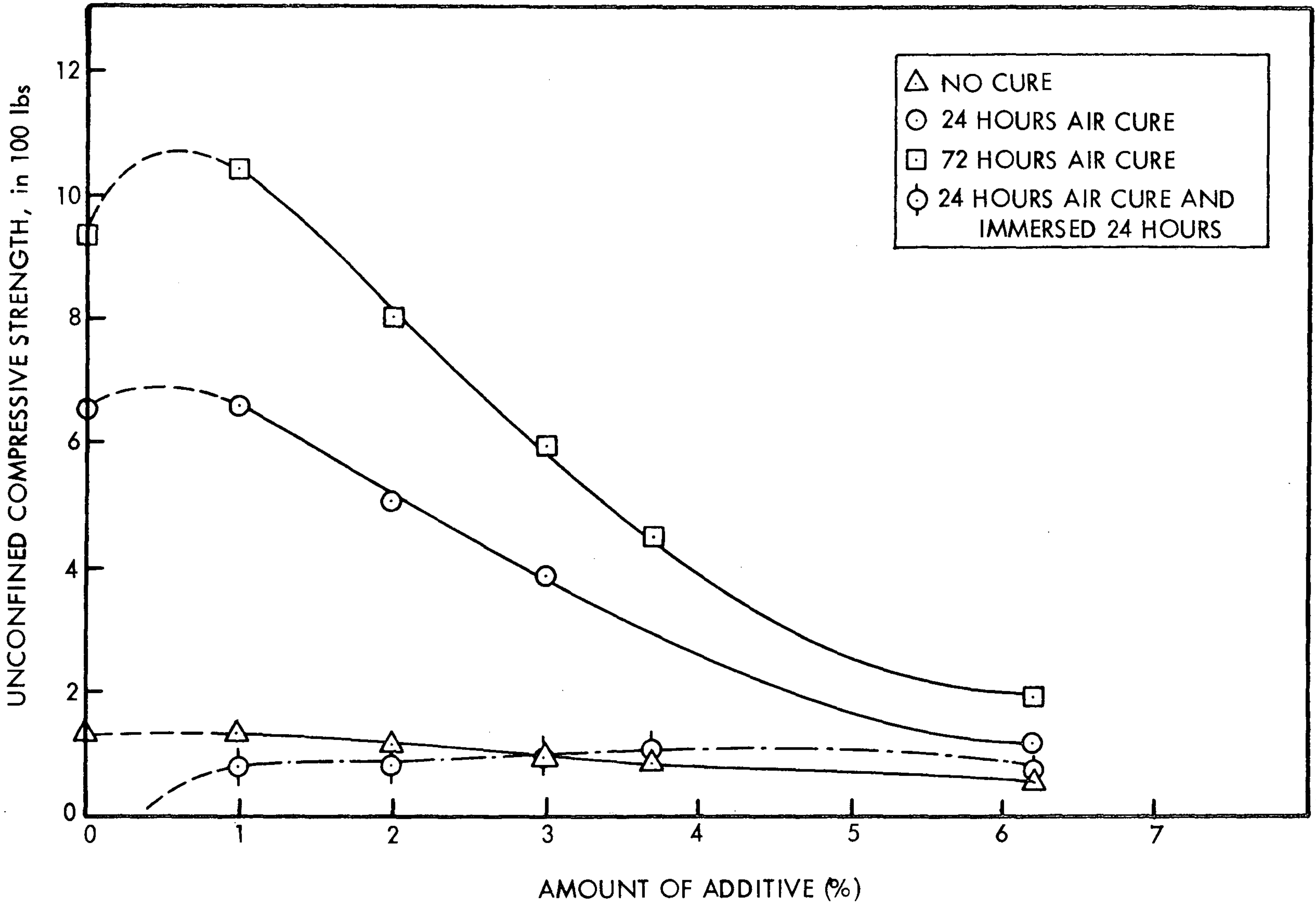


Fig. II-4. Unconfined compression test results, MC-800 and Bedford limestone.

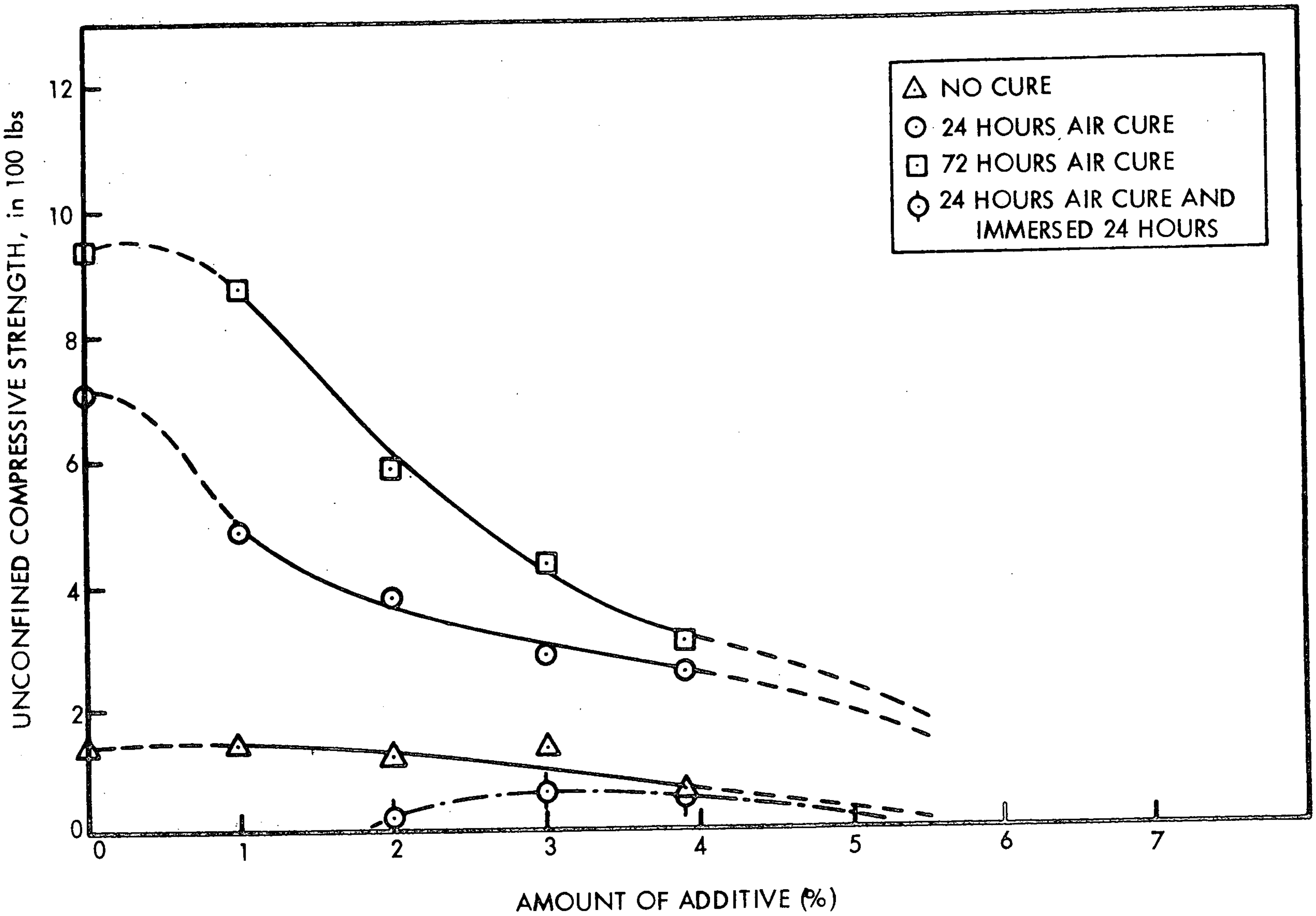
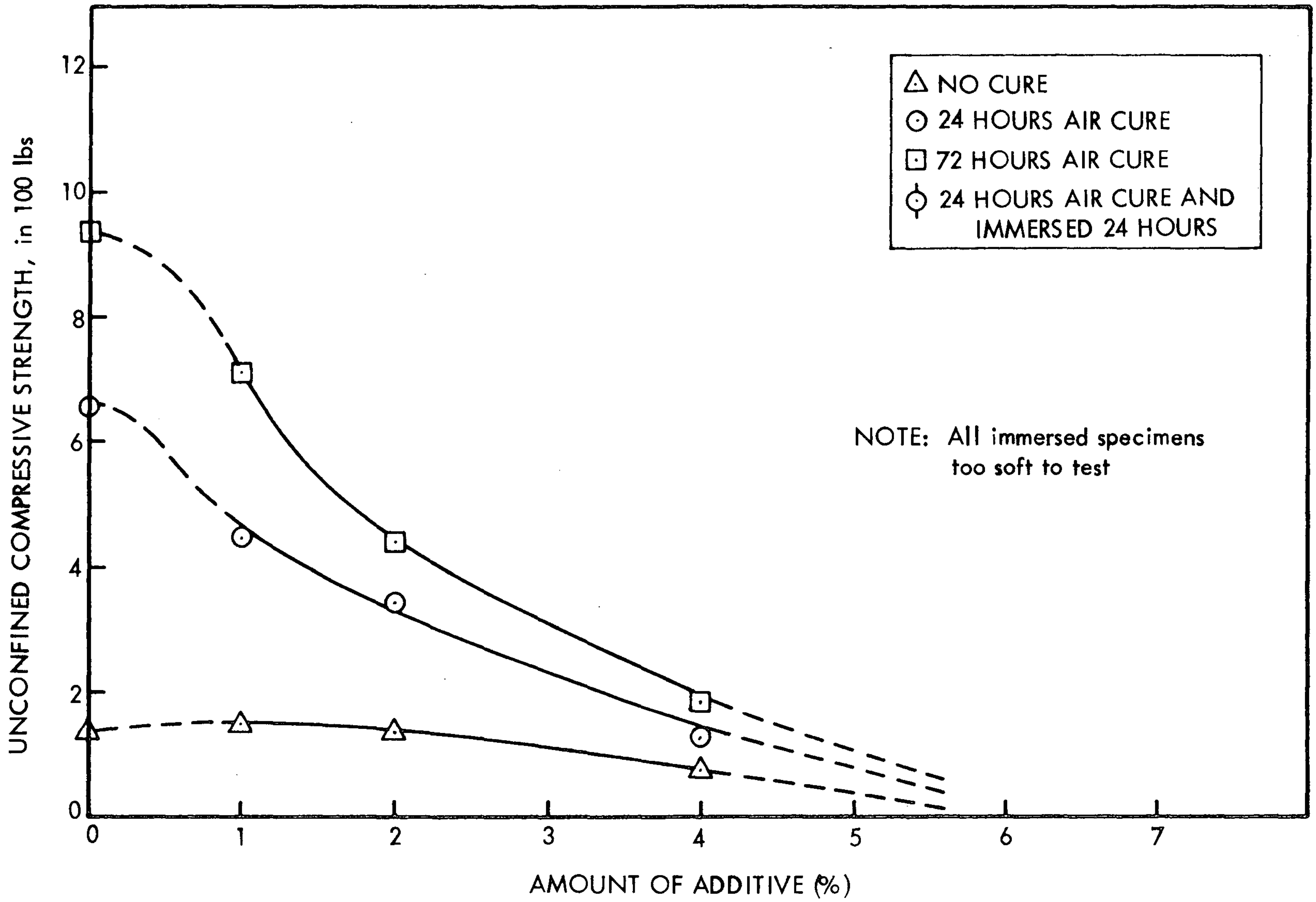


Fig. II-5. Unconfined compression test results, Peneprime and Bedford limestone.



NOTE: All immersed specimens too soft to test

Fig. II-6. Unconfined compression test results, Semi-Pave and Bedford limestone.

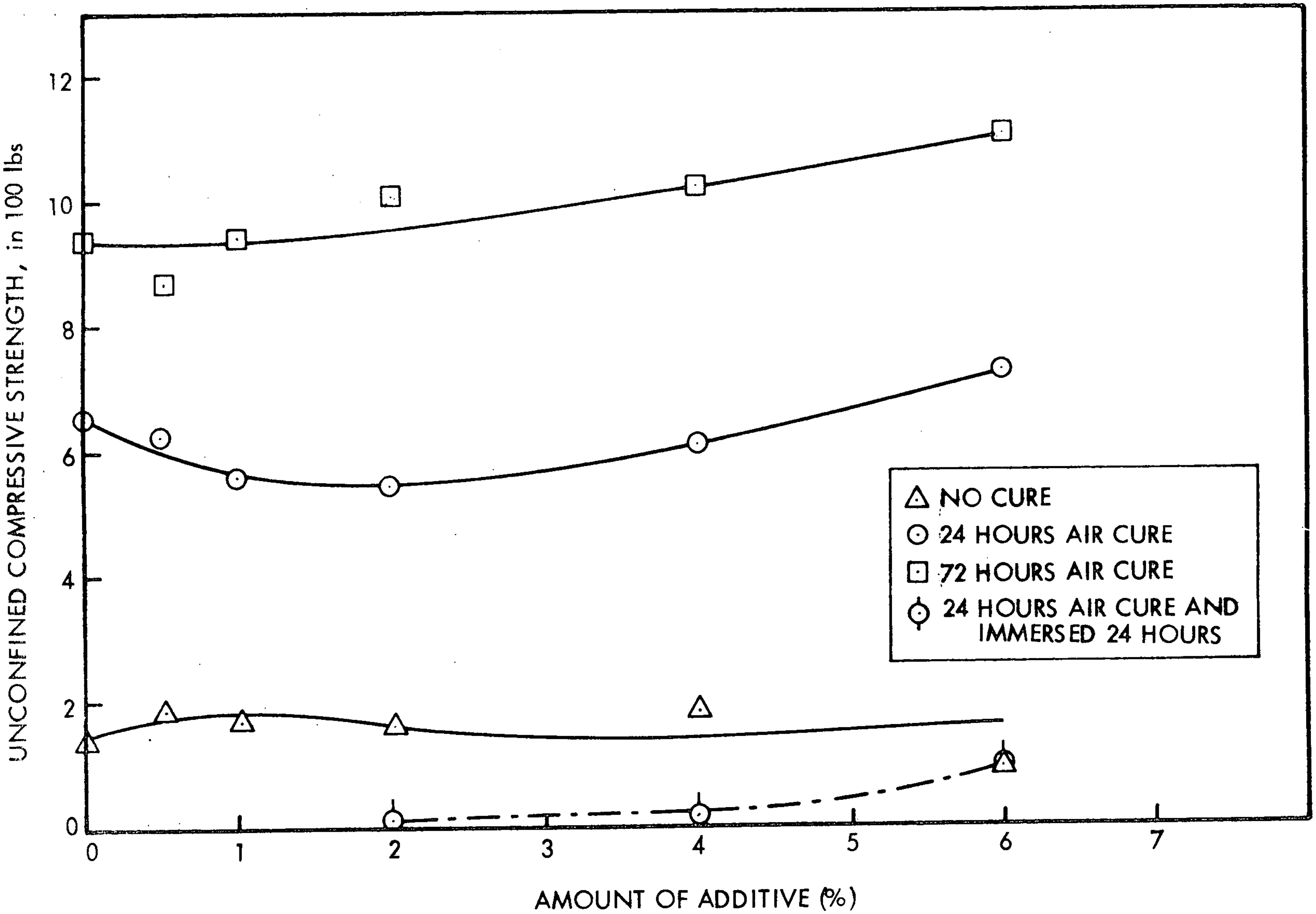


Fig. II-7. Unconfined compression test results, Petroset RB emulsion and Bedford limestone.

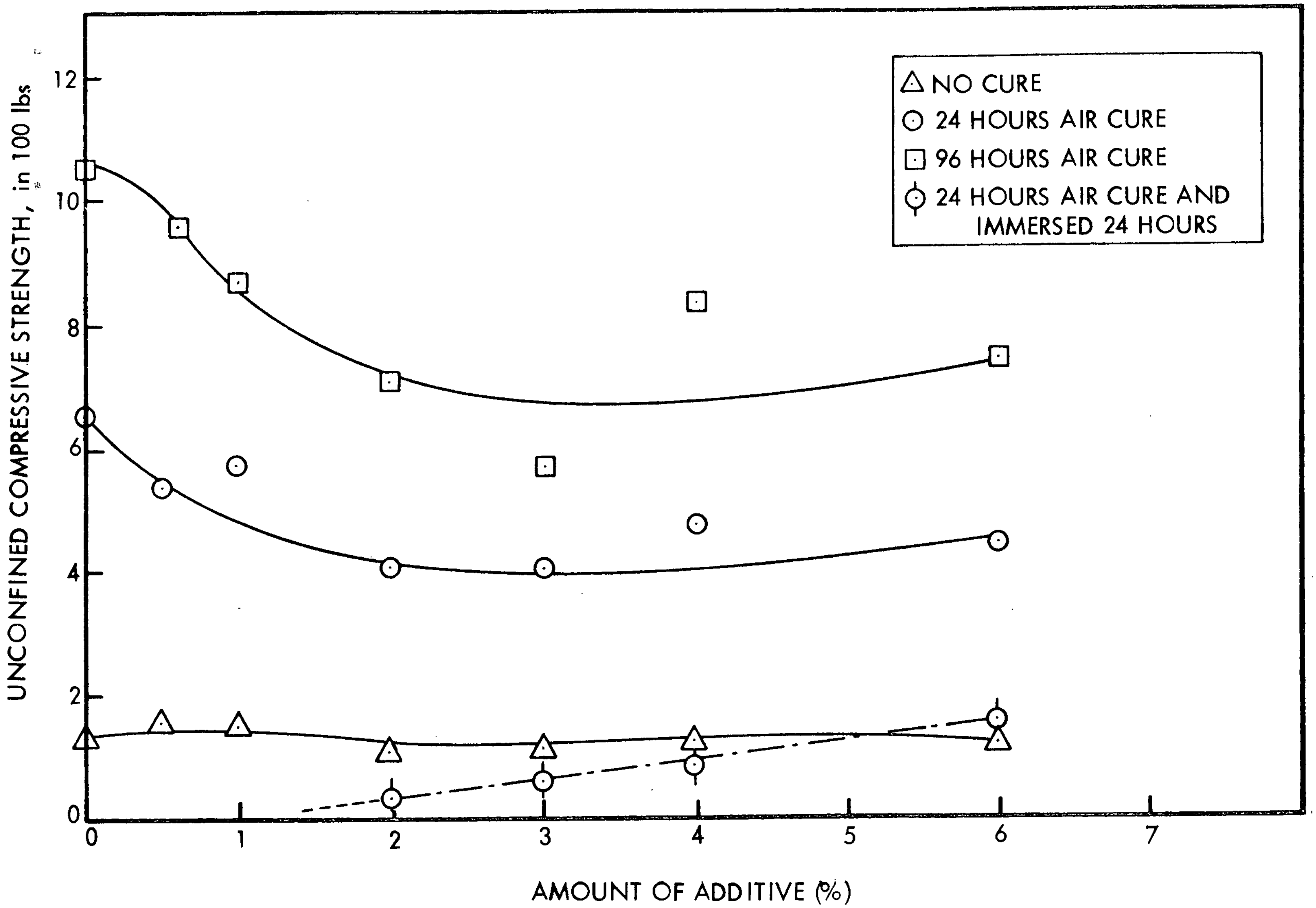


Fig. II-8. Unconfined compression test results, Petroset SB emulsion and Bedford limestone.

of additive irrespective of the type of additive. This was nearly a straight line decrease. (b) For specimens tested at 24 hours air cure there appeared to be a slight increase in strength, up to an additive content somewhat less than 1%, followed in general, by a rather abrupt decrease in strength as additive content was increased up to the 4-5% range. Past 4-5%, a more gradual decrease in strength was noted. Initial strength increase at additive contents less than 1% can probably be attributed to the cohesion imparted to the specimen by the asphalt.

The abrupt decrease is probably indicative of increasing lubrication by asphalt coated aggregates of the fine material as well as asphalt replacing former high strength evaporated water bonds as additive content increases. These same trends can be seen with the 72-hour air cured specimens. At high additive contents and for all times of air cure and additive type, there appeared a tendency to approach the strength of the uncured specimens.

Asphalt treated specimens which were air cured 24 hours then immersed for 24 hours also displayed characteristic curves for all additive types. With the exception of MC-800, all specimens treated with 1% additive either slaked to a pile or retained their form but were too soft to test after immersion. At 2% additive content, all specimens exhibited a waterproofing capability which enabled a test to be run. In general there was no significant increase in strength with increasing additive content but rather a plateau reached in the 2-3% range, which nearly duplicated the no-cure strength curve.

Failure mechanism in the unconfined compression test for asphalt treated specimens ranged from brittle fracture of the untreated and low additive content specimens, to a plastic failure at high additive contents in which the specimen bulged and cracks were formed but remained coherent. No distinguishable difference between asphalt additive types were observable on the basis of failure mechanism, however.

The latex emulsion specimens (Petroset SB and Petroset RB) behaved much differently than the asphalt treated specimens with respect to the untreated condition. Referring to Fig. II-7 and II-8, it can be seen that the specimens tested with no cure generated about the same type of curve as the asphalt treated samples with increasing additive content. The 24-hour air cured specimens, however, exhibited an initial decrease in strength up to about 2-3% additive content and then began to increase in compressive strength. Strength at 6% additive for both Petroset SB and Petroset RB was in excess of 450 lb compared to the approximate average of 150 lb for all the asphalt additives at the same content. The same tendency is noted for the 72-hour air cured specimens. Specimens tested after 24 hours air curing and 24 hours of immersion also behaved differently than those treated with asphalt. Referring to Fig. II-8 for Petroset SB, the first test possible was at 2% additive. At higher additive content, immersed strength began to climb, reaching about 150 lb compressive strength at 6%, well above the values of all other additives, indicating good waterproofing capability. The Petroset RB by contrast had the lowest compressive strength of all additives after 24 hours air curing and 24 hours immersion, and the highest strength of all additives under all other curing conditions as noted from Fig. II-7. It should also be noted that the curves generated for the latex emulsion are not nearly as well defined nor do they appear to follow a pattern with curing time as do the specimens treated with some form of asphalt. Individual test results for the latex emulsions were highly variable and reproducibility was difficult. The reason for this behavior is not known. Points plotted on Fig. II-7 and II-8 are the average value of at least two tests, some are the average of 4-6 specimens.

The reason for the latex emulsions exhibiting higher compressive strengths can probably be attributed to the physical properties of the latex versus those of asphalt. Latex in the Petroset SB and Petroset RB emulsions, which resembles in appearance and color ordinary rubber cement, has been modified to possess high tensile strength, in comparison with ordinary latex. It is probably this property of high tensile strength in contrast to the low tensile strength of asphalt that produced the higher compressive strengths of the latex emulsion treated specimens.

Failure mechanism for the latex emulsions ranged from brittle fracture at low additive contents to an apparent elastic type failure at high additive content. This was evidenced by a yield load at failure, that did not decrease with continued deformation, as the specimen bulged and cracks formed along specimen sides. On stopping and reversing the unconfined unit, some specimens appeared to rebound, in contrast to asphalt treated specimens, with side cracks visibly closing. This was evident in only a few specimens, all at the higher additive contents and was probably the result of the elastic nature of the additive. This indicates, however, that a high degree of flexibility would probably be imparted to a road surface using these additives.

Figures II-9-12 illustrate the combined plots of all specimens and all additives under each curing condition.

With respect to Figs. II-9-11, the asphalt treated specimens (i.e. Peneprime, Semi-Pave, MC-800 and Redicote emulsion) tend to band, in characteristic curves of decreasing compressive strength with increasing additive content. In general, the MC-800 specimens have higher strength initially for the 24-hour air cure and 72-hour air cure times up to an additive content of about 3-4% where the Redicote emulsions exhibit slightly greater strengths. In the no cure specimens, MC-800 had greater strength than the emulsions at all additive contents. Peneprime exhibited, in general, lower strengths than the other additives for all curing times, with the exception of the no cure specimens, at additive contents up to 3% (where it exhibited about the same compressive strength as MC-800 specimens). The lower strength for both Semi-Pave and Peneprime was probably due to their high volatile contents, 50% and 52% respectively.

Dry density variations of the unconfined compression specimens, as molded, are given in Table II-3. There was a general trend for decreasing dry density with increasing additive content noted for all additives with the exception of the cutback asphalt samples (i.e. MC-800, Peneprime and Semi-Pave) which exhibited a slight increase in density with higher additive contents. This is thought to be due to somewhat higher liquid contents and the oily cutback constituent acting as a lubricant during compaction. This is evidenced in comparing the dry density of untreated Bedford limestone with the dry density of the lowest additive content specimens for each type of additive. It can be seen that the density of nearly all additives, other than the cutbacks,

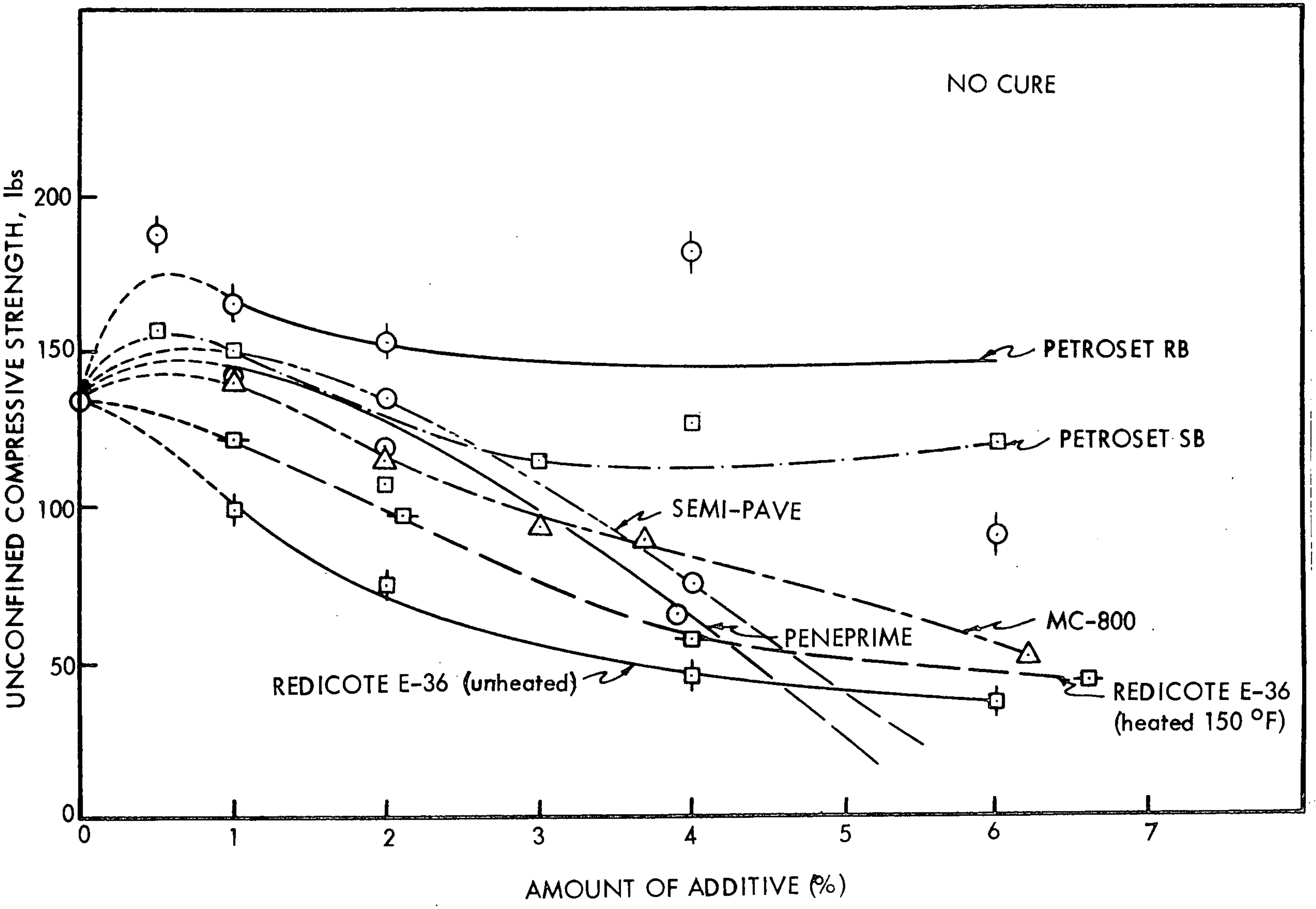


Fig. II-9. Unconfined compression test results, Bedford limestone.

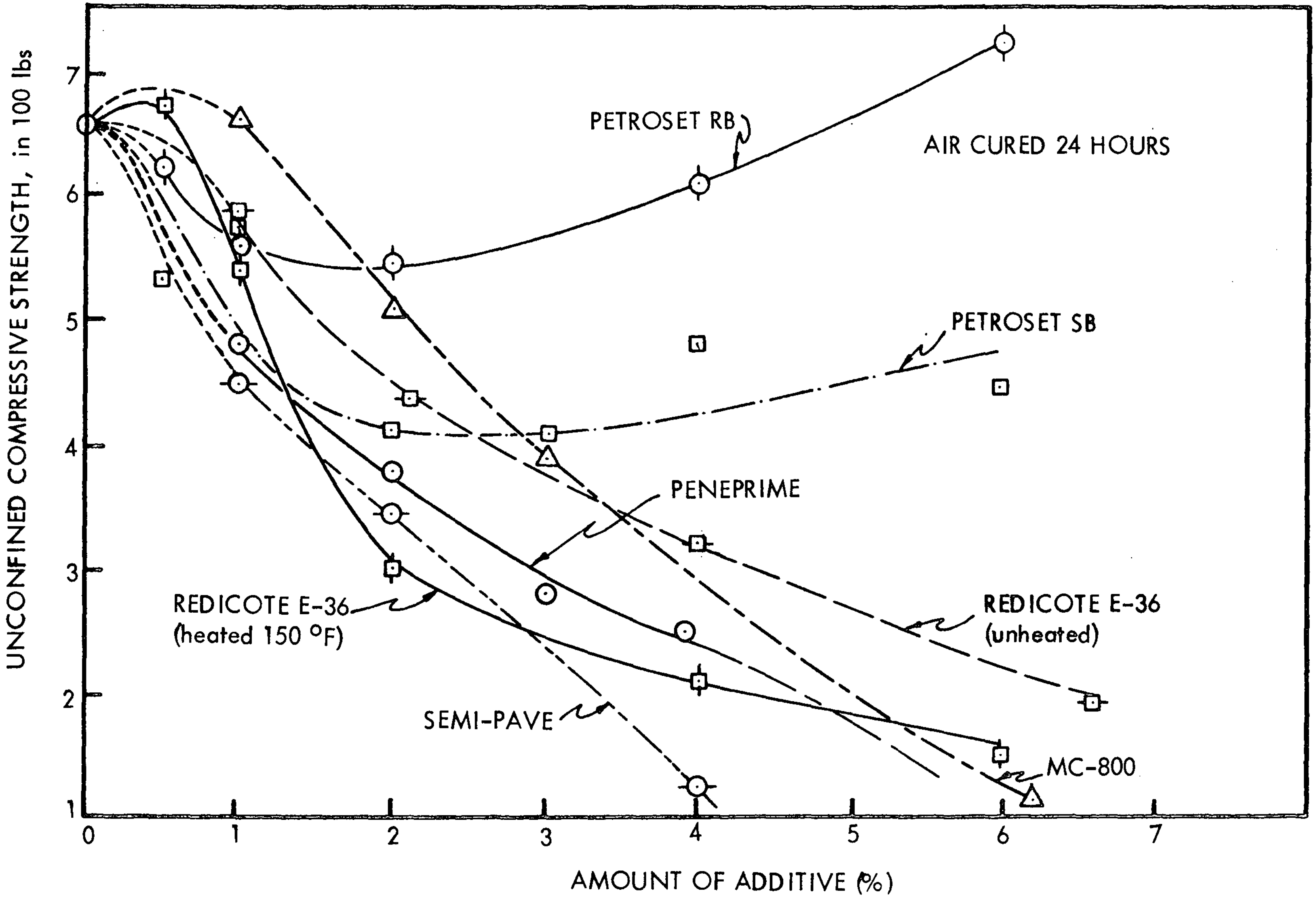


Fig. II-10. Unconfined compression test results, Bedford limestone.

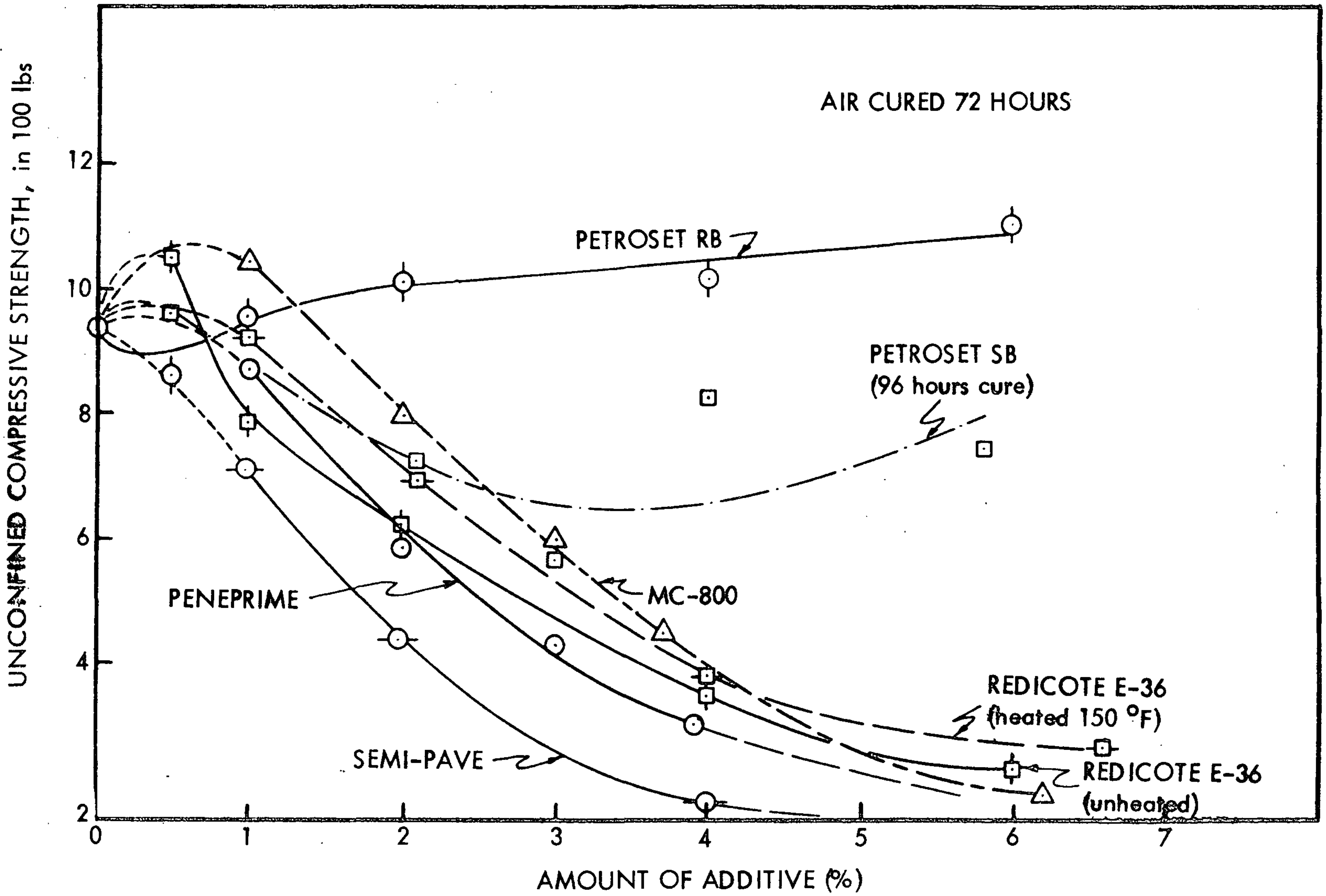


Fig. II-11. Unconfined compression test results, Bedford limestone.

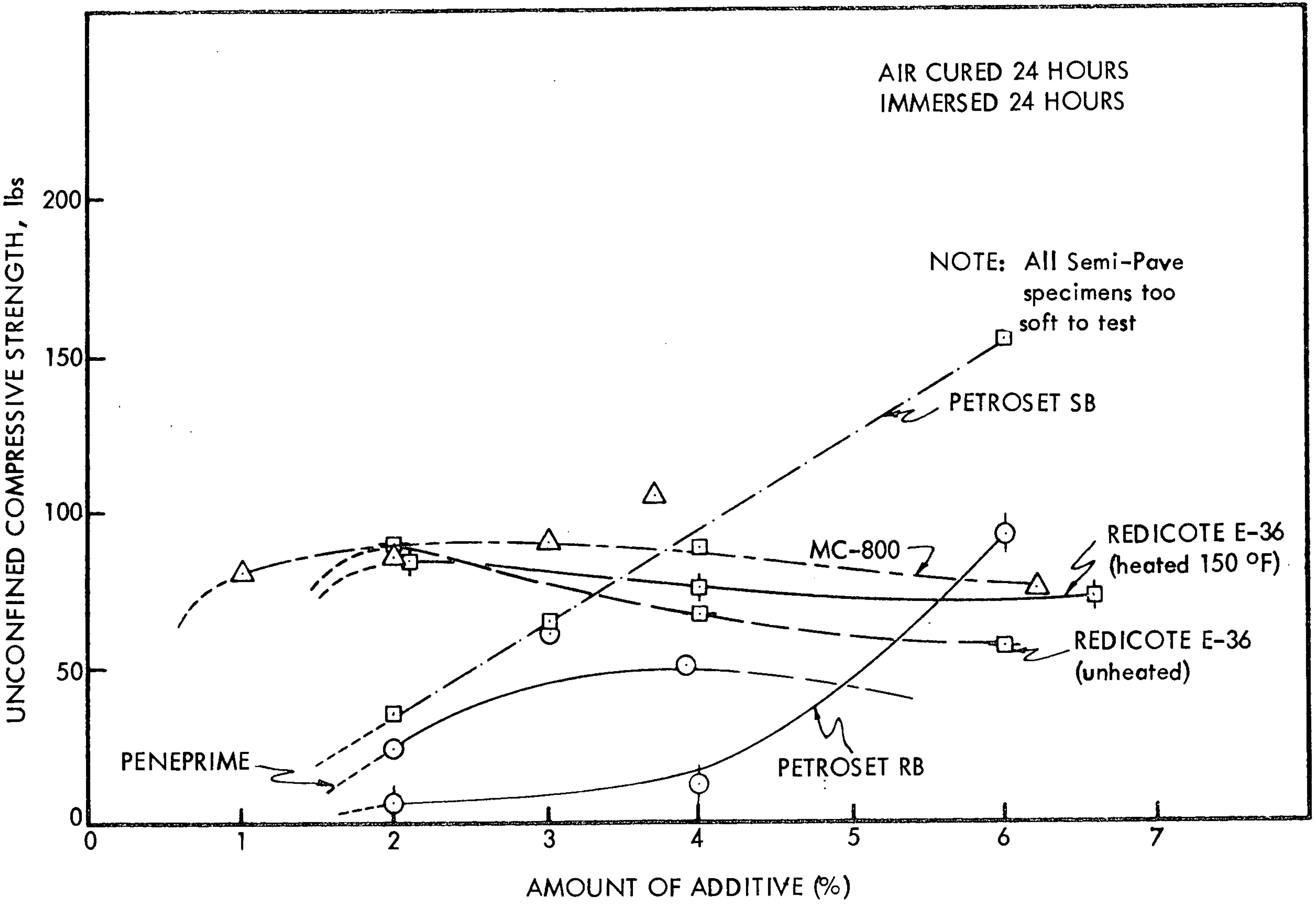


Fig. II-12. Unconfined compression test results, Bedford limestone.

are lower than the untreated dry density of 119.9 pcf. The cutback specimens, especially Peneprime and Semi-Pave which have high amounts of cutback solvent, gave dry densities nearly 5 pcf above the untreated material. Specimens treated with Redicote emulsion, which does not contain a cutback solvent, had dry densities below 119.9 pcf. Although the dry density variation of specimens probably affected the strength in some manner, an examination of Figs. II-9-11 indicate no discernible trend. The higher strengths noted for Peneprime and Semi-Pave, for the no cure condition on Fig. II-9, was probably partially due to their high densities. From Figs. II-10 and II-11, however, they exhibited the lowest strengths.

Petroset SB and Petroset RB exhibited much higher strengths past the 2-3% range, for air cured specimens, than all asphalt treated specimens. This is thought, again, to be due to the high tensile strengths developed by the latex as opposed to the lubrication effect of asphalt at higher contents.

Figure II-12 indicates the behavior of specimens air cured 24 hours and then immersed 24 hours. For MC-800 and Redicote emulsion, a plateau was reached at 2% additive content that only slightly decreased at higher contents. Petroset SB exhibited a sharp, nearly straight line increase in strength that was above that for all other specimens past 4% additive content. Petroset RB, which exhibited the highest air cured strength, had very low immersed strength. This was thought to be related to the fact that, for normal use, it is recommended by the manufacturer that an ammonia solution be used to act as a catalyst for "breaking" the emulsion at aggregate contact points. Petroset RB is designed as a rock binder and originally intended for use on materials such as railroad ballast where there are little or no fines present. Petroset SB, by contrast, is specifically designed as a soil binder, although most past usage has been as a surface application for dust palliation. An ammonia solution is not needed for use with Petroset SB. All Semi-Pave treated specimens were too soft to test on completion of immersion. This, again, was felt to be primarily due to high volatile content (50%). Peneprime, which also has a high volatile content (52%), exhibited low immersed strengths.

Summary of Results

1. All asphalt treated specimens (i.e. Peneprime, Semi-Pave, MC-800 and Redicote emulsion) for no cure, 24-hour air cure, and 72-hour air cure testing conditions tend to band in characteristic curves with a slight increase in strength at low additive contents followed by a sharp decrease in strengths as total additive content was increased to the 3-4% range. With increasing additive contents up to the 6-7% range a more gradual decrease in strength was noted.

2. Latex emulsion treated specimens (Petroset SB and Petroset RB) behaved much differently than asphalt treated samples for the no cure, 24-hour air cure and 72-hour air cure test conditions. Above approximately 2% additive content, strength began increasing following an initial decrease. Strengths at higher additive contents were well above those for asphalt treated specimens. This behavior is probably due to the high tensile strengths developed by the latex.

3. There was a general trend for decreasing dry density from the untreated condition, with increasing additive content for all additives

(with the exception of the cutback asphalt samples Peneprime, Semi-Pave and MC-800) which exhibited higher densities than untreated material. This is partially due to somewhat higher liquid contents and the oily cutback constituent acting as a lubricant during compaction. There was, however, no apparent trend in the unconfined strengths corresponding to density variations.

4. Results from specimens air cured 24 hours and immersed 24 hours indicated that MC-800 cutback asphalt and Redicote E-36 cationic asphalt emulsion appeared to waterproof most effectively over a broad range of additive contents although immersed strength did not increase with additive content but reached a "plateau" at approximately 2%. Past the 4% additive content, however, Petroset SB latex cationic emulsion exhibited superior waterproofing capabilities.

5. Redicote E-36 cationic emulsion used with both loess and Bedford limestone exhibited relatively the same strength trends for the air cured specimens. Immersed behavior was considerably different, however, with all specimens being too soft to test with the loess material, although specimen form was retained in varying degrees between 1-4% additive content. This may be related to the fact that very little distribution of asphalt was apparent in the loess specimens; asphalt appearing as small visible globules imbedded in a matrix of soil past 6% emulsion content. This behavior is probably the result of a combination of low moisture contents at the time of mixing of loess specimens and the large surface area of loess compared with Bedford limestone. The cation exchange capacity of the loess may also be reacting in some manner with the cationic emulsifier molecules present in the emulsion.

Traffic Simulator Tests

A traffic simulator apparatus developed by the Bituminous Research Laboratory at Iowa State University is shown in Fig. II-13.

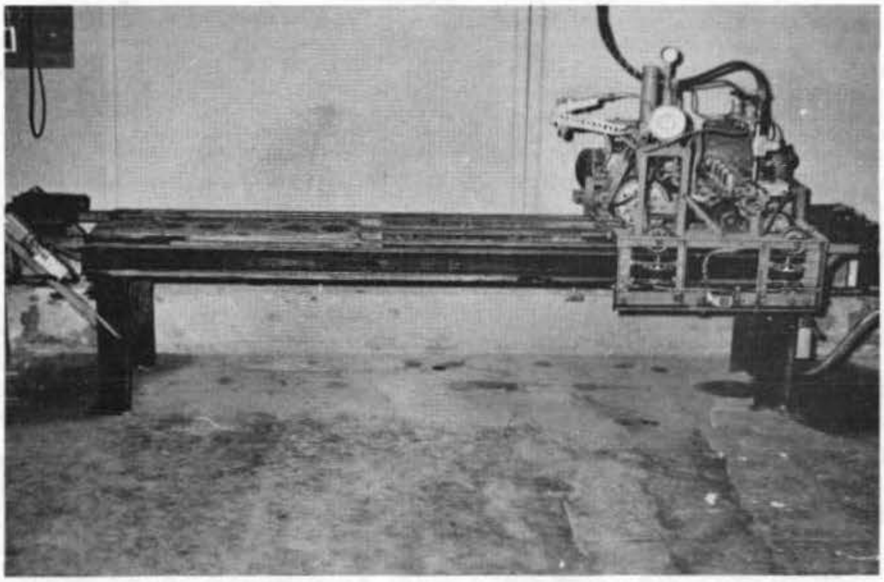


Fig. II-13. Traffic simulator apparatus.

This device was taken from storage, installed in the Soil Research Laboratory and modifications added. Only a brief summary of its operation and description will be given here. A detailed description of the traffic simulator and its operation is given by Csanyi and Fung[24].

The traffic simulator consists of a main frame about 11 ft long, by 3 ft wide, by 2 ft high supported by legs bolted to a concrete slab. The frame supports a traveling carriage and specimen retainer box. The carriage operates in an oscillating to-and-fro motion with an 8-in. diameter, 1-1/4-in. wide solid rubber tire imposing the load and driving the carriage during its forward motion. The carriage is powered by a 1/2-hp motor connected to the loading wheel and auxiliary, reverse driving wheels through a reducing gear and belt drive in such a manner that the wheels rotate in their respective directions continuously. During the reverse motion the loading wheel is retracted and the auxiliary drive returns the carriage. In this manner one-way traffic is simulated.

Execution of the oscillating operation is controlled automatically by an electric switching arrangement. The loading wheel and auxiliary reverse drive wheels are operated by a double-acting compressed air ram actuated by electric solenoid air valves. Compressed air and electrical power are fed to the carriage by flexible lines suspended overhead.

Side swaying and bucking are prevented by several rubber guide and retaining wheels. Travel speed of the carriage is slightly less than 4 mph, which allows somewhat over 1000 passes per hour. A counter, which can be preset, counts the number of forward passes made and automatically stops the carriage at any specified number of passes. Holding clamps are provided on the front of the carriage for a displacement measuring gage. Air pressure to the ram of the loading wheel was regulated to provide a contact pressure of about 85 psi, as determined by imprints of tire contact area on a flat, hard surface. A modified paint sprayer was attached to the front of the carriage to provide an even spray of distilled water, simulating rain over the specimens during operation. The sprayer was fed by a small pressurized water tank attached to the carriage. Pressure regulators were installed on the carriage enabling the pressure to the water tank to be controlled independently of the air pressure to the spray gun. In this manner it was possible to regulate a fine, uninterrupted spray of water amounting to 0.15-0.20 in./hr over the test specimens.

The specimen retainer box shown in Fig. II-14 is mounted in the forward end of the frame. Constructed of a 3/8-in. steel plate, it is 36 in. long, 8 in. wide and 3-1/4 in. high. It is fitted with a 1/4-in. thick steel cover in which six holes have been bored to hold, and centrally align the specimen holding rings in position along the line of travel of the loading wheel. Six specimens may be tested simultaneously. In the bottom of the box, under each specimen holding ring, three screws, acting upon a 1/4-in. steel holding-ring insert, are provided for adjusting the surface of each test specimen with the top of the retainer box.

Test specimens were molded in their individual holding rings, placed in the specimen retainer box with the steel top fitted over them

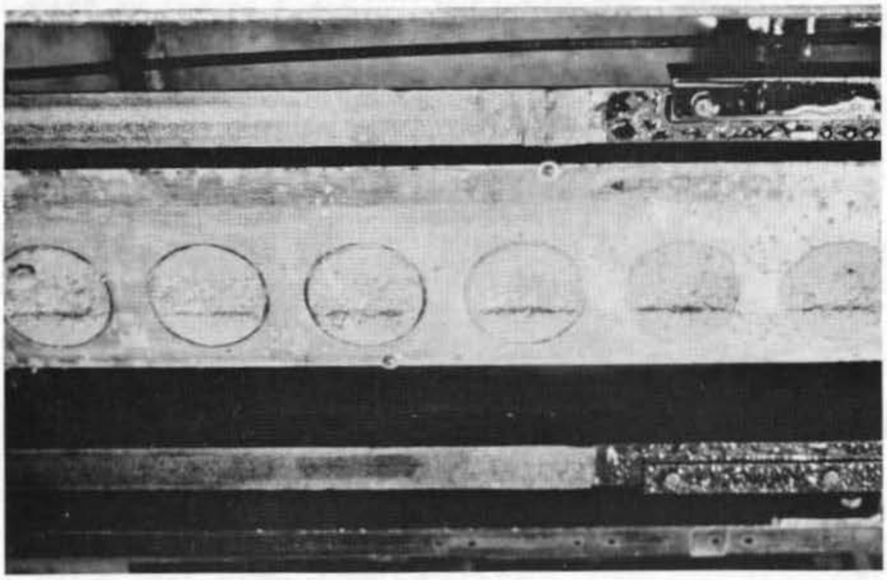


Fig. II-14. Traffic simulator samples mounted in specimen retainer box.

and secured. Prior to testing, the carriage was brought forward, the loading wheel retracted and an Ames dial gage, capable of measuring to 0.001 in. deflection, was attached. A reference reading was taken on top of the specimen retaining cover immediately in front of each specimen in the centerline of travel. Readings were then taken at quarter points across each specimen surface. In this manner, rut depth could be determined at any desired point in the test. Each set of six test specimens was composed of two duplicate sets of three specimens, each with a different treatment. Figure II-14 illustrates test specimens installed in the traffic simulator. Samples were placed in the specimen box end to end to reduce position effects (e.g. untreated, 2% emulsion, 2% cutback, untreated, 2% emulsion, 2% cutback). This procedure was followed throughout the study.

Specimen Preparation and Curing

All specimens to be tested in the traffic simulator were 4 in. in diameter by approximately 2.4 in. in height. Specimens were molded in holding rings capable of being installed directly, without extrusion, in the traffic simulator. During molding the rings were secured to a modified Proctor molding stand and a 1/4-in. thick, 4-in. diameter steel plate inserted in the bottom of the mold. This allowed the height of the samples to be adjusted after installation in the simulator. Specimens were compacted in two layers, each layer being rodded 25 times with a 3/4-in. diameter round tipped rod and subjected to 20 blows of a standard 5.5-lb Proctor hammer. Upon completion of this process a 1/4-in. thick, 4-in. diameter plate was inserted into the ring on top of the compacted specimen. The sample was then given an additional five blows of the hammer on the center of the plate and the plate removed. This procedure yielded a relatively smooth, partially sealed surface similar to what might be expected from field construction. On completion of molding, height and weight measurements were taken. Height of the specimen was normally slightly below the rim of the holding ring. Compacted specimens were then installed in the traffic simulator retainer box, the top holding plate attached, and all specimens raised by adjusting screws acting on the bottom steel plate to conform to the surface of the simulator retainer box. Curing was accomplished at room temperature for a period of 24 hours prior to testing.

Two specimens were molded at each additive type and percentage. Bedford limestone material was used for initial traffic simulator evaluation of the various additives.

For asphalt treated specimens MC-800, Peneprime and Redicote emulsion, the criteria that 2% emulsion or cutback was equal to 1% water was used to determine the amount of mixing water that was required to obtain the 9% total desired. Former work by Katti et al.[6] indicates, for cutbacks, that this is roughly equal to the "compromise moisture content" (cmc) that he proposed for obtaining maximum desirable mix qualities. For the latex emulsion Petroset SB total water content was maintained at 9% (i.e. added water plus water in emulsion was equal to 9%).

All traffic simulator specimens, emulsions and cutbacks were prepared in the following manner. A predetermined amount of distilled water was added to 2500 g of air-dry Bedford limestone, machine mixed in a Kitchen-Aid mixer 1 min, hand mixed 1 min and again machine mixed

1 min. The desired amount of additive was then slowly incorporated as the mixer operated at low speed. The following process was then used. Machine mixed 1 min, hand mixed 1 min, machine mixed 1 min, again hand mixed and then placed in a humid atmosphere for a minimum of 5 min prior to molding. Following molding, height and weight measurements were taken.

All additive contents used and referred to herein are on the basis of total amount of additive. Peneprime and Redicote emulsion were heated to 150 °F prior to use. MC-800 was used at 275 °F. The latex emulsion was used unheated.

Method of Testing

In order to develop a testing procedure that would be indicative of a treated versus untreated materials performance under simulated traffic and imposed environmental conditions, several trial runs using varying values of load, duration and sequence of rain and number of passes were performed on untreated and treated materials. Figure II-15 illustrates one of these trials in which center point rut depth was plotted against the number of passes for untreated, Redicote E-36 cationic emulsion treated and MC-800 cutback treated Bedford limestone. The testing sequence for this trial was as follows: (a) ram load at 80 lb (approximately 85 psi wheel contact pressure) and no rain up to 4000 passes, (b) ram load increased to 135 lb and no rain up to 7000 passes, (c) ram load remaining at 135 lb, rain started at 0.15-0.20 in./hr, tested until failure (i.e. rut depth greater than 0.5 in.). Several observations can be noted from this figure:

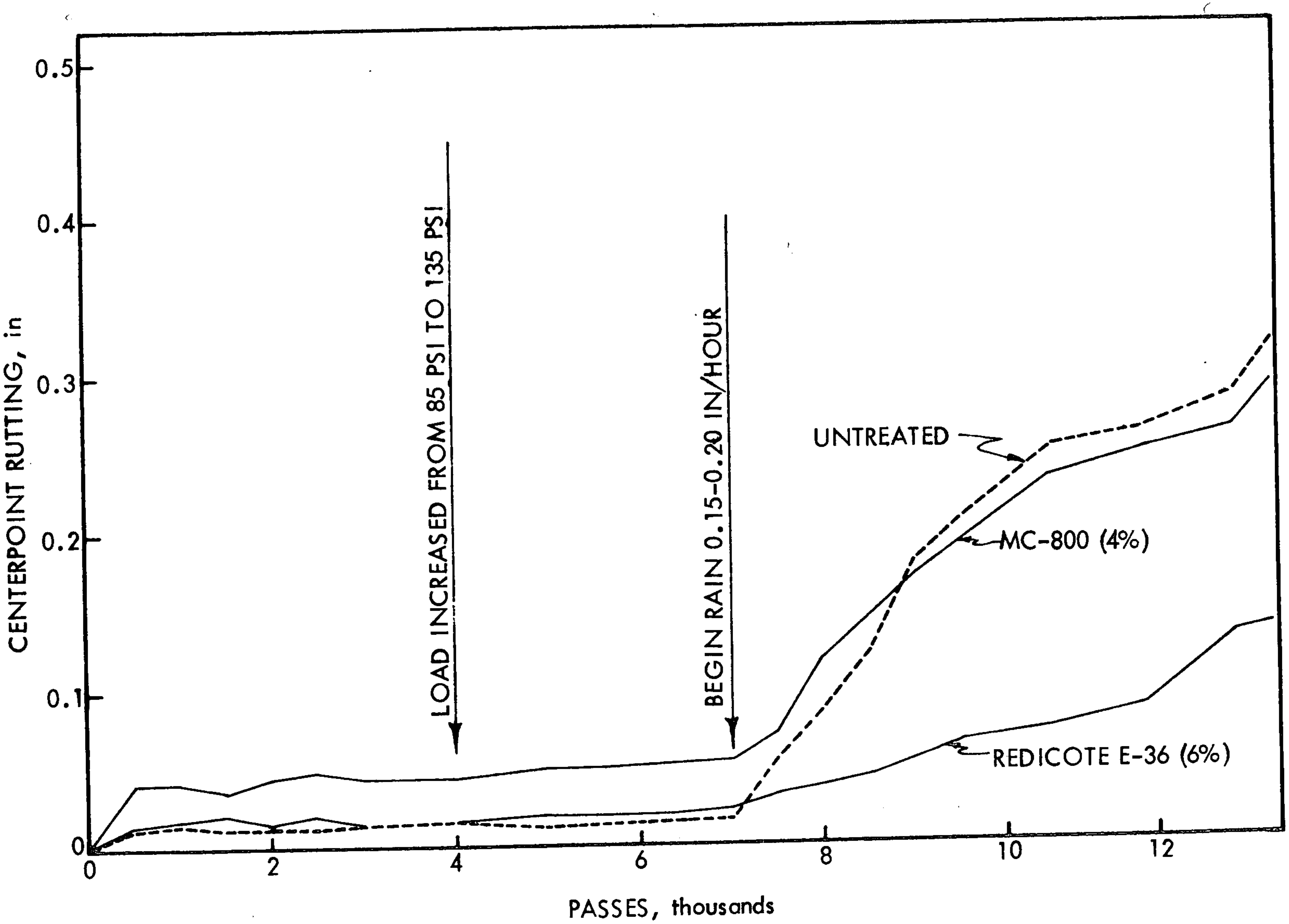


Fig. II-15. Traffic simulator data for establishing test procedure, Bedford limestone material.

1. There was an initial deflection at 500 passes that was probably the result of slight surface densification in the wheelpath and the "seating" of the specimen in the holding ring (in later tests it was found that the bulk of this initial deflection occurred at less than 50 passes, probably within the first few passes).

2. There is very little if any additional deflection up to 4000 passes.

3. Increasing the wheel load from 80 lb to 135 lb after 4000 passes only slightly increased the surface deflection at 7000 passes.

4. With the beginning of rain at 7000 passes there was a significant increase in rutting rate that continued until failure.

5. The mechanism of rut depth increase during rain appeared to be the result of the following factors.

   (a) Almost immediate "tracking" out of fine material on the untreated specimens.

   (b) Densification of the material in the wheelpath of all specimens.

   (c) After a definite visible rut had formed (0.1-0.2 in.) rain tended to puddle in the wheelpath resulting in "flushing" of coarser fine material as well as "tracking" out of fines. This process occurring to a lesser degree on treated specimens as compared to untreated specimens.

   (d) As the test proceeded the process described in (c) continued until the wheelpath surface appeared to be composed primarily of angular coarse particles with the fine binder material removed. Again this was most evident in untreated specimens.

(e) Upon continued testing, shear failure commenced in various forms. For the untreated specimens, there was general bulging and upheaval along wheelpath edges which was termed "local" shear. For some of the treated materials shear took place "en masse," away from the wheelpath along the walls of the holding ring; this was termed "full" shear.

On the basis of the foregoing results and observations the following test procedure, for traffic simulator evaluation, was adopted using an applied wheel contact pressure of about 85 psi.

1. One thousand passes with no imposed environmental conditions.
2. One thousand passes with simulated rain of 0.15-0.20 in./hr.
3. Two-hour fogging period with no traffic. Fogging adjusted so water was visibly evident on all specimen surfaces at all times.
4. One thousand passes (or until failure) with simulated rain of 0.15-0.20 in./hr.

Rut depth readings were normally taken at the end of each period of 250 passes. The above procedure was followed throughout the remainder of the study.

Results

Traffic simulator results for Redicote E-36 cationic asphalt emulsion, Petroset SB cationic latex emulsion, MC-800 cutback asphalt and Peneprime are given in Figs. II-16-21. For clarity not all tests are shown. Tests that are shown, however, generally define the band of rutting extremes for each additive type with the exception of Peneprime in which only the tests shown were conducted. Simulator tests on Semi-Pave,

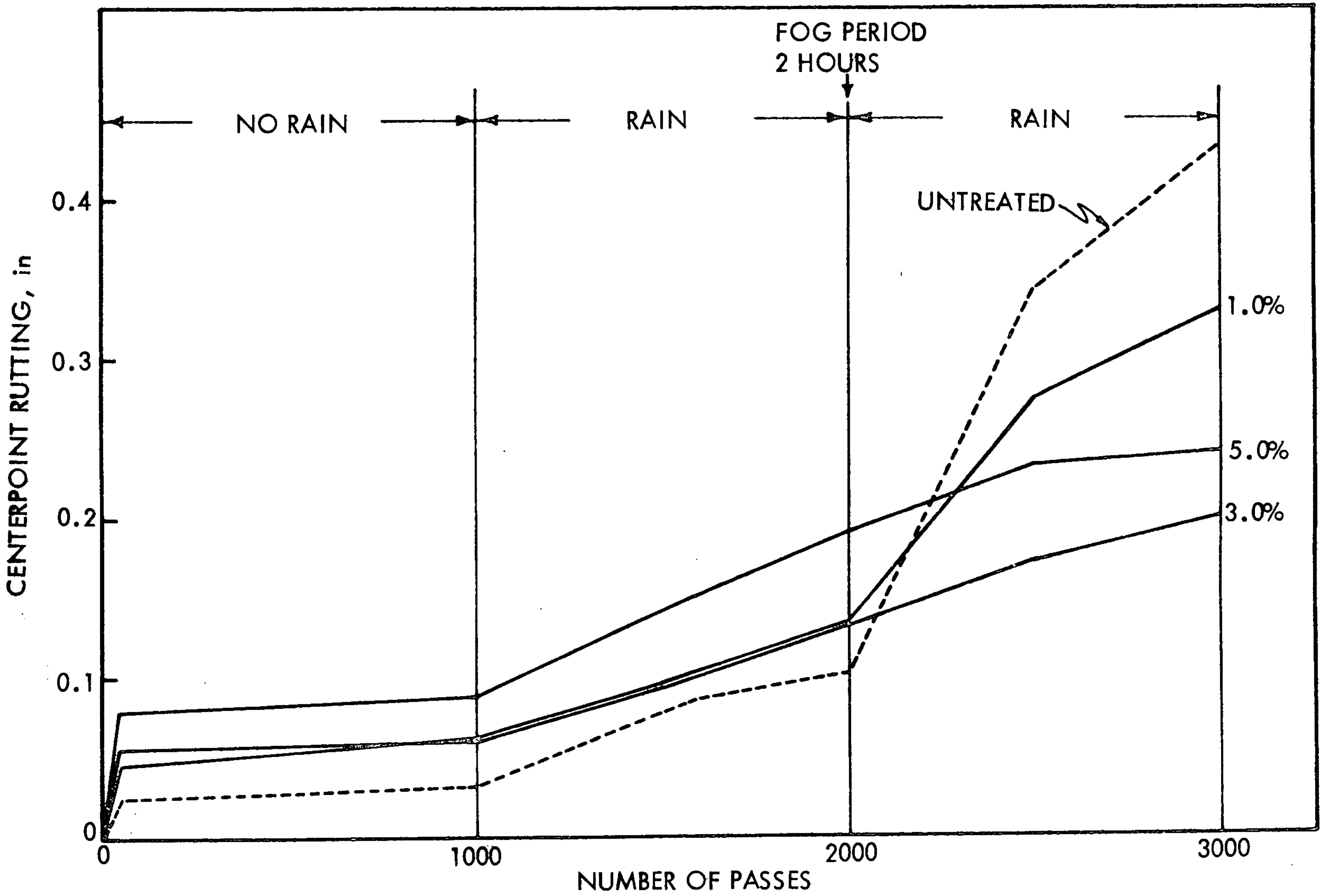


Fig. II-16. Traffic simulator results, Petroset SB cationic latex emulsion, Bedford limestone.

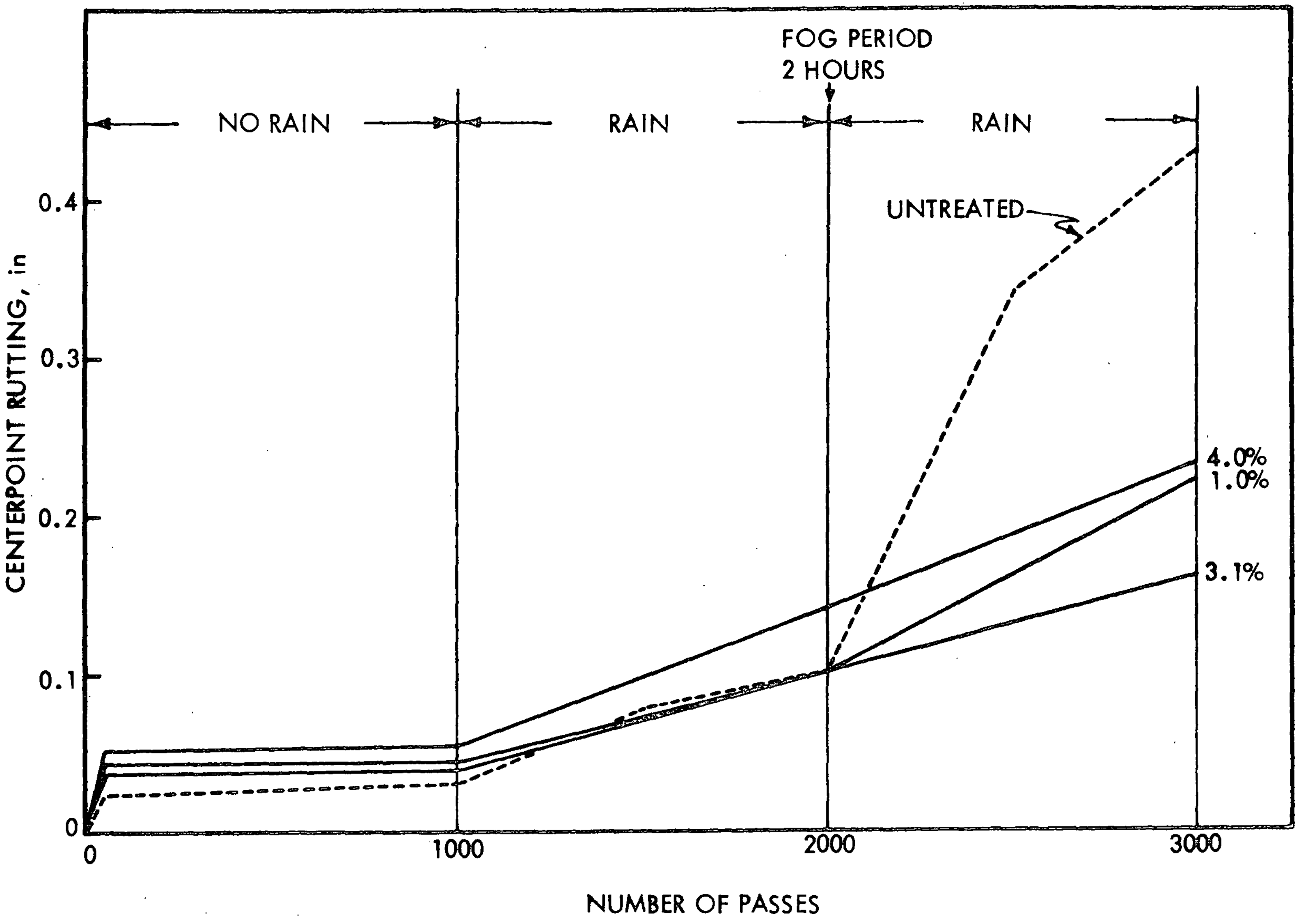


Fig. II-17. Traffic simulator results, Redicote E-36 cationic asphalt emulsion, Bedford limestone.

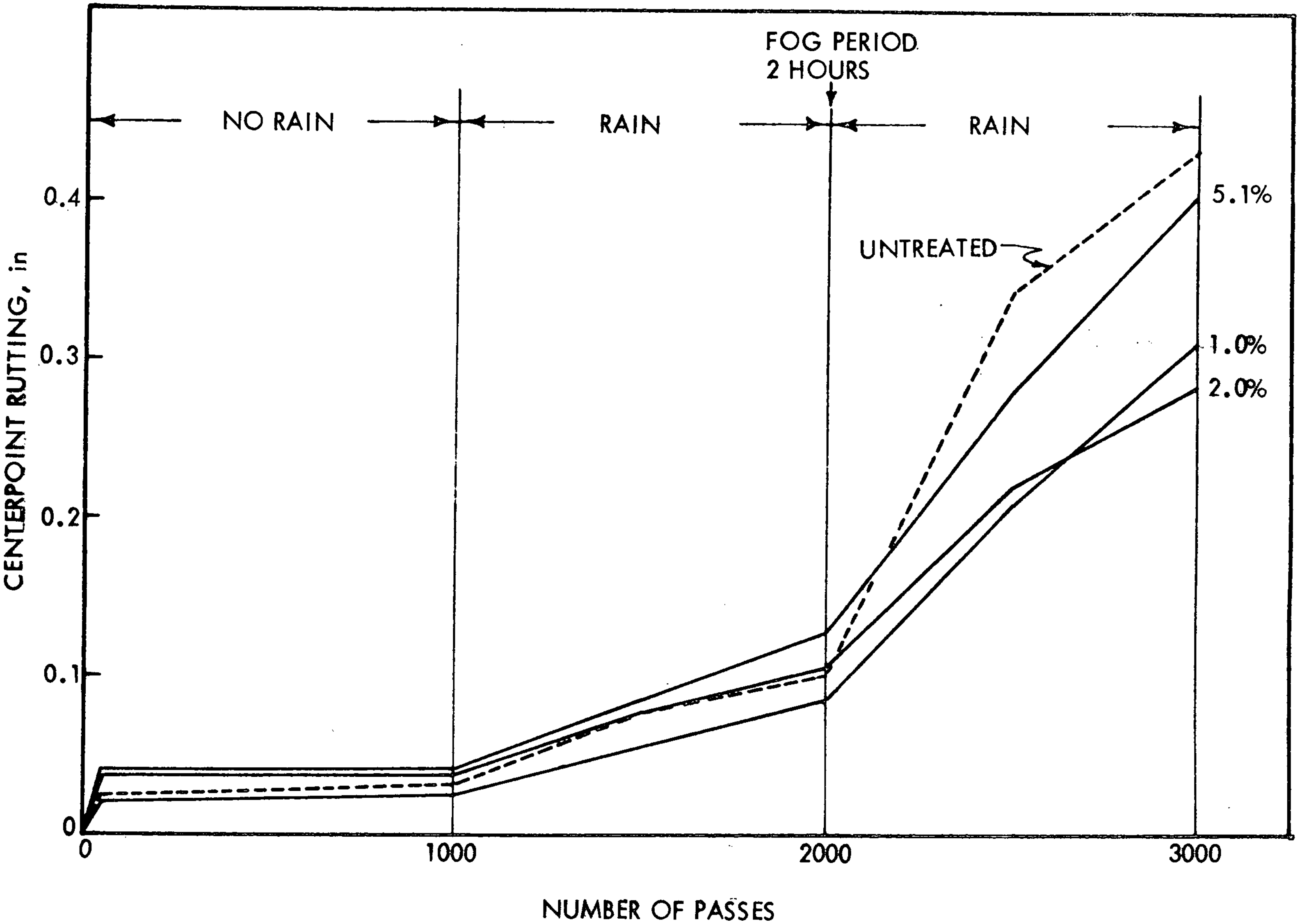


Fig. II-18. Traffic simulator results, MC-800 cutback asphalt, Bedford limestone.

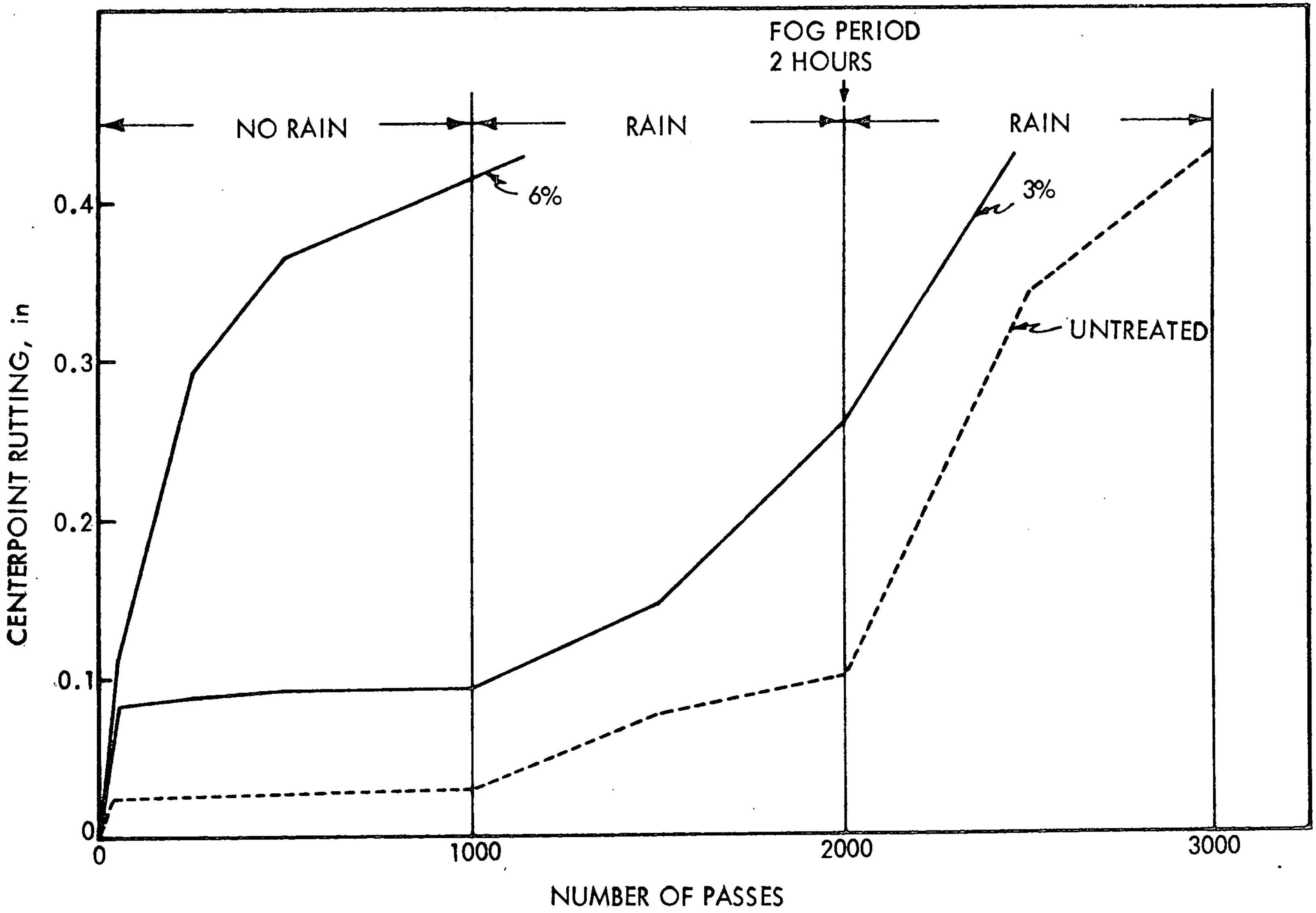


Fig. II-19. Traffic simulator results, Peneprime and Bedford limestone.

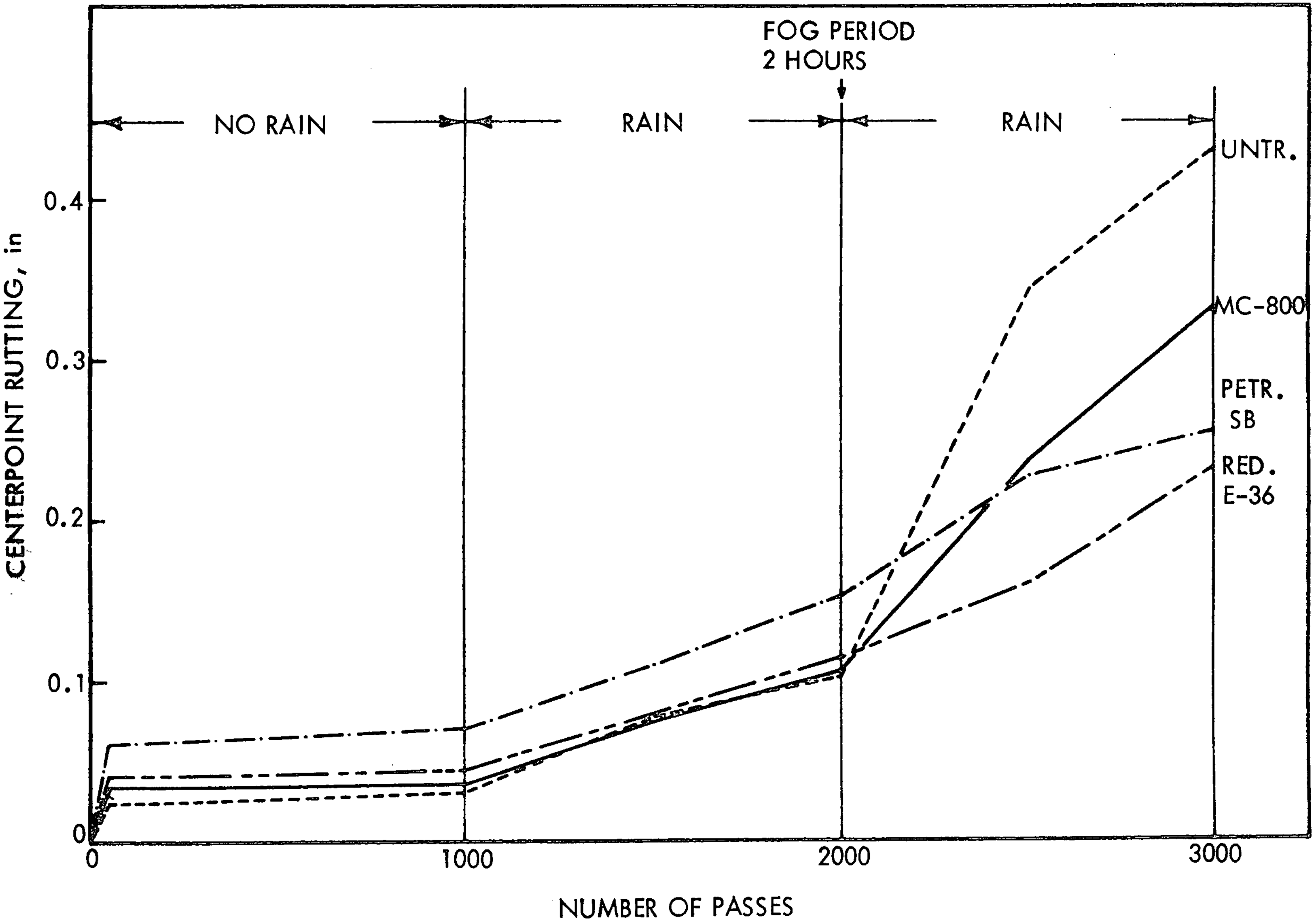


Fig. II-20. Traffic simulator results, average rut depth for each additive, Peneprime excluded.

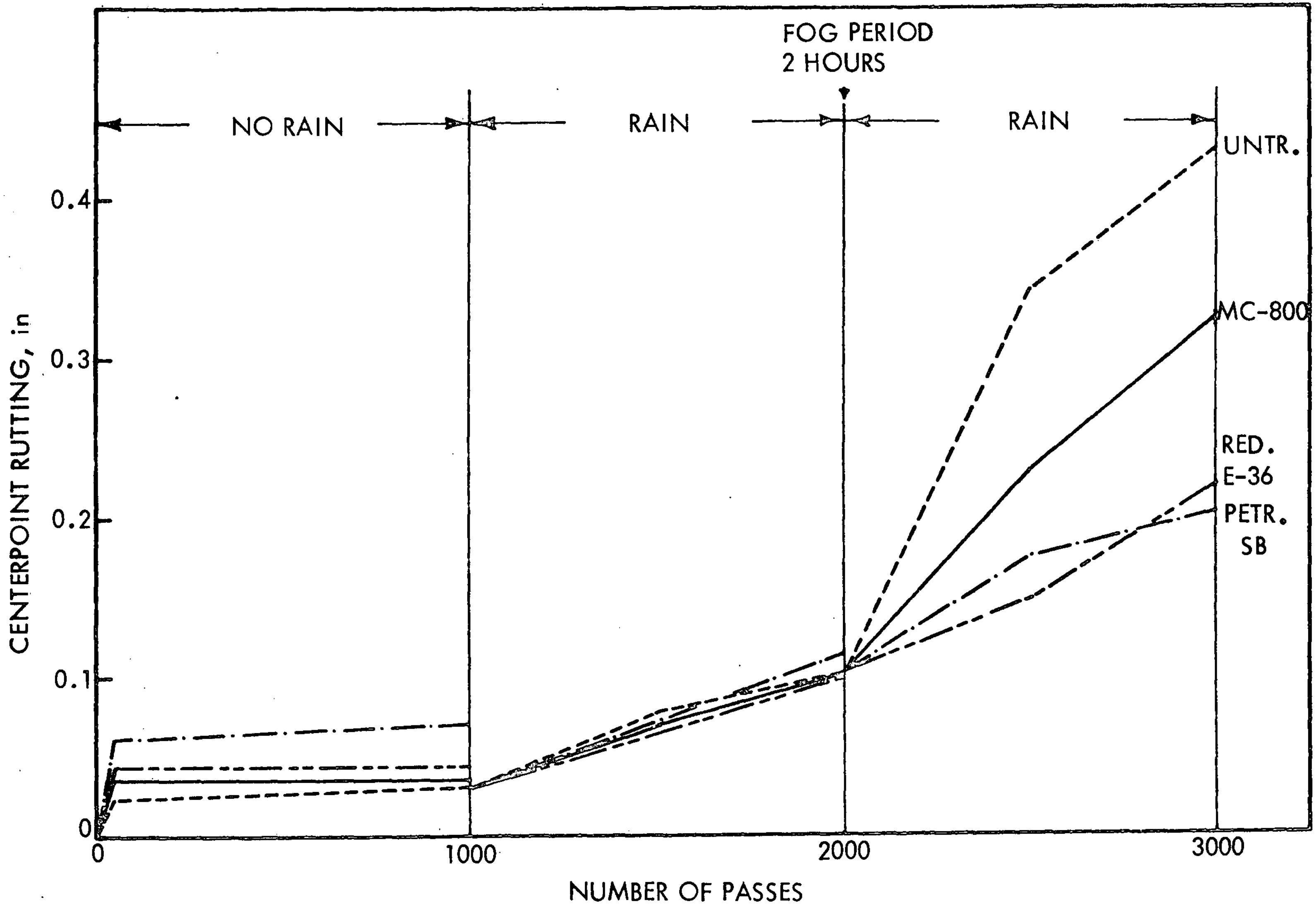


Fig. II-21. Traffic simulator results, average differential rutting rates for each testing period, Peneprime excluded.

which is very similar to Peneprime were not conducted. Semi-Pave treated specimens exhibited no immersed strength in the unconfined compression study and contained a high amount of volatiles (50%). Petroset RB was also not included in the traffic simulator study primarily because of low immersed strength shown in the unconfined study.

Figures II-16-19 present the results of the traffic simulator tests.

For the first testing period of no rain, nearly all treated specimens exhibited higher initial rutting than untreated material. This rutting occurred very early in the tests and in most cases increased very little up to 1000 passes. It was primarily attributed to seating of the specimen in the retaining rings as well as surface densification in the wheelpath.

Density variations of simulator specimens are given in Table II-4 and exhibit nearly the same trend as the unconfined compression specimens. Dry density generally decreased from the untreated condition with increasing additive content. MC-800 cutback specimens had somewhat higher densities probably due to the cutback constituent lubrication during compaction. Referring to Figs. II-16-19 it can be seen that the initial rutting value during the first 1000 passes generally increased with increasing additive content for each additive type. This was probably due to the fact that density decreased with increasing additive content for each additive. MC-800 specimens which had the lowest initial rutting value had higher densities.

Figure II-20 shows the average rut depth for each additive type. Figure II-21 presents the average differential rutting rates for each

Table II-4. Density variation of traffic simulator specimens Bedford limestone material.

Additive	Additive (%)	Water (%)	Approx. liquid[a] content (%)	Wet density (pcf)	Dry[b] density (pcf)
Untreated	–	9.0	9.0	142.0	130.2
MC-800	1.0	8.5	8.8	141.9	130.4
	2.0	8.0	8.5	138.2	127.4
	5.1	6.5	7.8	129.8	120.4
Redicote E-36	1.0	8.5	8.9	135.1	124.0
	3.1	7.5	8.6	132.2	121.7
	4.0	7.0	8.4	128.1	118.2
Petroset SB	1.0	8.6	9.2	133.3	122.0
	3.0	8.0	9.6	134.8	123.0
	5.0	7.2	9.8	131.2	119.4
Peneprime	3.0	7.5	9.1	136.1	124.8
	6.0	6.0	9.1	133.0	121.9

[a] Dry density and % liquids based on the following: MC-800 cutback asphalt @ 25% volatiles; Redicote E-36 cationic asphalt emulsion @ 35% liquid; Petroset SB cationic latex emulsion @ 52% liquid; Peneprime cutback asphalt @ 52% volatiles.

[b] Dry densities are the average of 2 specimens.

testing period. These values were obtained by averaging the rutting values given in Figs. II-16-18 (shown on Fig. II-20) and plotting these from the untreated rutting value at the beginning of each period.

Referring to Fig. II-21, Petroset SB exhibited the highest initial rutting value during the first 1000 passes. In the first rain period (1000-2000 passes) rutting rate was nearly the same for all specimens treated as well as untreated, with untreated material generally exhibiting the lowest total rut depth at the end of this period. From personal observation of both gravel and crushed limestone surfaced roads this action might be expected. Short duration light rains or even short duration heavy rains, generally (if the road is well crowned), do not cause severe surface deterioration in the form of rutting. It is the long drizzling type of rain in which moisture is constantly available for adsorption, aided by traffic action, that is conducive to rut formation as well as potholing of the surface. This action is believed to be indicated by the next period of rain (2000-3000 passes) after a period of 2 hours in which water was constantly available for adsorption by all specimens. From Fig. II-21 the untreated material rut rate in this period was much higher and of a different nature than the treated specimens. Initially there was a heavy period of "tracking" out of fine material followed by a flushing of coarser fines as the rut deepened. At test end, only local shear along the wheelpath was evident and most of the final rut depth for untreated materials was thought to be primarily due to removal of fine material. By contrast the treated materials (at high additive contents) exhibited a much lesser degree of fine material tracking out and flushing, but instead would begin to develop slight local shear along the wheelpath edges which in general turned into full shear "en masse" along the walls of the specimen holder as the test progressed. This action was more evident for the asphalt treated specimens, especially the Peneprime and MC-800 cutback treated material, than for the latex emulsion specimens. This was believed due to the lubricating affect of the asphalt itself for the Redicote E-36 emulsion (which has a water carrier) and a combination of the asphalt and the cutback constituent for the MC-800 and Peneprime.

Influence of the cutback constituent is most strikingly evidenced for Peneprime in Fig. II-19 at 3% and 6% additive contents. The 6%

treated specimens failed during the initial no rain period from full shear. The 3% treated specimens exhibited high initial rutting and failed in full shear at about 2500 passes. This can probably be attributed to the oily cutback constituents and short period of cure time. MC-800 cutback asphalt treated specimens also exhibited full shear at higher additive contents probably due to the cutback constituent. The action of full shear was not nearly as evident for the latex emulsions. This hypothesis was borne out when extrusion of the specimens was attempted after a series of testing was completed. The order of difficulty of extrusion was in direct agreement with type and degree of shear failure. The untreated materials, which normally underwent only local shear, were extremely difficult to extrude, developing very high frictional resistance along the holding ring walls. Peneprime and MC-800 treated specimens, which underwent full shear, by contrast were very easily extruded. A light oily coating was evident on the inside of the ring walls both by feel and beading of water on the surface. The order of increasing difficulty of extrusion was Peneprime, MC-800 cutback asphalt, Redicote E-36 cationic emulsion, Petroset SB latex emulsion and untreated. This corresponds to the following modes of shear failure respectively; full shear, full shear, partial local and full shear, partial full and local shear, local shear. Therefore in evaluating the traffic simulator results the foregoing considerations must be kept in mind. The reasons just given are thought to be the cause of the higher rut depth noted for the MC-800 specimens at high additive contents. This is felt to be due primarily to the oiling effect of the cutback constituent and the short period of air curing before test. The Redicote treated specimens did not exhibit nearly as high a rut depth at 3000 passes (Fig. II-20). The Petroset treated specimens generally yielded lower rut depths at 3000 passes with increasing additive contents.

Initially it was thought that traffic simulator results would indicate an optimum amount of additive with respect to rutting and fine removal (i.e. stability). The interaction of these two factors, coupled with shearing along the walls of the holding rings, however, make it difficult to predict optimum additive contents from this limited study. With respect to simulator results for MC-800 cutback asphalt in Fig. II-18 there is a significant drop in rut depth from the untreated condition to 1% additive content followed by a smaller drop as additive content was increased to 2%. Rut depth for 5.1% additive raised considerably, nearing the untreated rut depth. This same tendency was noted for the Redicote E-36 treated specimens in Fig. II-17. This behavior, for asphalt treated specimens, was thought to be due to the following. At small additive contents slight cohesion is imparted to the material with waterproofing and fine retention ability increased considerably, and overall shearing strength affected very little. As additive content is increased to some optimum value for stability, waterproofing and fine retention ability is again increased with shearing strength becoming affected by lubrication from the asphalt. Past an optimum content for stability, waterproofing and fine retention capabilities are still increasing, however, shearing strength is being reduced substantially.

In viewing the results for Redicote E-36 from Fig. II-17 one might be led to the conclusion that 3% Redicote emulsion provides more stability than 4% Redicote. From Fig. II-18 a similar conclusion might be arrived at for MC-800, that 2% provides much more stability than 5%. These results, from field experience, are very misleading and erroneous and are believed due primarily to the effect of shearing along the walls, previously mentioned, for asphalt treated materials. This is thought to vary with the amount of cutback constituent as well as the asphalt itself. The effect of asphalt, as a lubricant, is probably evidenced in Fig. II-17 between 3% and 4% content for Redicote emulsion. The effect of asphalt and cutback constituent is evidenced in Fig. II-18 between the 2% and 5% for MC-800, a high difference in final rut depth noted. In support of this, Fig. II-16 for Petroset SB, which is not felt to be acting as a lubricant, does not indicate an increase in rut depth with high additive contents but rather an optimum content for stability in the 3-5% range.

Traffic simulator results summarized and presented in Figs. II-20 and II-21 must be viewed with the foregoing considerations in mind. Indications were that Redicote E-36 cationic asphalt emulsion and Petroset SB cationic latex emulsion were superior in ability to resist rutting. MC-800 ranked third for reasons mentioned above. Waterproofing and fine retention, although not evaluated quantitatively, appeared nearly equal for all additives.

Although traffic simulator results were less than desirable from the standpoint of evaluating stability, it is felt that there are valid indications that establishing optimum additive content, with regard to stability, is possible through modification of test procedure and testing conditions as follows:

1. Enlarge test specimen size to 6 or 8 in. in diameter by 4-6 in. deep so that if full shear failure takes place it takes place through the material rather than along holding ring walls.

2. Test only two specimens (at the same additive content) at one time and collect fine material tracked out and "flushed" for a quantitative evaluation of fine retention capabilities.

Summary of Results

1. For untreated material, rut depth at the end of the test is thought to be primarily due to fine material (less than #4 sieve) removal with some local shearing (bulging) along wheelpath edges.

2. For asphalt treated materials (i.e. MC-800, Redicote emulsion and Peneprime) rut depth at the end of a test is due to a combination of light fines removal and full shear along the walls of the holding ring at high additive contents. This is most evident for the cutback asphalt treated specimens in which the oily cutback constituent, as well as the asphalt itself, is thought to be lubricating the holding ring sides. For asphalt emulsion treated specimens full shear was present but less evident, probably due to only asphalt acting as a lubricant.

3. The fact that asphalt treated materials exhibited full shear along the holding ring walls at the higher additive contents precluded establishing an optimum content for stability. Simulator results did, however, appear to adequately differentiate between the various additives but due to the foregoing statement comparisons are difficult and results must be viewed with this in mind.

4. The latex emulsion (Petroset SB) treated specimens exhibited light fine removal, light full shear and some local shear along the wheelpath edges. The elastomers of high tensile strength apparently do not act as lubricants as does asphalt.

5. With the above considerations in mind, simulator results indicated Petroset SB latex cationic emulsion and Redicote E-36 cationic asphalt emulsion were superior to MC-800 in ability to resist rutting, however from observation, waterproofing and fines retention capabilities appeared nearly equal for all three additives.

Conclusions

1. From the unconfined compression study it appeared that the air-cured-24-hours-immersed-24-hours test provides an indication of an additives potential as a waterproofer. It does not, however, define optimum content or appear to provide an adequate indication of stability. The tests conducted with varying periods of air cure all generated nearly the same shape of strength curve for asphalt treated (i.e. Peneprime, Semi-Pave, MC-800 and Redicote emulsion) specimens making evaluation on this basis difficult.

2. Traffic simulator results are thought to give valid indications of both fine material retention and waterproofing, as well as an indication of a materials stability under moving load and imposed environmental conditions. Although the simulator results differentiated adequately between the various additives, optimum additive content for a particular additive was difficult to pinpoint. This was due, mainly, to asphalt treated materials exhibiting full shear along holding ring

walls at higher additive contents. It is felt, however, that defining optimum is possible through modification of test procedure and enlarging sample size to reduce or eliminate shearing along the wall of the specimen holder.

3. On the basis of both the unconfined compression study and the traffic simulator study, Petroset SB cationic latex emulsion, Redicote E-36 cationic asphalt emulsion and MC-800 cutback asphalt appear to be the most effective in providing waterproofing, fine material retention and stability.

RECOMMENDATIONS FOR FIELD TRIALS

Laboratory evaluation of the possible additives to be used in test road field trials indicated that Petroset SB cationic latex emulsion, Redicote E-36 cationic asphalt emulsion and MC-800 cutback asphalt appeared most promising. Cost comparison of the above additives follows:

- Petroset SB @ $1.50 per gallon, F.O.B., Borger, Texas.
- Redicote E-36 @ $0.18 per gallon, delivered.
- MC-800 cutback asphalt @ $0.19 per gallon, delivered.

Prices for the Redicote emulsion and MC-800 cutback were obtained from a local producer. Price for Petroset SB was obtained from a price listing of the manufacturer for bulk delivery.

On the basis of economic considerations, Petroset SB cationic latex emulsion is extremely costly and is not competitive with either MC-800 cutback or Redicote emulsion. To be economically competitive with either of the other two additives it would have to be used in application rates of 0.4% or less. For an additive which primarily functions physically as an adhesive and waterproofer, with very little chemical activity involved, such quantities did not appear sufficient for field applications envisioned in this project.

On the basis of the economic considerations mentioned above, Redicote E-36 cationic asphalt emulsion and MC-800 cutback asphalt were recommended for field trial.

TEST ROAD

The test road, constructed in September 1971, is located between sections 1 and 12 T81N, R13W in Poweshiek County. Topography of the surrounding area is gently rolling with general drainage being north and east to Salt Creek. Predominant soil types are developed on glacial till and loess, being generally of the broad class of silty loams. Figures II-22 and II-23 show the test road, and dust, prior to construction. Maximum elevation difference along the road was estimated to be 30 ft.

According to local residents and the Poweshiek County Engineer, the test road has a history of poor service, including frost boil development, potholing and general spring break-up. The road is subjected to periods of heavy traffic resulting from upstream bridge closures along Salt Creek during spring floodwaters. As a consequence when the road is in its worst condition (spring thaw and rains) it experiences its heaviest traffic. A traffic count conducted in August of 1971 yielded 150 vehicles per day. This count is probably well exceeded at times.

Fig. II-22. Test road, prior to construction, viewed from the west end.

Fig. II-23. Test road, prior to construction, viewed from the east end.

In the spring of 1971 the road had been resurfaced with approximately 1 in. of crushed limestone.

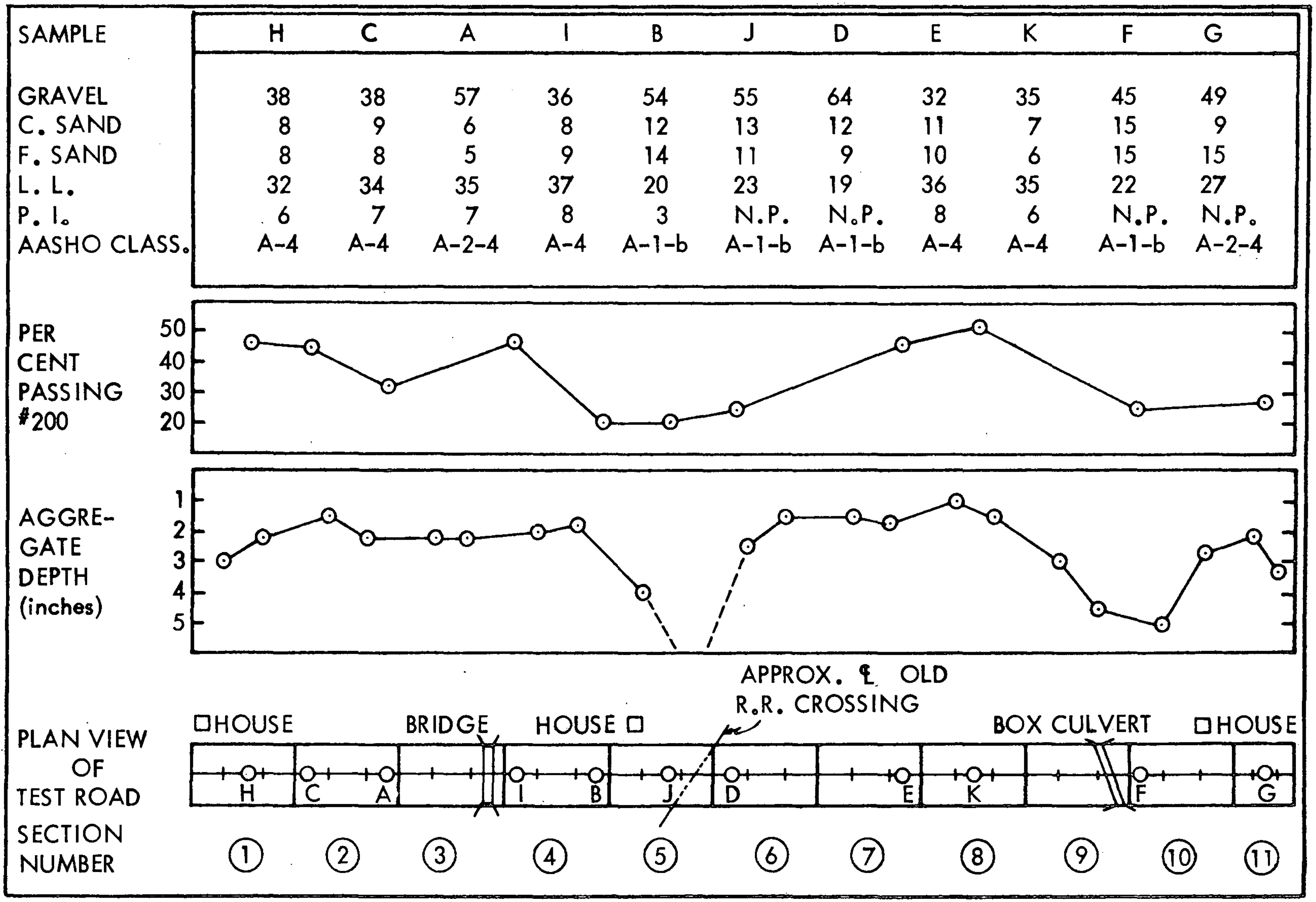
The test road is approximately 1 mile in length and consists of 11 test sections of which 10 are 500 ft long. The last section, which is approximately 300 ft in length, and located on the east end was utilized as one of three control sections. A plan view of the test road showing test section locations and major features is shown in Fig. II-24.

Pre-Construction Testing

Materials

Prior to construction of the test road, numerous samples were taken from the roadbed to a depth of 4 in. Gradation curves and Atterburg limits were determined for these materials and the results shown on Fig. II-24. Sample locations are designated "A" through "K" on the test section layout diagram. Total aggregate depth was determined at the third points of each test section.

Gradation results indicated two distinct groups of materials on the basis of percent passing the #200 sieve. The first group, samples "A", "B", "D", "F", "G" and "J", ranged from 20% to 32% passing the #200 sieve. The second group, samples "C", "E", "H", "I" and "K", ranged from 45% to 52% passing the #200 sieve. The clay content of the second group ranged from 10% to 15% 5-micron clay. All of the first group were relatively nonplastic with the exception of "A" which had approximately 10% 5-micron clay. Figure II-25 shows the gradation curve bands for the two groups of test road materials compared with Iowa State Highway Commission[25] specification ranges for soil-aggregate base materials (specification 4122). It can be seen that both test road



SAMPLE	H	C	A	I	B	J	D	E	K	F	G
GRAVEL	38	38	57	36	54	55	64	32	35	45	49
C. SAND	8	9	6	8	12	13	12	11	7	15	9
F. SAND	8	8	5	9	14	11	9	10	6	15	15
L. L.	32	34	35	37	20	23	19	36	35	22	27
P. I.	6	7	7	8	3	N.P.	N.P.	8	6	N.P.	N.P.
AASHO CLASS.	A-4	A-4	A-2-4	A-4	A-1-b	A-1-b	A-1-b	A-4	A-4	A-1-b	A-2-4

Fig. II-24. Plan view of test road sections and pre-construction field test results.

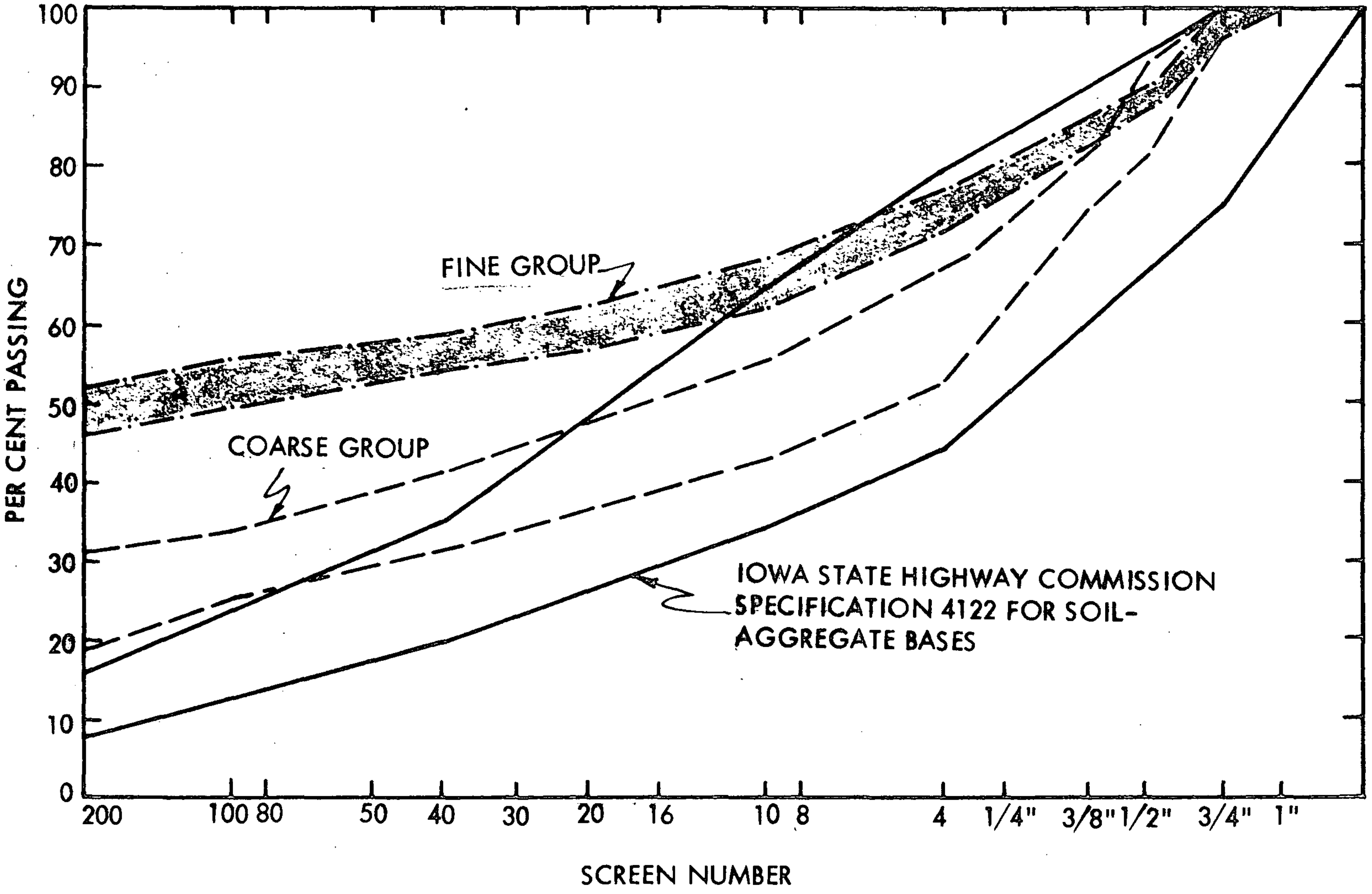


Fig. II-25. Gradation characteristics of test road materials.

material groups fall above the specification range on the fine end of the gradation curves.

Two combinations of materials were made within the first group, namely "C, E" and "H, I, K". One combination "F, G, D" was made for the coarse group. Optimum moisture-density curves were established for these combinations and the results given in Table II-5.

Table II-5. Optimum moisture-density results. Poweshiek County test road materials.

Material combination	Optimum moisture (%)	Maximum dry density (pcf)
F, G, D	7.7	133.5
H, I, K	10.7	123.9
C, E	11.7	121.1

Laboratory Tests

Unconfined compression tests as well as limited traffic simulator tests were conducted on the previously noted combinations of test road material for additive evaluation as well as comparison with results using Bedford limestone.

Unconfined compression tests were performed on all three combinations of materials (i.e. "C, E"; "H, I, K" and "F, G, D") at two testing conditions of 24 hours air cure and 24 hours air cure — 24 hours immersed. The unconfined study on Bedford limestone indicated that testing at different periods of air cure generated nearly the same strength curves. Therefore, for this material only one air cure period was used. Material supplies were limited and only 2% and 4% additives contents were used with the exception of combination "H, I, K" where 2%, 4% and 6% was used. The 2% and 4% tests, however, define the trend. Results of the unconfined tests are summarized in Figs. II-26 and II-27. All additive contents referred to herein are on the basis of total amount of additive.

For the 24-hour air cured specimens, Fig. II-26, untreated strength for materials "F, G, D", "H, I, K" and "C, E" were 1400, 1300 and 850 respectively, considerably higher than the 650 lb noted for Bedford limestone specimens and the 750 lb noted for the loess specimens at the same cure time. The general shape of the curves were nearly the same as the Bedford material with decreasing strength as additive content increased. The MC-800 treated specimen strengths were considerably lower than those of Redicote treated specimens, a reversal from the Bedford study.

Results of specimens air cured 24 hours and immersed 24 hours are presented in Fig. II-27. MC-800 treated specimens generated about the same strengths as for the Bedford material. Redicote E-36 treated strengths were much lower than the MC-800 specimens of the test road material as well as being much lower than Redicote E-36 treated strengths for the Bedford material. The amount of material passing the #200 sieve for materials "F, G, D", "H, I, K" and "C, E" was 26%, 48% and 46% respectively. This indicates that the waterproofing capabilities of Redicote E-36 may be affected considerably by the amount of fine material present as was also noted in the loess study. The waterproofing capability of MC-800 seems to be affected very little.

Dry density variations of unconfined compression and traffic simulator specimens are given in Tables II-6 and II-7 with the same

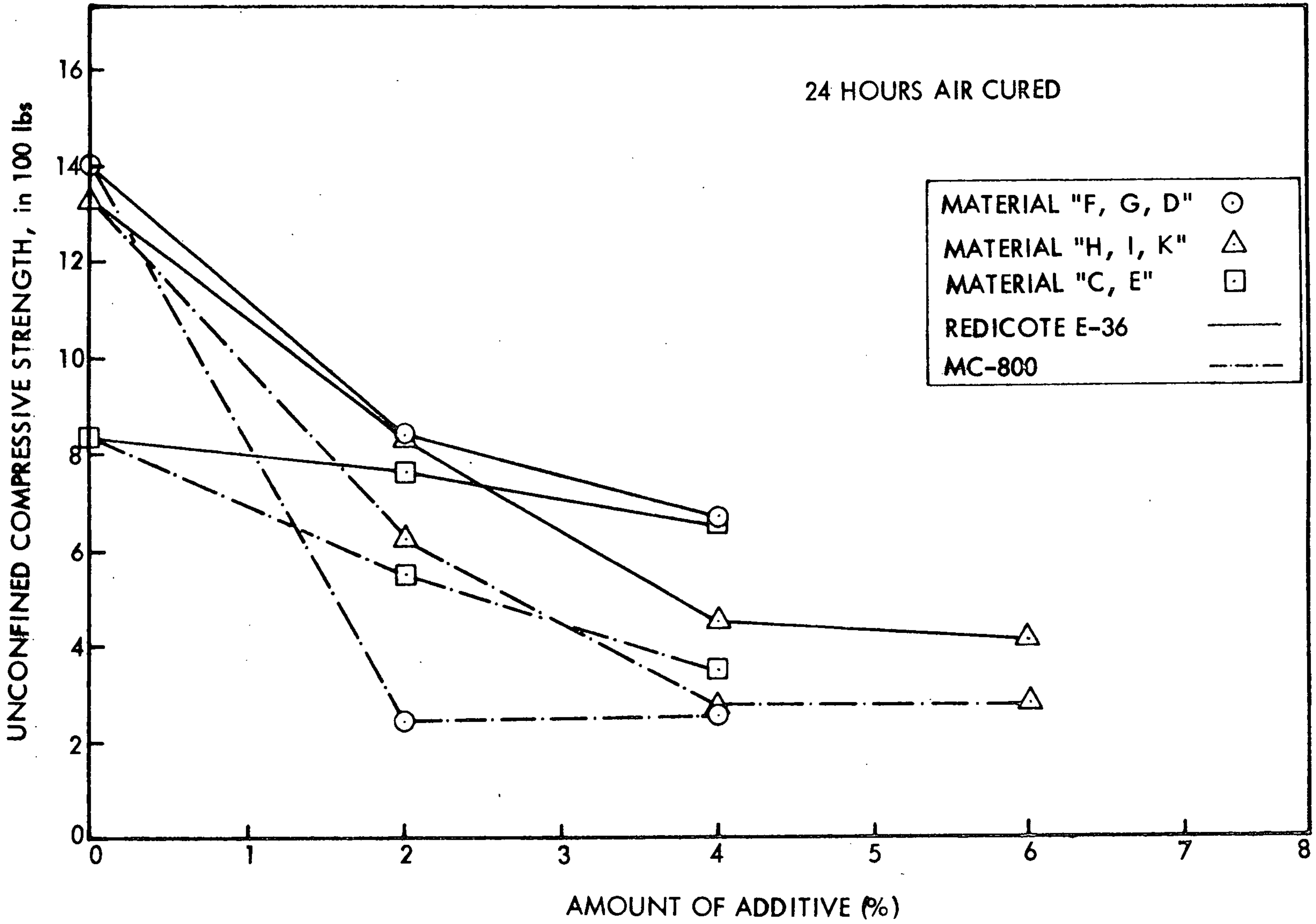


Fig. II-26. Unconfined compression test results, Poweshiek County test road material.

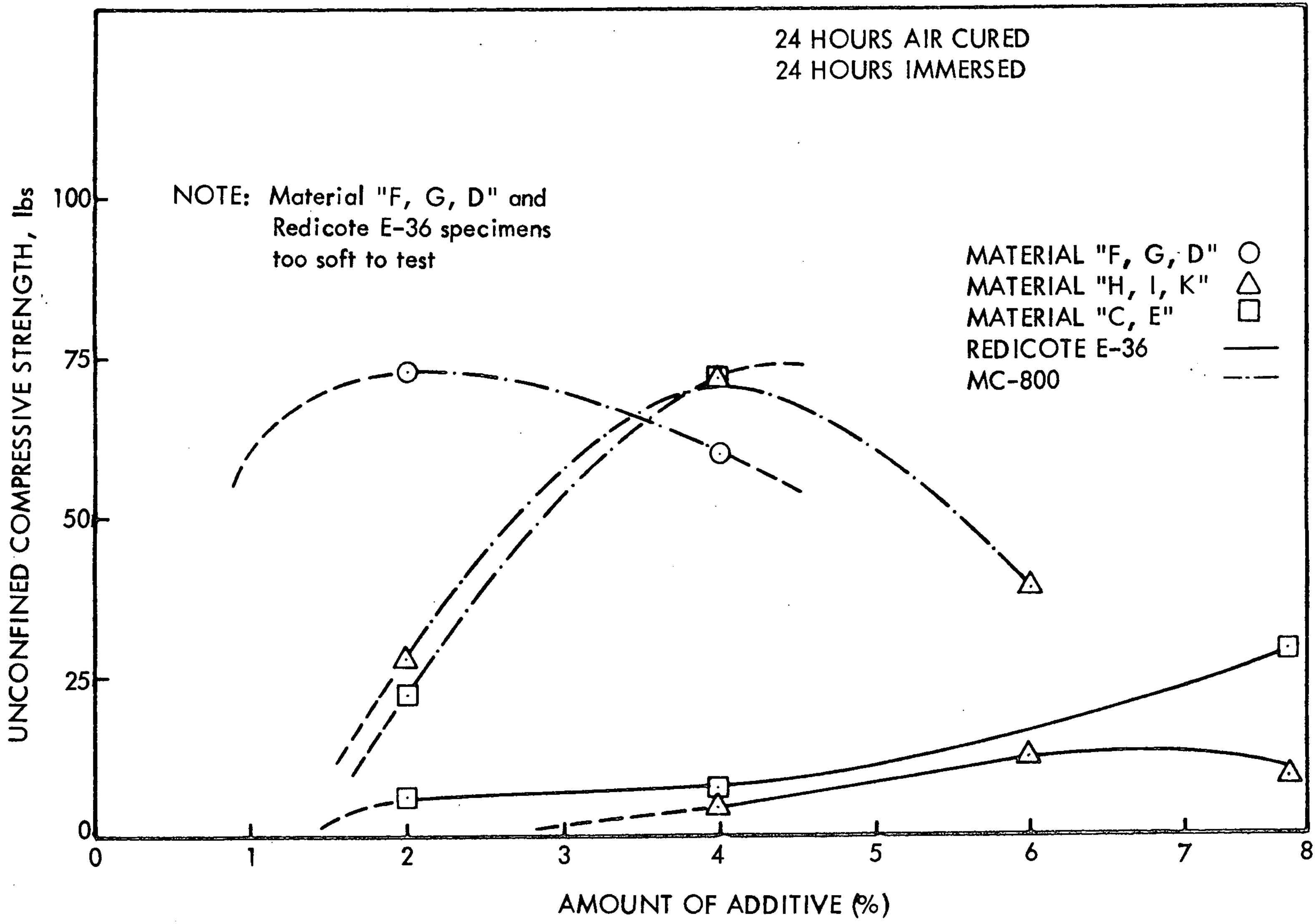


Fig. II-27. Unconfined compression test results, Poweshiek County test road material.

Table II-6. Density variations of unconfined compression specimens, Poweshiek County test road material.

Material	Additive	Additive amount (%)	Approx.[a] liquid cont. (%)	Wet density (pcf)	Dry[b] density (pcf)
"F, G, D"	Untreated	–	9.0	141.8	131.3
	MC-800	2.0	9.0	137.7	127.5
		4.0	9.0	138.1	127.8
	Redicote E-36	2.0	9.0	135.1	125.1
		4.0	9.0	133.4	123.5
		8.0	9.0	131.7	118.6
"H, I, K"	Untreated	–	11.0	136.1	122.6
	MC-800	2.0	11.0	136.5	122.9
		4.0	11.0	132.2	119.1
		6.0	11.0	132.3	119.2
	Redicote E-36	2.0	11.0	135.0	121.7
		4.0	11.0	131.8	118.7
		6.0	11.0	128.2	115.5
"C, E"	Untreated	–	12.0	139.0	124.1
	MC-800	2.0	12.0	135.4	120.9
		4.0	12.0	132.4	118.2
	Redicote E-36	2.0	12.0	135.1	120.7
		4.0	12.0	134.7	120.3
		8.0	12.0	129.6	115.7

[a] See Table II-4.

[b] Dry densities are the average of all specimens.

trend of decreasing density with increasing additive content noted for Bedford limestone and 20-2 loess.

Traffic simulator data for the test road materials are given in Figs. II-28 and II-29. Due to limited material quantities material "A" was used to represent the low fine content material (32% passing

Table II-7. Density variations of traffic simulator specimens, Poweshiek County test road material.

Material	Additive	Additive amount (%)	Approx.[a] liquid cont. (%)	Wet density (pcf)	Dry[b] density (pcf)
"H, I, K"	Untreated	–	11.0	134.4	121.1
	MC-800	4.0	10.0	131.0	119.0
	Redicote E-36	4.6	10.3	130.3	118.2
"A"	Untreated	–	8.0	138.6	128.3
	MC-800	3.5	7.2	132.1	123.3
	Redicote E-36	4.4	7.5	132.0	122.8

[a] See Table II-4.

[b] Dry densities are the average of all specimens.

the #200 sieve). Material "H, I, K" was used to represent the high fine content material (49% passing the #200 sieve). Several general observations can be made. The untreated material of high fine content failed much sooner than that of low fine content. A more definite indication of waterproofing and stability capability of the additives was noted for material containing the higher amount of fines. Relative rutting of treated specimens for the test road materials was somewhat higher at the end of testing than for Bedford limestone materials. MC-800 treated specimens performed somewhat better than Redicote treated specimens, a reversal from the Bedford material study.

Simulator results for material "A", which contains about 32% material passing the #200 sieve, approximate the results for Bedford limestone material. Traffic simulator results should be viewed,

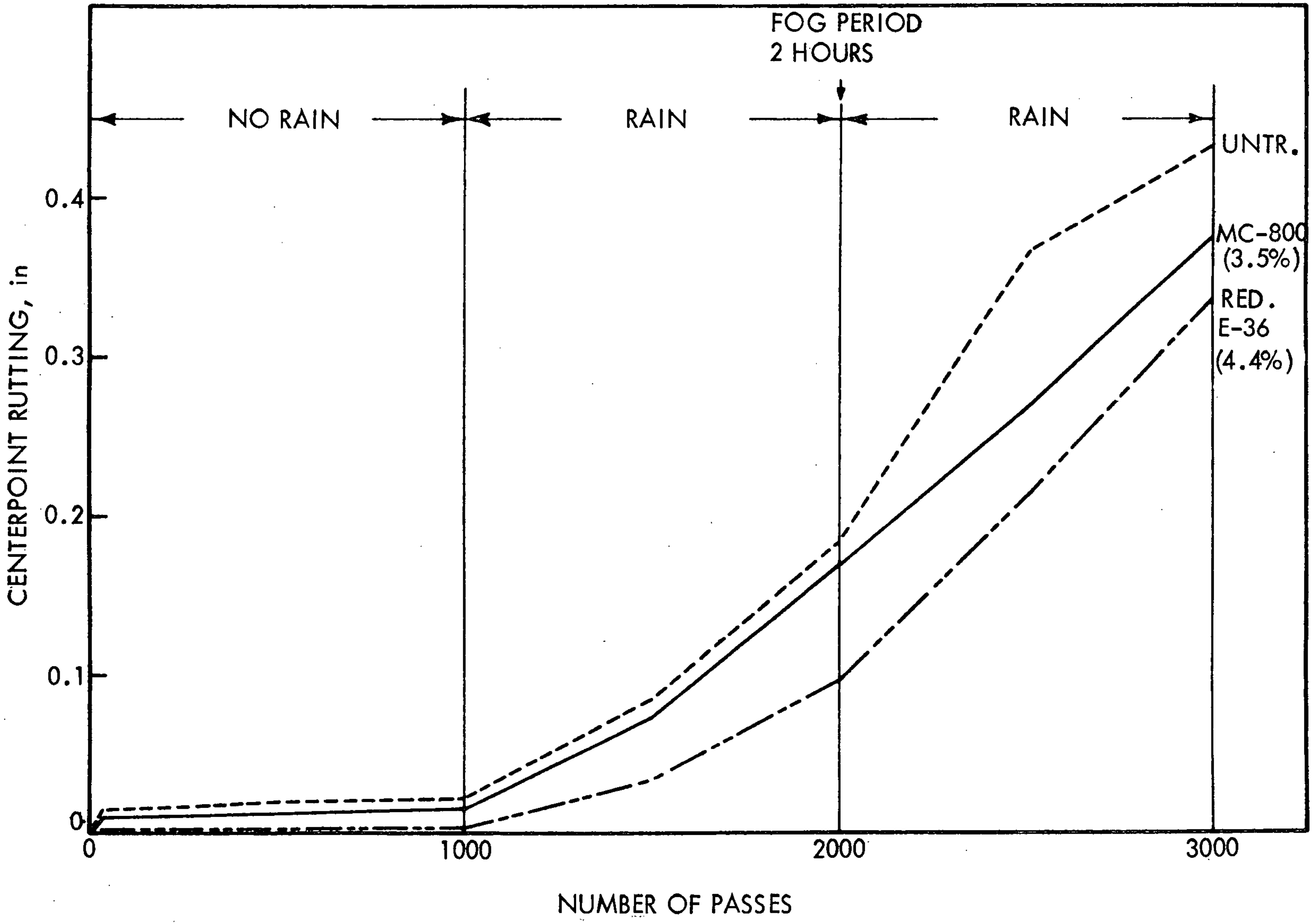


Fig. II-28. Traffic simulator results. Poweshiek County test road material "A" (32% passing #200 sieve).

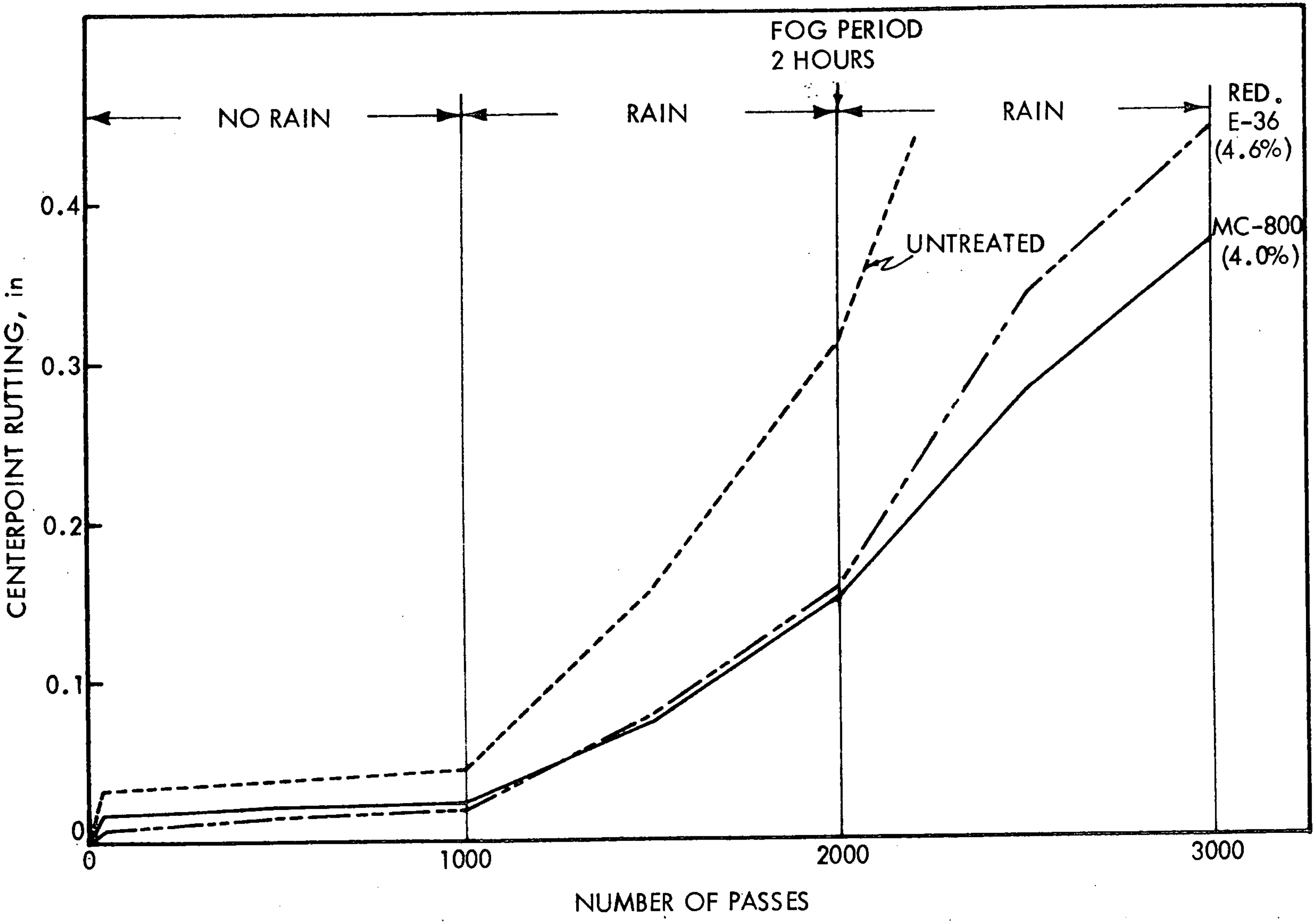


Fig. II-29. Traffic simulator results. Poweshiek County test road material "H, I, K" (49% passing #200 sieve).

however, with the considerations mentioned for the behavior of asphalt treated specimens in the Bedford material evaluation kept in mind.

Overall evaluation of the unconfined compression results and traffic simulator results indicate the benefit of addition of both additives with respect to waterproofing, fine retention and stability under imposed environmental conditions.

A very observable difference in the test road materials compared with Bedford limestone material was the amount of specimen shrinkage after the air curing period. This was unnoticeable for the Bedford material but was easily observable for the untreated test road materials by shrinkage away from holding ring walls. For treated specimens this shrinkage was substantially reduced. This may indicate the presence of montmorillonitic clays. Material combination "H, I, K" which contained the highest amount of fines evidenced this behavior to the greatest degree. Mixing and color change characteristics appeared about the same for both MC-800 and Redicote emulsion with the Poweshiek materials as well as Bedford limestone. There were no observable differences noted during the mixing operations other than that the Redicote emulsion treated materials remained very friable and easily handled after additive incorporation. MC-800 treated material tended to become more cohesive in nature. Redicote emulsion also appeared to coat coarse aggregate somewhat better than the MC-800. Examination of the specimens after testing in the simulator and following a short period of air drying, however, a difference was noted on the sides of the samples. Referring to Fig. II-30 it can be seen that there is a slight difference in surface appearance between the emulsion and MC-800

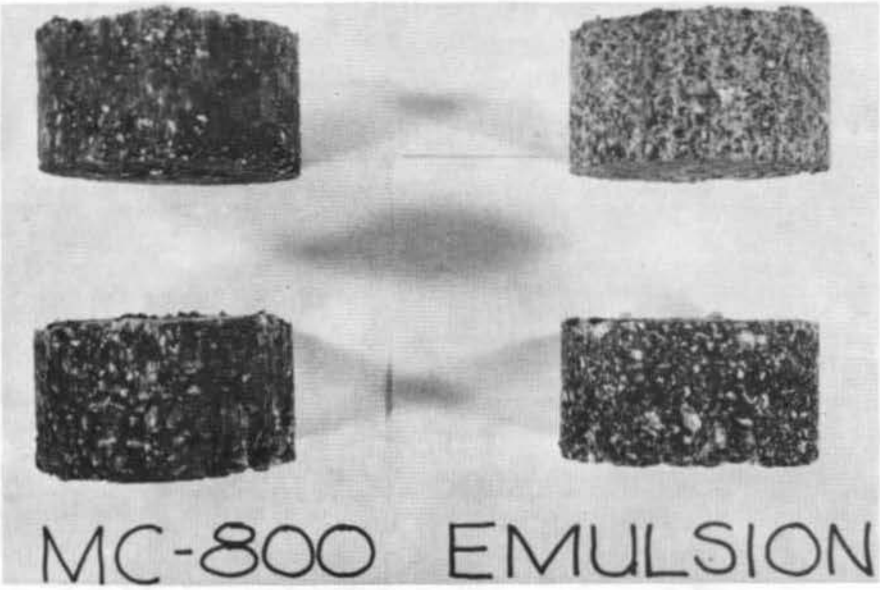


Fig. II-30. Surface characteristics of simulator specimens for Bedford limestone (bottom) and test road material (top).

for the Bedford material (bottom). For the test road materials (top) however, the emulsion treated specimens appeared to be "peppered" with globules of asphalt. Residual asphalt content for all of the specimens in Fig. II-30 was approximately 4%.

Figure II-31 shows the reaction of MC-800 and Redicote emulsion to a wet and dry limestone surface. Approximately the same amount of additive was poured on each limestone surface from about the same height. Temperature of the Redicote emulsion and MC-800 was 150 °F and 275 °F respectively. The need for water as a dispersing agent is readily apparent for the emulsion. This spreading reaction for the emulsion takes place almost immediately with the asphalt being "plated" out very quickly. Note the balling effect of the emulsion similar to MC-800 for the dry block.

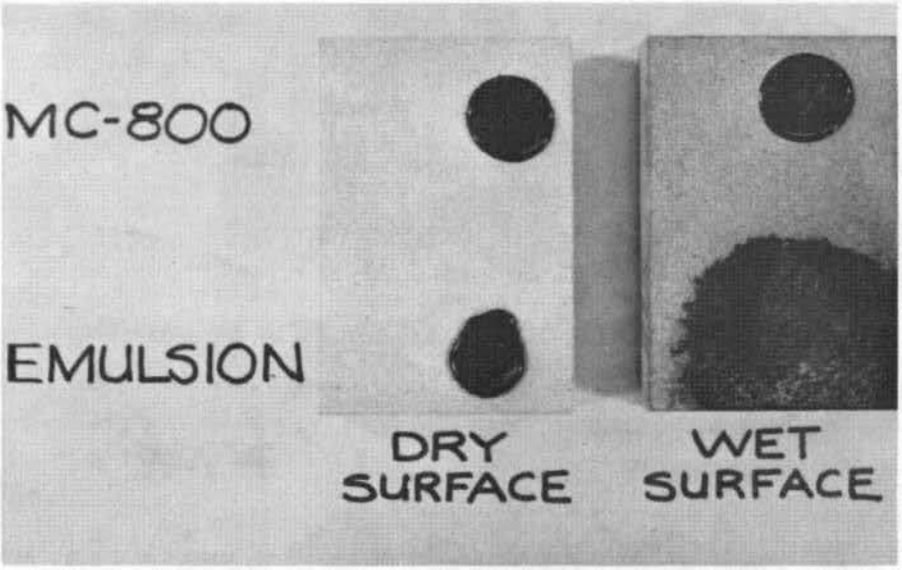


Fig. II-31. Spreading tendencies of MC-800 and Redicote emulsion on a limestone surface.

Field Tests

Prior to test road construction in-place field tests were conducted consisting of spherical bearing value (SBV) tests, Benkelman beam tests, and in-place density tests.

The SBV test is shown being conducted, using a 6-in. diameter sphere, in Fig. III-34 (Part III Final Report). It was used to determine the relative bearing capacities of the various sections and has been shown to have far better reproducibility than either CBR or plate bearing tests[26].

Deflections of the road surface were determined with a Benkelman beam test (see Fig. III-33, Part III Final Report) under a 17,280-lb rear axle load with the test truck moving. Deflections were then converted to a "stiffness" factor[27] by dividing the wheel load in thousands of pounds by the maximum deflection.

In-place density tests were conducted using the rubber-balloon method in accordance with ASTM designation D2167-66[5].

Benkelman beam and SBV tests were conducted at the third points of each test section. One in-place density test was conducted on each test section for the pre-construction tests. Results of the pre-construction tests will be presented later and compared with post-construction test results.

In general, all tests reflected the depth of aggregate at each point tested. Thick aggregate depths resulted in high spherical bearing values (SBV), high Benkelman beam stiffness, high dry density and low moisture content. The reverse was generally true for shallow aggregate depths. At the point where an old railroad crossing had been, Fig. II-24, aggregate depth was in excess of 7-1/2 in. A very high SBV value was noted here, as well as a relatively high Benkelman beam stiffness and high density.

Dust collectors were installed, north and south of centerline, in section 6. These devices were installed approximately 3 ft above existing ground level and were fabricated to be relatively simple yet conform, as nearly as possible, to ASTM designation D 1739 for collection and analysis of dust fall. The containers were half filled with distilled water and remained in place, for a period of 21 days, from Aug. 13 to Sept. 3. Collectors were checked and refilled with distilled water weekly. At the end of the collection period the containers were sealed, brought to the laboratory and the excess water allowed to evaporate in low-temperature ovens. Upon completion of evaporation the dust was brushed and washed into a small clean beaker that had been heated, to remove surface moisture, cooled in a desiccator and tared on a balance capable of weighing to 0.0001 of a gram. Insects, chaff, seeds, and other large contaminates were removed and a solution of 70 ml, 0.3N hydrogen peroxide, 30 ml of distilled water and 3 drops of hydrochloric acid was added to the beaker to dissolve the remaining organic matter. The beaker was then placed in an oven, allowed to evaporate, cooled in a desiccator and weighed. Results of the dust collection are presented in Fig. II-32. Data points encircled with dashed lines on the north side are corrected points. In preparing the solution for dissolving organic matter (for the samples collected north of the road) an inordinate amount of hydrochloric acid was used, resulting in a white deposit being formed on evaporation. This was thought to have been calcium chloride resulting from a chemical reaction between hydrochloric acid and calcium carbonate present in the dust. Corrected values were obtained by averaging the five weights of dust from 100-500 ft on both sides, obtaining the difference and subtracting this difference from each actual recorded weight obtained for the samples on the north side of the road. Volumetric measurements of dusting have previously been reported[33].

Referring to Fig. II-32, a very rapid nearly straight line drop-off of dust is noted from the shoulder of the road out to 30-40 ft followed by a more gradual, curved drop, out to approximately 150 ft. From 150 to 500 ft the dust deposition rate is again nearly a straight line with a very gradual decrease noted. Figure II-33 compares this data with dust collection data from a limestone surfaced road located near the Iowa State Center at Iowa State University in Ames, Iowa.

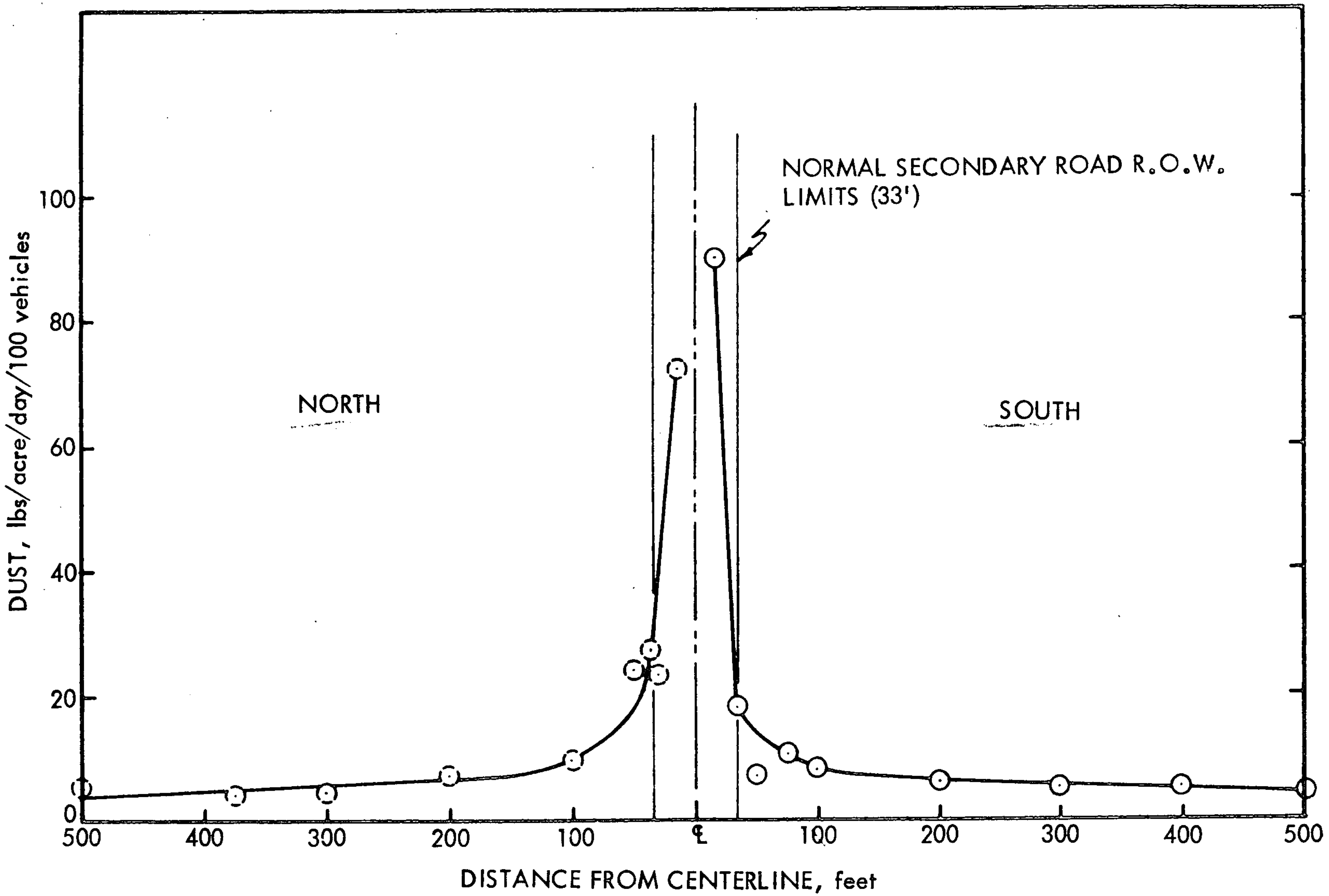


Fig. II-32. Dust collection results, Poweshiek County test road, crushed limestone surface.

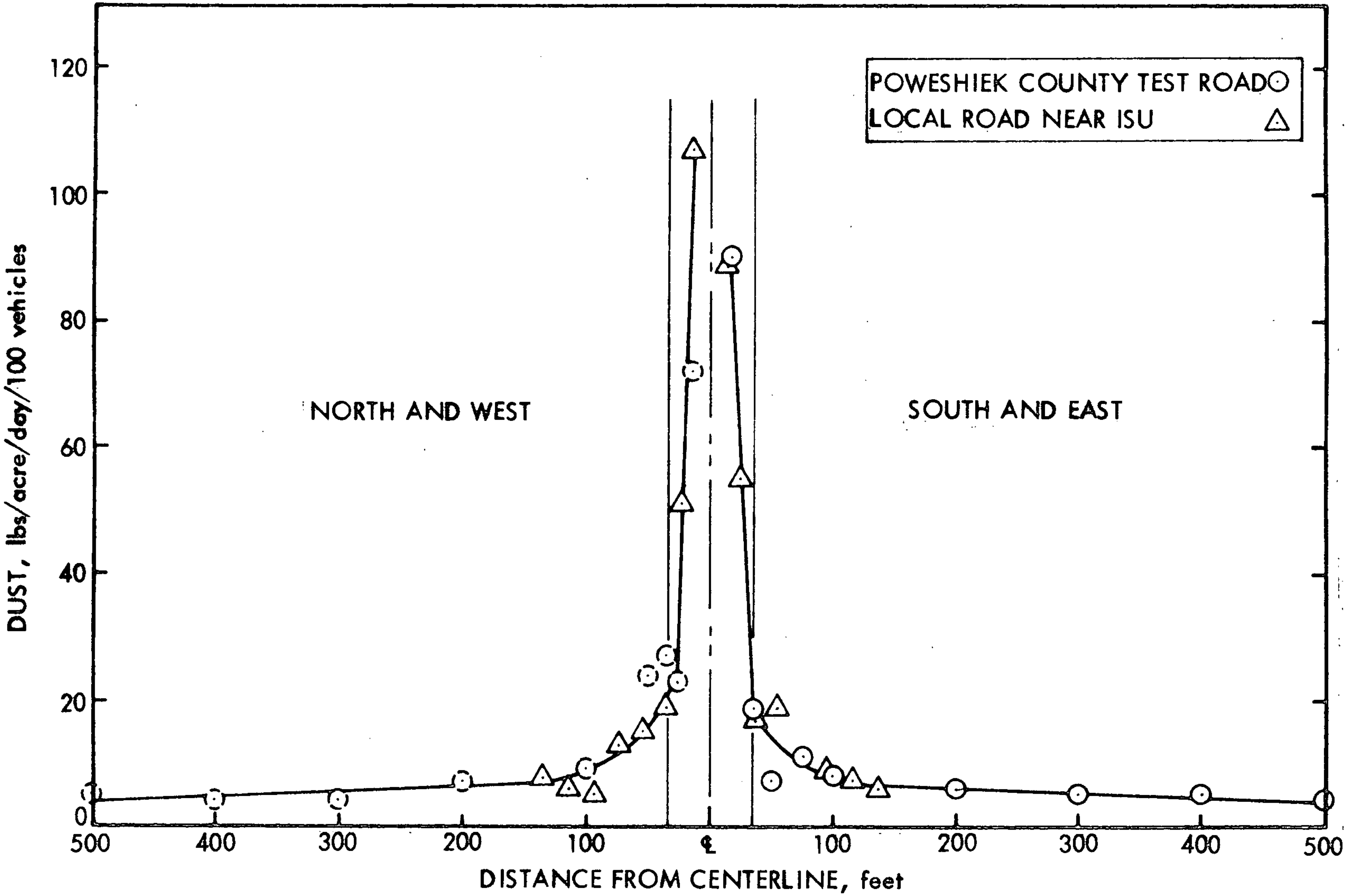


Fig. II-33. Dust collection results, crushed limestone surfaced roads.

Traffic on this road was 110 vehicles per day. For comparative purposes all data was placed on the basis of 100 vehicles per day. It can be seen that the curves generated are nearly identical. This indicates, for limestone surfaced roads, the amount of dust deposited at a particular distance from centerline is proportional to the amount of traffic the road carries and may be estimated if the traffic count is known. Dust collection results indicated that the bulk of the dust deposition occurs at less than 150 ft on either side of centerline, the major portion being deposited within the normal secondary road right-of-way limits (usually 33 ft). Beyond 150 ft a nearly constant deposition rate is indicated out to at least 500 ft. Sieve analysis of a portion of the collected samples south of the test road yielded the following. Near the shoulder of the road approximately 30% of the collected dust was retained on the #200 sieve. At 50 ft from centerline 15% was retained. At 75 ft 2% or less was retained, nearly all the dust being smaller than the #200 sieve.

Figure II-34 presents the estimated amount of road material lost, in the form of air borne dust, per mile, per year at a traffic count of 100 vehicles per day. These estimates were calculated using the data presented in Fig. II-33 and the following assumptions:

1. Right-of-way width of 66-ft and 24-ft roadway width. This gives a deposition width of 42 ft within the right-of-way.

2. Approximate average deposition rates for each area from Fig. II-33.

   (a) Within right-of-way = 55 lb/acre/day/100 vehicles

   (b) From 33 ft to 150 ft from centerline = 10 lb/acre/day/100 vehicles

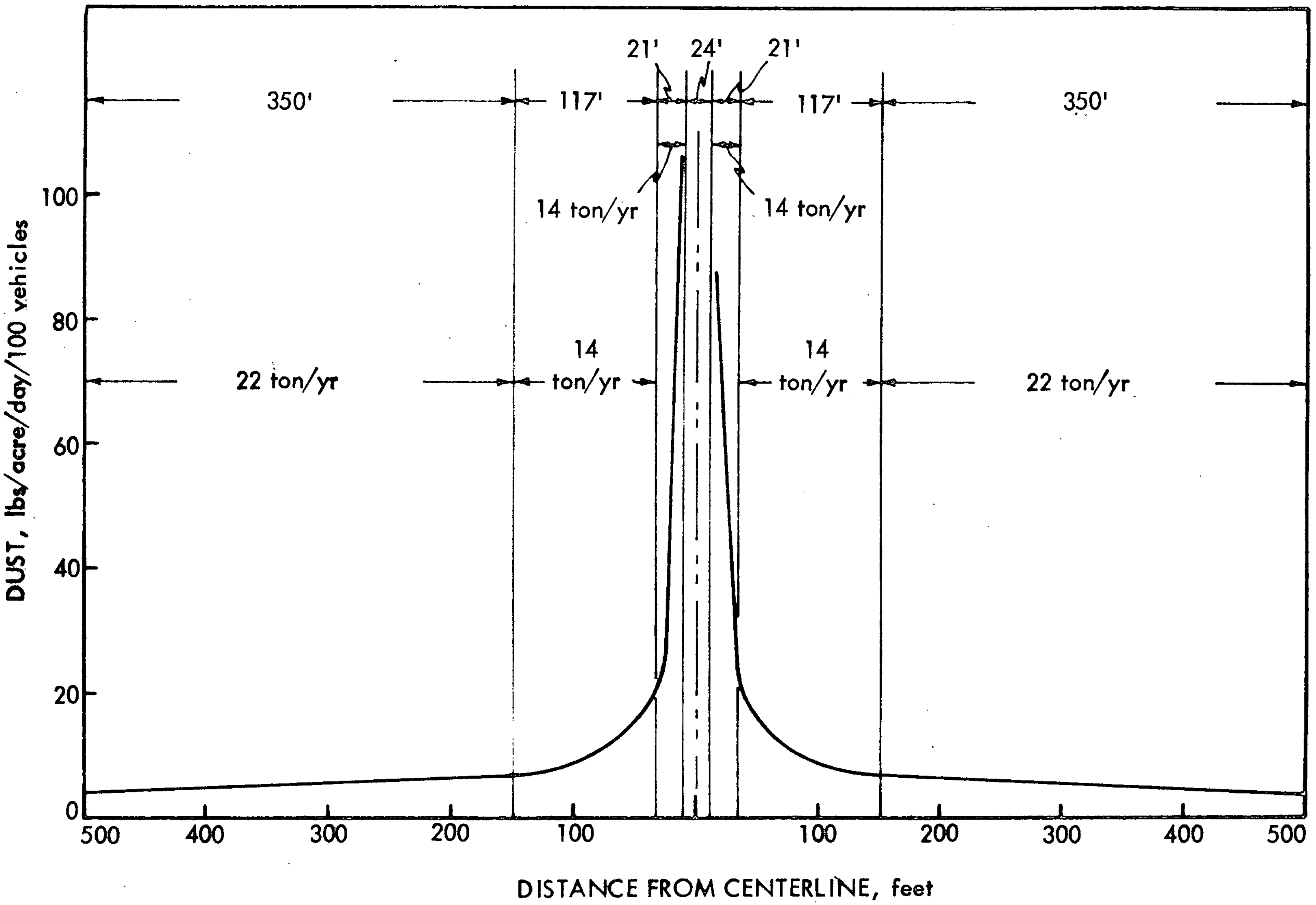


Fig. II-34. Estimated road material loss, in the form of airborne dust, for limestone surfaced roads at 100 vehicles per day.

(c) From 150 ft to 500 ft from centerline = 5 lb/acre/day/ 100 vehicles

3. 200 days/yr conducive to dusting.

It should be realized that the data shown in Fig. II-34 are estimates only, having been extrapolated from relatively small sampling area and amount of collected dust. Summing the amounts for the various sections indicates that approximately 100 tons per mile of road material may be lost per year at a traffic count of 100 vehicles per day. This loss takes place over a 1000-ft corridor, 500 ft on either side of centerline. Approximately 28 tons/mile/year is lost within normal right-of-way limits and another 28 tons/mile/year is lost from 33 ft — 150 ft either side of centerline. A Purdue study in 1961[28] indicated annual losses of road material, in the form of dust, to be about 73 tons/mile for unpaved roads at 100 vehicles per day. It is not known, however, over what corridor width or to what type of surface (gravel or limestone) this value was related.

Test Section Layout

The test section layout showing the location of each section, additive type and percentage, and control sections is shown in Fig. II-35 and Table II-8.

The range of additive amounts to be used for each additive was set at 2%, 3% and 4% residual asphalt content. These percentages were selected on the basis of laboratory data and present application rates being used in Pottawattamie County, Iowa. The primary purpose in using the rates selected was to span the range of applications presently

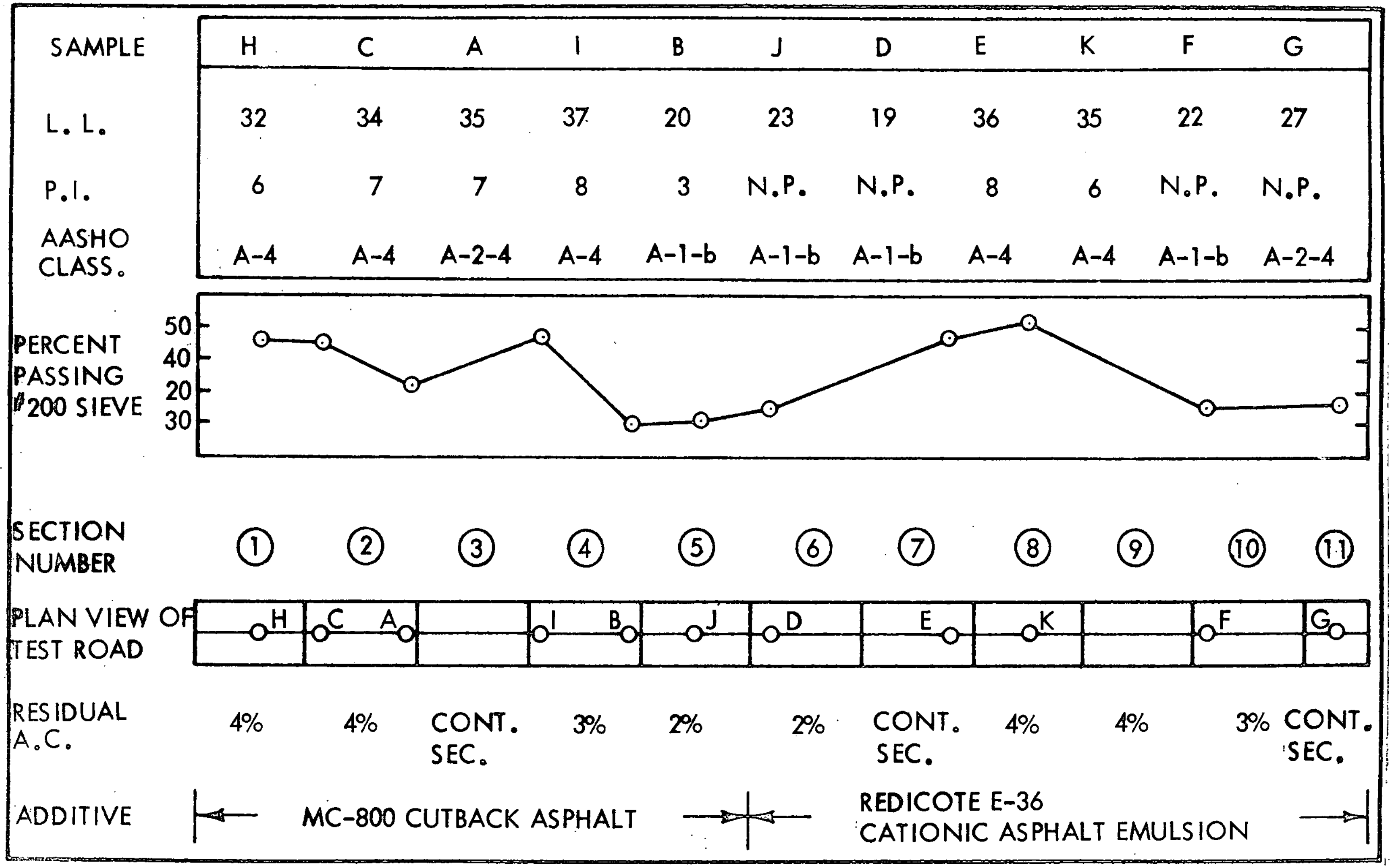


Fig. II-35. Poweshiek County test road — test section layout.

Table II-8. Test road sections and treatments.

Section number	Length (ft)	Additive	Residual A.C. by dry soil weight (%)	Moisture control method[a]
1	500	MC-800	4	A
2	500	MC-800	4	B
3	500	Control section	—	—
4	500	MC-800	3	A
5	500	MC-800	2	A
6	500	Redicote cationic emulsion	2	A
7	500	Control section	—	—
8	500	Redicote cationic emulsion	4	A
9	500	Redicote cationic emulsion	4	B
10	500	Redicote cationic emulsion	3	A
11	300±	Control section	—	—

[a] A — Moisture to be controlled at the most desirable mixing content. This may entail drying or additional water.

[b] B — Moisture as existing in roadway at time of construction.

being used from a minimum amount up to, but not exceeding, an optimum amount. Field experience in Pottawattamie County indicated that considerable rutting and shoving was experienced when the residual asphalt content, using MC-800, approached 5%. For soil-aggregate construction using in-place materials, the amount of material passing the #200 sieve can be expected to be high after incorporation of the subgrade material and consequently rutting could be a problem at higher residual contents.

In using the range of additives selected, it was desired to determine optimum content with regard to dust palliation and surface improvement as well as to establish guidelines and potential specifications for future construction.

All additive contents referred to herein are on the basis of residual asphalt content [assuming Redicote E-36 emulsion at 65% solids (asphalt) and MC-800 at 75% solids] unless otherwise noted.

The MC-800 cutback sections and Redicote cationic emulsion sections were located on the west and east ends, respectively, for ease of construction and to provide for smooth transitions between sections. Percentage of additive used in each section was based primarily on (a) amount of − #200 material (e.g. high additive contents were placed in sections of high − #200 material), and (b) AASHO classification (e.g. high additive contents were used where A-4 soils were encountered). A-4 soils are susceptible to capillary saturation and frost heave. In addition an attempt was made to have sections containing the same percentage of different additives placed in sections with similar soils. This becomes difficult in a short test road having a high variability of soils but was attempted in order to evaluate comparative performance of additives at the same residual content.

It can be seen on Fig. II-35 and Table II-8 that there are two sections containing 4% residual asphalt cement for both the MC-800 cutback and Redicote E-36 emulsion. One 4% residual section, for each additive, was to be mixed at the existing field moisture content of the soil-aggregate. The remaining 4% residual sections were to be placed in the same manner as the rest of the sections with the natural water content adjusted, if needed, to the most desirable mixing content as determined in the field. Water is normally needed in soil-aggregate mixtures with asphalt for two reasons: (a) to facilitate dispersion of the asphalt throughout the mixture, and (b) to aid compaction by acting as a lubricant. In the case of the cationic emulsion, water is also needed to provide a charged surface on aggregate particles. Adding water is, however, costly and time consuming. Pottawattamie County, in its program of MC-800 cutback road mixing, does not add water but utilizes existing field moisture, with apparently fair results. Their application rate is in the 3-4% residual range. Thus, it was desired to evaluate the field performance of sections having the same amount of residual asphalt but placed at different moisture contents.

Three control sections, numbers 3, 7 and 11, were provided because of the high variability of aggregate depths of 2-1/4 in., 1-1/2 in. and 3 in. respectively. Performance of these untreated sections was evaluated to provide some indication of the affect of varying aggregate depth and for comparison with treated sections performance with regard to dust palliation and surface improvement.

Construction

Construction Method

The test road was constructed in September 1971 using Poweshiek County personnel and equipment. Construction method and procedure is given in Table II-9. This was based on present construction methods being used in Pottawattamie County for in-place road mixing using MC-800 cutback asphalt and previous experience in the construction of similar test roads, with other additives, in Linn and Clinton Counties.

Table II-9. Proposed construction method, Poweshiek County test road.

1. Scarify and pulverize the top 4 in. of the existing roadbed.
2. Blade the material into two equal windrows, one on each shoulder.
3. Compact the subgrade trench with a sheepsfoot or tamping foot roller.
4. Blade both windrows into the center to uniform thickness. Apply water, if needed, in accordance with Table II-8. Blade into two equal windrows near each shoulder and mix thoroughly. Repeat process, if needed, until desired moisture range is reached.
5. Blade the mixture into the center to uniform thickness. Apply additive. Blade into two equal windrows near each shoulder and mix thoroughly. Repeat process, if needed, until desired additive content is reached and a homogeneous mixture is obtained.
6. Blade mixture to one side. Begin blading 1 to 2 in. lifts across roadbed while rolling with a 75-80 psi pneumatic roller. Shape to a 4-in. crown.
7. Tight blade surface prior to finish rolling.

Construction Problems

Several problems arose during the construction process which required field adjustment of the foregoing procedure (see Table II-9). These problems will be discussed along with recommendations for possible future construction.

Construction began at the west end of the test road, with the MC-800 cutback sections and proceeded easterly, the Redicote emulsion sections being constructed last. Equipment consisted of 3 blade graders, 1 Seaman mixer, a tractor-drawn rubber-tired roller and sheepsfoot roller. A 2000-gallon water truck was available but had to be called from another job when needed.

Initial scarification was accomplished by one blade grader equipped with a scarifier attachment. Procedure on the first sections was to scarify to the full 4-in. depth with several passes followed by a blading of the scarified material into windrows and mixing with the Seaman. On scarifying to the full 4-in. depth; however, large chunks and strips of the fine grained subgrade material were "plowed" up. After several passes of the Seaman, and blade mixing, the strips and chunks were reduced in size but the mixed material contained many lumps and balls of fine grained material of approximately 1 in. or better in diameter. Further passes of the Seaman and blade mixing did not substantially reduce the number of size of the balls. It was thought that the addition of the MC-800 and subsequent mixing operation would reduce their size and number. Although this occurred to some extent, the mixed material prior to rolling still contained very firm balls and lumps of fine grained material which had an outer coating of MC-800. Existence of the balls was not evident in the sections on completion of rolling but appeared a few weeks later, after a rainy spell, in the form of circular depressions in the surface (see Fig. II-36).

In order to alleviate the balling problem during construction, the scarification method was altered to very light passes of the scarifier (scoring of the surface about 1 in. deep) followed by tight blading the scored material up into a small windrow. This reduced the chunking and number of lumps substantially. This procedure was continued

Fig. II-36. Mudballs in MC-800 sections 1 and 2 (4% residual asphalt).

until a windrow of material of proper size had been built up. This procedure of scarification was followed throughout construction of the remaining sections.

Balling and lumping of subgrade material can probably be expected on most secondary roads if scarified initially to a thick depth. Although the Seaman used in this construction was quite old and delivering much less than peak efficiency, it is probably average of what most counties can be expected to own. Good reduction of lumps and strips of cohesive subgrade material into small aggregates of soil should not be expected to be achieved through the use of a Seaman alone.

After the scarified material had been mixed and windrowed to one side an examination of the subgrade was made. The difficulty of scarifying and blading operations had indicated a very firm, cohesive subgrade. This was confirmed, and the need or benefit in using the sheepsfoot roller on the subgrade material was considered unnecessary. Further examinations of the subgrade condition for all sections of the test road yielded the same conclusion. For the depths of treatment considered, this condition can probably be expected on most secondary roads and the need for the use of a sheepsfoot may be unnecessary in most cases, but should at least be considered.

On completion of final shaping and rolling of the first sections it was noted that there seemed to be insufficient treated material on the shoulder portions of the roadway. Further examinations, conducted by digging a small hole through the compacted material at various points, indicated that near the center of the roadway, treated depths ranged from 5-6 in. while shoulder area treated depths were 2-3 in. Crown measurements at various points averaged 4-4-1/2 in. It was concluded that the cause was due to a relatively flat subgrade cross section upon completion of scarification and mixing. When the mixed material was then laid down and the cross section shaped with a 4-in. crown the result, because of the flat subgrade, was a thickened center portion and thinned shoulder areas. The fact that the subgrade was flat after scarification and mixing was unnoticeable. The volume and cross section of 4 in. of scarified and mixed material put into two windrows near each shoulder nearly covers the roadway except for 4-5 ft in the center. An attempt was made in succeeding sections to crown the subgrade to 4 in. This was very difficult to accomplish or to measure, due again to the presence of the windrowed material. Alteration of the scarification procedure, however, seemed to aid slightly in obtaining a crown in the subgrade. The process of scoring the surface, alternated with tight blading, enabled the desired scarified depth to be obtained gradually, while maintaining the crown that existed on the surface. The crown obtained in this manner, however, reflected the initial crown, which in most cases was very small. Several investigations have indicated that good crown is highly important.

Huang[29], in a 1961 study of potholes and washboarding occurring on soil-aggregate roads in Illinois, concluded that roads with poor or inadequate crowns were much more susceptible to both potholing and washboarding of the surface. He also concluded that potholing was more severe on crushed stone surfaces and washboarding more severe on gravel surfaced roads.

In a 1965 study of crown on soil-aggregate roads in New York, Spencer[30] concluded that potholing was the result of a combination of poor crown and degree of longitudinal grade.

The proposed method of blading the material into one windrow in the center prior to application of water or additives was altered, after trial, to a more efficient procedure in which the entire equipment train was able to move nearly continuously. The procedure was as follows: blading the scarified and mixed material into two flat windrows, approximately 8 ft wide near each shoulder. The distributor would then proceed down one windrow applying water or additive over the entire 8-ft width followed by two blade graders blade mixing, the Seaman self-propelled mixer and another blade grader, again flattening the windrow and mixing. By this time the distributor had reached the other end, turned around and was proceeding back applying water or additive to the opposite windrow. This operation would continue until the proper amount of water or additive had been incorporated. Following this operation both windrows were combined and final mixing accomplished using all three blade graders and Seaman. This process worked very efficiently with respect to incorporation of the additives.

Considerable difficulty was encountered in all sections in obtaining a homogeneous mixture. The volume of material required to obtain a 4-in. compacted treatment is considerable, especially in view of the relatively narrow confines of a roadway cross section. Even in two windrows the material cannot nearly be moved in full depth by blade graders nor can the Seaman penetrate deep enough into the flattened windrow to provide good mixing. This amount of material cannot be handled and mixed efficiently with the equipment that most county departments can be expected to have. In view of the foregoing comments and the experience with the test road a more reasonable depth for in-place road construction would be a 3-in. compacted depth with respect to mixing ease and homogeneity of the final mix. It is felt this amount of material can be handled more efficiently and a more homogeneous mixture obtained as well as reducing the amount of incorporated subgrade material and hence the amount of fines. These factors, however, must be weighed against the loss of stability that may result from the decrease in treated depth.

Table II-10 summarizes the moisture variations for all treated sections at various periods during and after construction. It was desired, initially, to have the final mix be near or above the estimated optimum moisture content for each section (from pre-construction laboratory tests) immediately prior to rolling. Moisture contents in the field were controlled using a calcium carbide moisture tester and personal judgment.

Table II-10. Construction moisture content variations.

Row number	Treatment	MC-800 cutback asphalt				Redicote E-36 cationic asphalt emulsion			
1	Section number	1	2	4	5	6	8	9	10
2	Residual A.C. (%)	4	4	3	2	2	4	4	3
3	Total additive amount[a] (%)	5.4	5.4	4.0	2.7	3.1	6.2	6.2	4.6
4	Approximate optimum moisture (%)	10.5	10.5	10.5	7.5	10.5	10.5	7.5	7.5
5	Amount of liquids in[a] additive (%)	1.3	1.3	1.0	0.7	1.0	2.1	2.1	1.6
6	Moisture content of field material immed. prior to addition of additive (%)	8.6	9.3	7.1	6.6	10.8	9.6	4.5	4.9
7	Calculated liquid content of field mix immed. prior to rolling (%)	9.9	10.6	8.1	7.3	11.8	11.7	6.6	6.5
8	Actual liquid content of field mix immed. prior to rolling (%)	6.6	6.1	5.9	3.7	10.1	8.2	5.7	6.0
9	Actual liquid content of compacted mix 1-2 days after rolling (%)	7.3	6.9	5.0	3.2	10.0	9.1	5.6	6.7

Table II-10. Continued.

Row number	Treatment	MC-800 cutback asphalt				Redicote E-36 cationic asphalt emulsion			
10	Amount of liquids lost in mixing operation (%)	3.3	4.5	2.2	3.6	1.7	3.5	0.9	0.5
11	Dry density (pcf)	120	119	116	122	109	105	110	115

[a] Amount of liquids based on assuming: MC-800 at 25% volatiles; Redicote E-36 at35% water.

Liquid contents referred to in Table II-10 include water plus volatiles for the MC-800 sections and only water for the Redicote emulsion sections. The calculated liquid content of the field mix immediately prior to rolling is shown in row 7, Table II-10 and is obtained by adding row 5 to row 6. Comparison of values for each section shown in row 7 with the estimated optimum for that section from row 4 are in approximate agreement. The actual liquid content (volatiles and/or water) of the field mix immediately prior to rolling (as determined in the laboratory from field samples) are shown in row 8, with considerable difference noted between those shown in row 7. This difference is the liquid loss that occurred during the mixing operation and is shown in row 10. Liquid contents shown in row 9 were obtained from density tests on the compacted mix. Comparison of row 9 with row 8 indicates very little loss occurred in any section during the rolling operation.

Average liquid loss during the mixing operation for all sections was about 2.5%. Average loss for the MC-800 cutback and Redicote emulsion sections was 3.3% and 1.7% respectively. The greater loss for the cutback sections was probably due to a combination of volatile loss and the fact that the mixing operation was slower during the initial stages of test road construction until construction personnel became familiar with the procedure.

The high degree of moisture loss during the mixing operation was not anticipated and was not readily apparent during construction of any of the MC-800 treated sections or section 6 which was the first Redicote emulsion treated section. It was apparently the result of the considerable handling, necessary in an attempt to get the large volume of material worked into a homogeneous mixture, as well as the length of time required to apply the additive.

It was mentioned earlier that water is needed in soil-aggregate mixtures primarily to aid in asphalt dispersion and to facilitate compaction. With respect to all the sections treated with MC-800 cutback asphalt, dispersion of asphalt throughout the mixture appeared good with very few balls or lumps of pure asphalt noted in all sections. Coarse aggregate coating would have to be classified as fair to poor and appeared to vary directly with amount of residual asphalt. As the residual content decreased, coarse aggregate coating decreased. This evidenced itself in the form of aggregate pullout (Fig. II-37) of coarse material under traffic after construction, with the same variation; little to no aggregate pullout at the 4% residual content, with aggregate pullout of coarse material increasing with lower residual contents.

Fig. II-37. Aggregate pullout on section 5 (2% residual asphalt) after construction.

Dry densities of the MC-800 treated sections are shown in row 11 of Table II-10 and are relatively consistent for all sections with an average of about 119 pcf. Although the liquid content at the time of rolling was in excess of 3% below optimum this appeared to affect the dispersion of asphalt for the MC-800 cutback very little. The average density of 119 pcf for all sections is only 5 pcf below optimum density for untreated material and compares very well with treated densities, from Table II-6, obtained on similar materials in the laboratory studies.

The Redicote emulsion treated sections did not behave nearly the same as the MC-800 cutback sections during mixing and rolling. The difference was not noted in the 2% emulsion section as much as in the 3% and 4% emulsion sections. The most striking difference in the 3% and 4% residual emulsion sections was the nearly complete lack of color change of the soil-aggregate mixture after complete incorporation of emulsion. The MC-800 sections, by contrast had turned dark brown to black even at the 2% residual content. On examination of the emulsion treated material of sections 8, 9, and 10, small droplets of asphalt could be seen interspersed among the soil with very little plating of asphalt on soil particles or aggregate. This was most evident on sections 9 and 10 where actual balls and lumps of asphalt as well as droplets developed throughout the mixture with little coating of particles evident. Prior to construction a local asphalt producer and the manufacturer of the Redicote emulsifier had recommended that the soil-aggregate mixture be at or above optimum moisture content prior to the addition of the emulsion. From Table II-10 row 6 it is seen that for sections 6 and 8 the moisture content prior to emulsion addition was near optimum. The color change for sections 6 and 8 was very minor on addition of the emulsion, with the asphalt appearing as small interspersed droplets. There were very few observable lumps or balls present in sections 6 and 8. Section 9 also contained 6.2% total emulsion but was to be placed at field moisture which was approximately 3% below optimum. From Table II-10 the moisture content for section 10 prior to addition of the emulsion was nearly 2-1/2% below optimum moisture content. It was in sections 9 and 10 that the lumps and balls of asphalt were most noticeable. It is therefore concluded that a partial explanation for the asphalt dispersing as droplets and balling is that the moisture content of the soil-aggregate mixture was low, especially for sections 9 and 10 where balling was most evident. Indications are, however, that other factors appear to also be contributing to this behavior. In sections 6 and 8, for instance, moisture content was near optimum prior to additive addition and, although there was little evidence of balling, the asphalt still appeared as droplets with very little plating action on aggregate, or color change noted.

This behavior may be due to either (a) the Redicote emulsion being relatively unstable and "breaks" too quickly, or (b) it may be due to the cationic nature of the emulsion. In either case what may have happened is that the cationic emulsifier constituent, which is theoretically supposed to function as a bonding agent to wet particle surfaces, is being partially utilized to satisfy the cation exchange capacity of the soil. If this is happening, coalescence of the asphalt may take place in varying degrees, with very little bonding being capable of developing with wet particle surfaces. This would be indicated by particles of asphalt possibly ranging in size from droplets to lumps or balls.

It was noted for 20-2 loess, which has a 5-micron clay content of about 20% and a cation exchange capacity (CEC) of 13.4 me/100 g (whole material) that droplets of asphalt were visible along unconfined compression specimens sides surrounded by a matrix of apparently asphalt-free soil. It was shown on simulator specimens for test road materials in Fig. II-30 that the asphalt on the emulsion specimen sides also appeared as globules and droplets of asphalt, on close examination. The "peppering" of the surface with asphalt was more apparent in the loess and test road specimens than the Bedford specimens possibly because of the larger surface areas of the former. Bedford limestone material has a 5-micron clay content of about 5.5% and a CEC of 10.9 me/100 g (whole material). Poweshiek test road materials have a 5-micron clay content of 10-15% with an unknown CEC. Cation exchange capacity of a soil is primarily a function of the amount and type of clay minerals present[31]. Soils containing montmorillonite minerals are relatively high in exchange capacity. It was noted in the laboratory traffic simulator tests that the Poweshiek test road materials had shrunk away from holding ring walls during the 24-hour air curing period. This may be indicative of montmorillonitic clays and hence a high CEC for the test road materials.

The balling and droplet formation using the Redicote emulsion was not at all evidenced during sample mixing and preparation, in the laboratory studies, with either the Bedford limestone material or the test road materials. In contrast, coating and plating action appeared to be quite good in comparison with the MC-800 specimens. The reason for this behavior occurring in the field and not in the laboratory is probably related to moisture content variations and differences in laboratory mixing and field mixing, with time of mixing and intensity of mixing being the primary factors. The laboratory mixing took place with the soil being continuously stirred as the emulsion was slowly added and received a total amount of mixing time of about 3-4 min. The same amount of soil in the field probably received only a fraction of this. The span of time over which mixing and compaction took place is again considerable, with the laboratory mixes being prepared in 5-10 min where field mixes took several hours for the entire amount of additive to be incorporated, and final mixing and compaction to be accomplished.

An additional factor, to those mentioned above, with respect to the dispersion and balling effect of sections 9 and 10, is that the temperature of the emulsion applied to sections 6 and 8 was approximately 150 ^{o}F. The temperature of the emulsion applied to sections 9 and 10 was about 75 ^{o}F, considerably lower. It was shown in the laboratory screening study that heating of the Redicote emulsion, before application, somewhat increases the unconfined compressive strength. In preparing specimens with both heated and unheated emulsion, however, no visible differences were noted in mixing qualities or mixture appearance. It therefore becomes difficult to say what overall effect the temperature difference would create.

Summary

Several problems arose during test road construction that required adjustment of the proposed construction method. These problems also provided a basis for recommendations on future construction.

Scarification of the roadbed to the full 4-in. depth on the first test sections resulted in "plowing" up of chunks and strips of cohesive subgrade materials. These could not be reduced in size beyond lumps and balls of about 1 in. in diameter with the equipment available. A process of lightly scoring (1 in. or less) the surface with the scarifier followed immediately by tight blading resulted in a windrow being gradually built up to proper size which was relatively free of chunks and strips of cohesive material. This process substantially reduced the number of lumps and balls of cohesive material present in the soil-aggregate mixture.

On the first test sections constructed, the subgrade cross section was relatively flat prior to placement of the treated material. This resulted in a thickened center (5-6 in.) and thinned shoulder portions (2-3 in.) of the compacted treatment placed with about a 4-in. crown. An attempt was made to put a 4-in. crown in the subgrade on subsequent sections. This was difficult to control, however, because of the volume of scarified material present on the subgrade surface. The process of scoring and tight blading the surface, tended to produce a crown in the subgrade which was a reflection of the crown existing prior to treatment. This worked well where good crown existed but was the exception rather than the rule. The crown obtained for most of the test sections was unsatisfactory. Difficulty in obtaining the 4-in. crown was aggravated by the method of rolling in relatively thin lifts and the volume of material present. This allowed very little shaping to be done until there was insufficient material available to form a good cross section.

The volume of material generated from 4 in. of compacted material is difficult to handle and mix efficiently on the confines of a 24-ft roadway. This is especially true with regard to the type and size of equipment most counties can be expected to have. A 3-in. treatment is probably a more reasonable depth to use in this respect. This must be weighed, however, against the stability loss that might be expected by decreasing mat thickness. A more homogeneous mixture may not be worth the loss in stability.

Moisture loss during the mixing operation was much greater than anticipated ranging from ± 3% for the MC-800 sections to ± 2% for the emulsion sections. The MC-800 sections appeared to be affected very little with respect to asphalt dispersion and final density. The effects of this were not readily apparent until construction of sections 8, 9 and 10, treated with emulsion. Sections 9 and 10 showed the largest reaction to the low moisture in the form of balling of the asphalt. The emulsion treated sections indicated very little color change and a high dispersion of asphalt in the form of very small droplets. The size of droplets appeared to increase to balls and lumps as moisture content decreased. Very little plating action or particle coating was noted for any of the emulsion sections. It is felt, however, that low moisture content may not be the only factor contributing to this action. The considerable amount of material passing the #200 sieve having approximately 15% 5-micron clay may be influencing this behavior. What may be happening is that the cationic emulsifier constituent, which is theoretically supposed to function as a bonding agent to damp (and therefore theoretically charged) particle surfaces is being partially utilized to satisfy the cation exchange capacity of the soil. If this is happening, coalescence of the asphalt would likely take place with very little bonding being capable of developing with the wet particles.

Recommendations for Future Construction Procedure

1. A 3-in. compacted treatment rather than a 4-in. treatment is recommended for efficiency of handling and to obtain a more homogeneous mixture. Obtaining a more homogeneous mixture, however, must be weighed against the stability loss that might occur in reducing treated thickness.

2. Alteration of scarification procedure to reduce chunks and lumps of subgrade material. A process of scoring the surface followed by tight blading, with gradual windrow buildup, worked reasonably well for the Poweshiek test road.

3. Eliminate the sheepsfoot roller for the subgrade unless adverse weak conditions are expected.

4. Some means must be sought to insure that the proper crown is present on the subgrade as well as the finished surface. This is difficult with the volume of material present on the subgrade prior to treatment.

5. High moisture losses may be anticipated during the mixing operation ranging from 2-3% and should be provided for.

6. Adoption of the procedure of mixing and applying the additives to two flattened windrows near each shoulder is recommended. This provides for continuous movement of the equipment train. Upon complete addition of the additives the windrows are combined and final mixing takes place.

Post-Construction Testing

The test road has been in service about 1-1/4 years at the time of this writing.

Field Tests, 1971

Spherical bearing value, Benkelman beam stiffness, and in-place density test results of the treated sections, are shown in Fig. II-38 compared with untreated results shortly after construction. In general, results from all tests were lower than results for the same tests on the untreated road and appeared to be related to differences in densities obtained.

Average dry density of the MC-800 treated sections was about 119 pcf compared with an average of about 110 pcf for the emulsion treated sections. This was probably partially due to the low moisture contents prior to compaction but more likely due to the fact that asphalt coating of soil aggregates and aggregate particles was better in the cutback treated sections and consequently acted as lubricants during compaction, enabling higher densities to be obtained. There was also a greater drop in density, from the untreated condition, noted for the emulsion sections.

There appeared to be only slight variations in spherical bearing values and Benkelman beam stiffness for all the cutback sections and the 2% emulsion section. The values for the same tests on section 8, 9 and 10 were slightly lower, probably because of the lower densities. There also appeared to be greater variation in the Benkelman beam tests from the untreated condition for sections 8, 9 and 10, again probably related to the lower densities obtained on these sections.

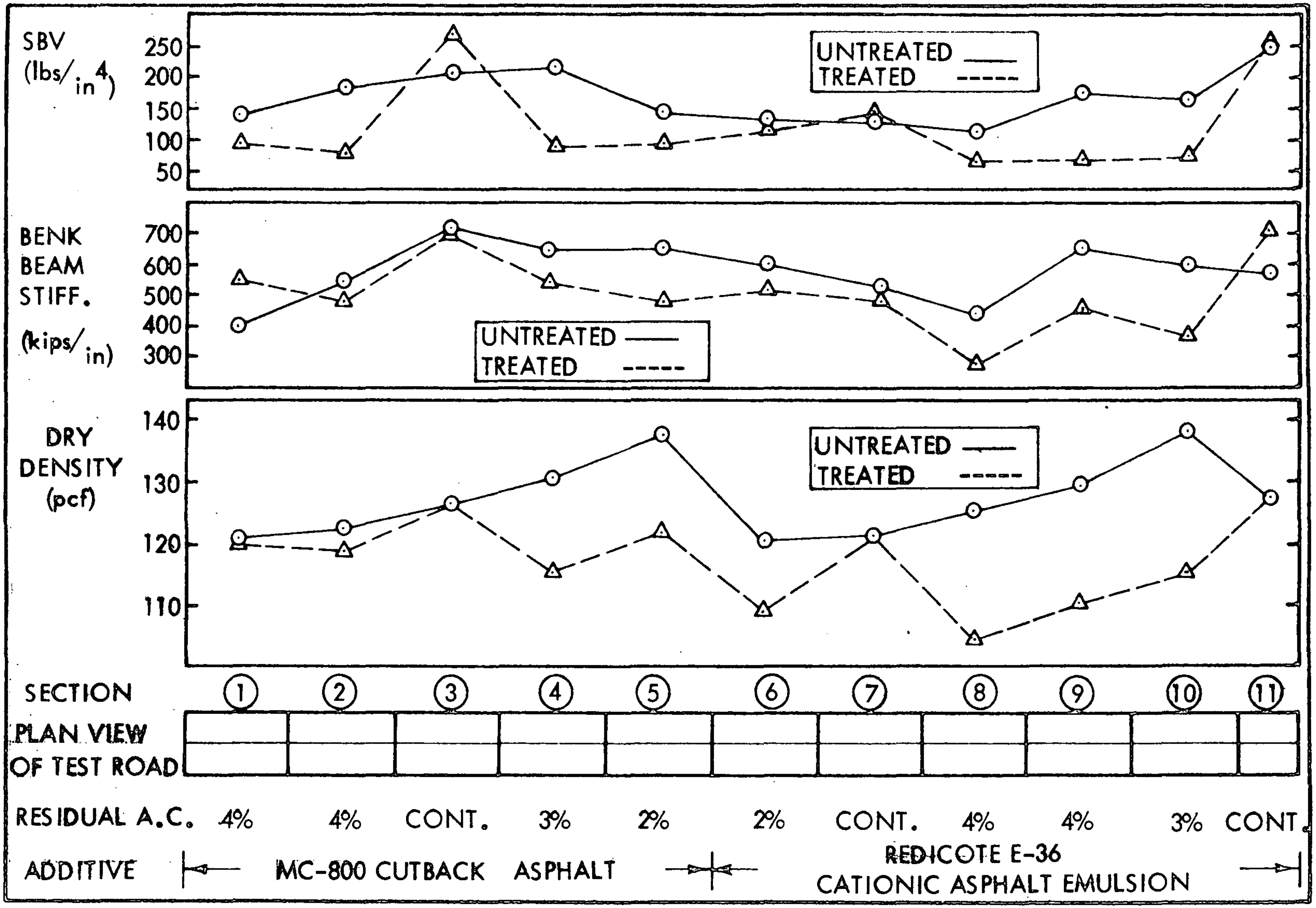


Fig. II-38. Field test results, Poweshiek County test road, 1971.

Crown measurements on all sections are shown in Fig. II-39, as taken 11 ft right and left of centerline, again shortly after construction. Two measurements were made on each section, at the third points, using a string line and line level. The values shown are the average of the right and left measurement at each third point. In some cases a negative or zero value was obtained for one of the measurements. As can be seen, the obtained crown was in most cases much less than the desired 4-in. crown, averaging about 2 in. In all likelihood this should affect the overall performance of the various sections particularly over several years' time. Studies by Huang[29] and Spencer[30] indicated that poor crown on soil-aggregate roads was the major cause of potholing and washboarding of the surface. This results from water during rainy periods, "ponding" in slight depressions and subsequent tracking out and flushing of surface material under traffic. Figure II-41 illustrates this behavior on an untreated section of the test road. This behavior might also be expected if poor crown is obtained on a treated surface that had not been rendered completely waterproof. The result being that ponded water would gradually soften the semi-waterproof matrix of binder material and additive with subsequent removal of the material by traffic. This process was noted on the MC-800 sections (Figs. II-42 and II-44). The Redicote emulsion sections exhibited this behavior only slightly as noted on Figs. II-43 and II-45.

Observations, 1971

It had been recommended to the county engineer that no maintenance be performed on the test road unless absolutely necessary. To the best of our knowledge no maintenance has been required on the treated sections.

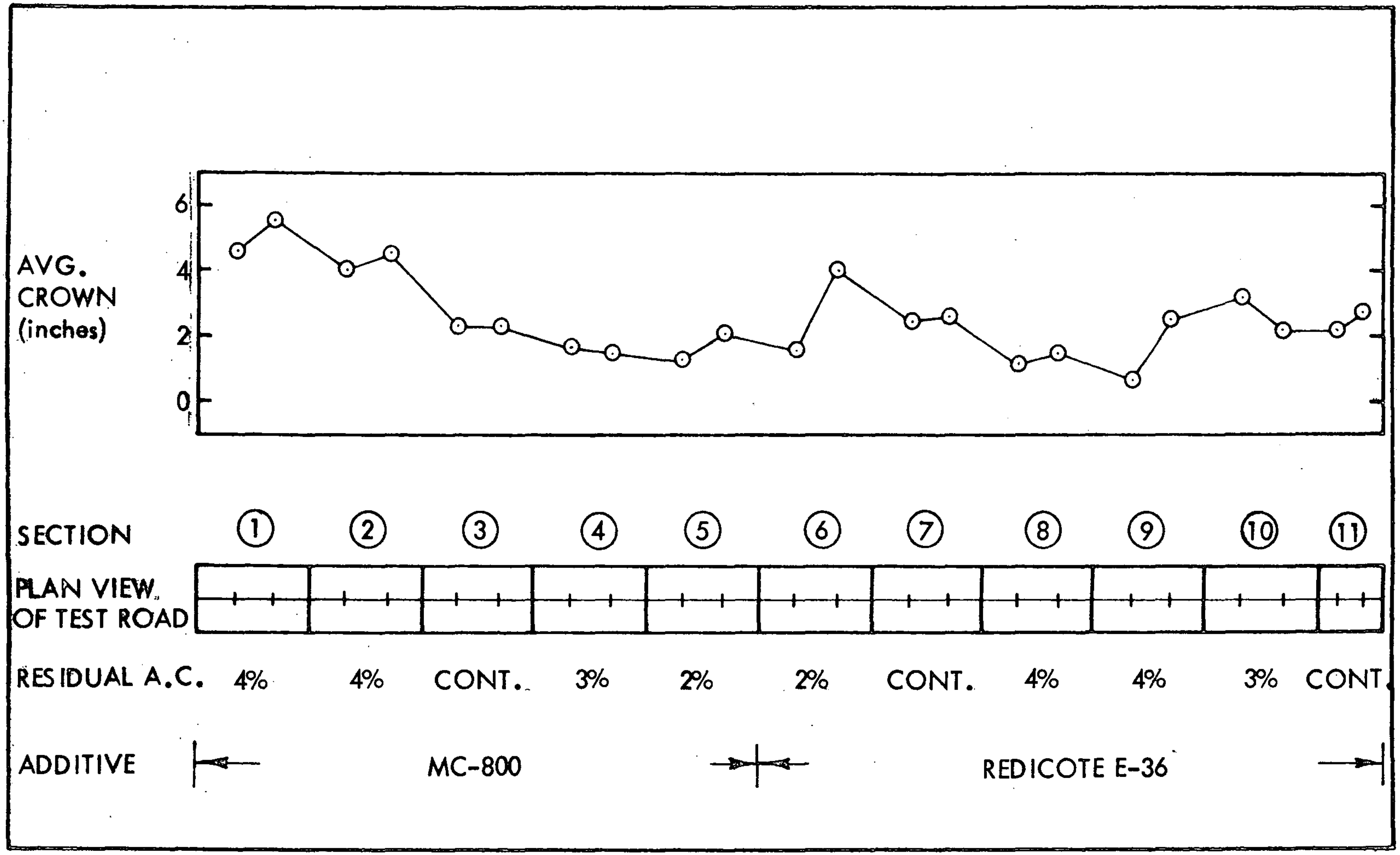


Fig. II-39. Post-construction field test results, Poweshiek County test road, 1971.

Fig. II-40. Completed test road approximately two months after construction (note vehicle and light dusting from section 7).

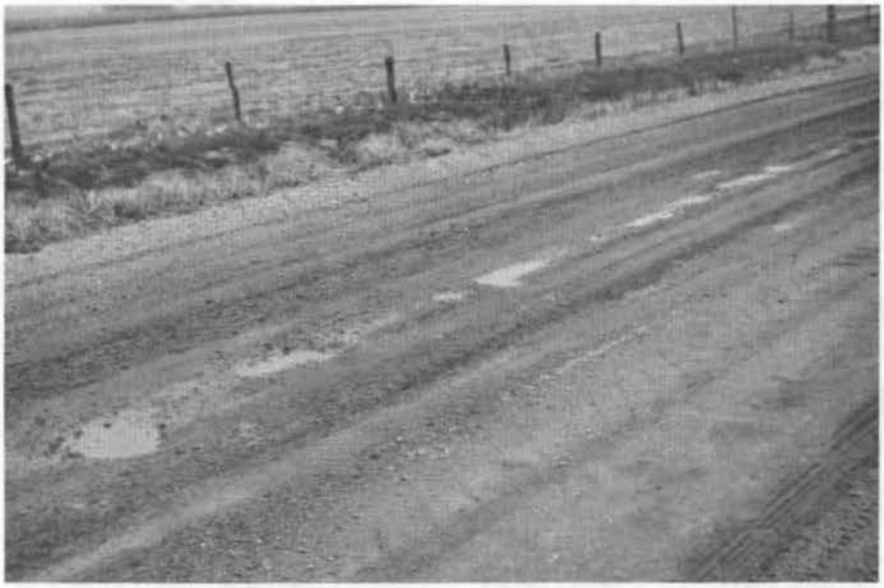


Fig. II-41. Potholing on section 7 (untreated) approximately 1-1/2 months after construction.

Fig. II-42. Section 5 (2% MC-800) shortly after construction and long drizzling rain, looking west.

Fig. II-43. Section 8 (4% Redicote emulsion) shortly after construction and long drizzling rain, looking east.

Fig. II-44. Section 2 (4% MC-800) 3 months after construction, looking west.

Fig. II-45. Section 8 (4% Redicote emulsion) 3 months after construction, looking east.

Periodic examinations of the test road were conducted during the fall of 1971 with the following visual observations:

Dusting was visibly reduced on all treated sections (compare Figs. II-40 and II-23) in varying degrees. Shortly after construction the 2% and 3% MC-800 treated sections began exhibiting aggregate pullout (predominately + #4) which migrated, under traffic, to the shoulder area leaving a tight surface in the traveled way (Fig. II-37). The aggregate pullout effect was heavier on the 2% section than the 3% section with none evident on the 4% cutback sections. The dusting at this time, in order of severity, was 4% (none to slight), 3%, and 2% and appeared to be related to the amount of aggregate pullout.

The emulsion treated sections behaved quite differently. On completion of construction and final rolling there was a thin film of very fine, apparently untreated material on the surface of each emulsion-treated section. This produced dusting on all sections immediately after opening to traffic and created a slippery, greasy surface during rain (Fig. II-43). Approximately three months of traffic, coupled with several rainy periods partially removed this initial film of fine material and resulted in a tight surface in the traveled way (Fig. II-45) with only a small amount of aggregate pullout evident in the shoulder areas, relative to the MC-800 treated sections. There did appear to be a light accumulation of fine material (predominately - #4) near the shoulder areas where traffic was light. As mentioned earlier there was very little darkening of the soil-aggregate material following addition of the emulsion additives on all sections, with the asphalt appearing in the form of droplets. Following final rolling the same observation was noted on the completed emulsion sections. It appeared as if no additives had been incorporated into the soil-aggregate emulsion treated sections. Since construction, however, there has been a visible darkening of the surface, especially in the traveled way. It appears that the asphalt present was gradually worked into the surface and provided a waterproof matrix around coarse aggregate particles (Fig. II-45).

With respect to surface improvement it was evident that the stability of the surface for all treated sections, with the exception of the 2% emulsion section, was improved. All three untreated sections developed potholes, chuckholes and washboarding that required maintenance in mid-December 1971 (Fig. II-41).

Shortly after construction the test road underwent a 3- to 4-day drizzling rain at the end of which a semi tractor-trailer loaded with cattle attempted to traverse the test road from west to east. He became stuck in the 2% emulsion section and had to back out the 1/2 mile. As a consequence the 2% emulsion section, which had become very soft during the rain, sustained 2- to 3-in. deep ruts over nearly 2/3 of its length. During every rain period in late 1971 these ruts filled with water and its condition was aggravated. Performance of the 2% emulsion section has been entirely unsatisfactory with little waterproofing accomplished. This condition initially was aggravated by the poor crown obtained during construction.

Prior to a freeze-thaw period early in December 1971 the surface of all sections was relatively free of any signs of deterioration other than circular depressions apparently formed when mudballs (Fig. II-36), present in the surface at the end of construction, were softened by rain and flushed out by traffic. These were most apparent in the MC-800 treated sections. The freeze-thaw period and subsequent traffic began to enlarge and deepen these initially small depressions to the point where they resembled very small potholes (Fig. II-44). This occurred to the greatest extent on the MC-800 sections in the following order: 4% (none to slight), 3% and 2%. Emulsion treated sections exhibited this behavior only very slightly (Fig. II-45). This was probably partially explained by the alteration of the scarification procedure previously mentioned and the tighter surface, obtained with time and traffic, exhibited by the emulsion sections.

A visual examination of the surface of all sections, in December 1971, indicated that the MC-800 sections were much coarser in surface texture than the emulsion sections. This was apparently resulting from binder material being removed from around aggregate particles on the cutback sections. The emulsion sections, by contrast, had a matrix of binder material around coarse aggregate particles that had not been removed with time and traffic. It was also observed, and confirmed by local residents, that the emulsion sections were still slippery and greasy when wet as compared to the cutback sections (Figs. II-42 and II-43). This was thought to be due to the slight amount of asphalt-free binder material remaining on the surface which should, and has, decreased with time and traffic.

Thermal Characteristics

Temperature recorders were installed in sections 4, 7, and 10 with probes at 4 and 8 in., for thermal evaluation of the treatments with respect to the untreated sections. The recorder in section 7 was a 3-probe unit with one probe monitoring ambient air temperature.

The test road was observed immediately after two snowstorms and sleet storms, and in both cases melting of snow and ice was estimated to be 25-50% faster on all the MC-800 test sections (compare Figs. II-44 and II-45). This was probably due to surface color differences, with the cutback sections being much darker and hence absorbing heat faster. The thermal characteristics of the cutbacks versus the emulsions, however, may also have contributed to this behavior.

Observations of thermal characteristics are given in Table II-11 for a number of periods during which ambient temperature rose and dropped over various lengths of time. It can be seen that for periods of ambient temperature drop, the treated sections exhibited higher temperatures at both 4- and 8-in. depths than the untreated sections; an indicator that both treated sections were functioning to some degree as insulating layers with 3% residual asphalt each. In general, it appeared that the MC-800 treatment was functioning somewhat better with respect to insulation of the subgrade as noted from the 8-in. temperatures with the exception of March 3, 1972, when an unexplainable anomaly occurred in the Redicote emulsion section. The Redicote emulsion, however, exhibited slightly higher temperatures at the treatment interface (4 in.) than any of the MC-800 readings during ambient temperature drops, another unexplainable anomaly.

During periods of ambient temperature rise, the treated sections exhibited both higher and lower temperatures at both 4- and 8-in. depths than the untreated. So long as the ambient temperature was still near — or less than — freezing, the treated sections exhibited higher temperatures at both depths than the untreated. When the ambient temperature was well above freezing, the treated sections exhibited

Table II-11. Temperature variations of treated sections versus untreated sections.

Date	Temperature drop or rise	Depth (in.)	Temperature (°F) Section 7 untreated	Temperature (°F) Section 4 MC-800	Temperature (°F) Section 10 Redicote emulsion
Dec. 20, 1971	17° drop in 18 hr	0	- 2	—	—
		4	12	17	19
		8	19	28	23
Dec. 21 1971	14° rise in 2 hr	0	10	—	—
		4	16	22	24
		8	20	29	28
Jan. 4, 1972	12° drop in 3 hr	0	- 12	—	—
		4	4	10	12
		8	12	22	16
Jan. 6, 1972	26° drop in 9 hr	0	- 20	—	—
		4	4	16	a
		8	10	20	a
Jan. 11, 1972	20° rise in 14 hr	0	50	—	—
		4	28	24	26
		8	24	28	25
Feb. 19, 1972	36° drop in 10 hr	0	- 6	—	—
		4	9	13	17
		8	14	22	17
Feb. 20, 1972	44° rise in 11 hr	0	38	—	—
		4	24	26	29
		8	20	25	22

Table II-11. Continued.

Date	Temperature drop or rise	Depth (in.)	Temperature (°F)		
			Section 7 untreated	Section 4 MC-800	Section 10 Redicote emulsion
Mar. 3, 1972	20° drop in 10 hr	0	26	—	—
		4	23	25	39
		8	22	29	32
Mar. 4, 1972	52° rise in 14 hr	0	78	—	—
		4	52	55	40
		8	35	35	37
June 8, 1972	44° rise in 12 hr	0	102	—	—
		4	90	90	84
		8	80	80	73
June 9, 1972	40° drop in 13 hr	0	62	—	—
		4	70	72	69
		8	72	78	72
Nov. 3, 1972	17° drop in 4 hr	0	29	—	—
		4	[a]	40	44
		8	41	46	43
Nov. 3, 1972	18° rise in 8 hr	0	47	—	—
		4	[a]	43	46
		8	43	44	42

[a] Recorder ran out of ink.

similar or lower temperatures at both depths than the untreated. This latter effect was probably due to ground temperatures below the 8-in. depth still being of lesser temperature and creating a lag in the gradient.

The data of Table II-11 indicates a lowering of the thermal gradients in the treated as compared with the untreated sections during freezing and thawing. A lowering of the thermal conductivity is advantageous in a probable lessening of the number of freeze-thaw cycles of the subgrade materials but does not necessarily indicate a lowering of the freezing point. Lower portions of the surface material may also have a lowering of the gradient, but due to adsorption of the surface heat from the sun, the upper portion of the surface would receive freeze-thaw cycles in direct accordance with ambient air temperatures.

Field Tests, 1972

Spherical bearing value and Benkelman beam stiffness of the test sections are shown in Fig. II-46, approximately 9 and 14 months after construction. Each point is the average of at least two SBV tests and four Benkelman beam tests in each section.

In general SBV tests results at about nine months were slightly higher than those shortly after construction, but were generally lower than all the sections prior to construction in 1971 (Fig. II-38). This effect was probably due to the above normal moistures encountered during the 1971-72 winter, and spring of 1972, thus retaining a great deal of moisture in the subgrade.

The effect of wet subgrade is further illustrated by the reduction of Benkelman beam stiffness values in Fig. II-46 as compared with pre- and immediate post-construction values in Fig. II-38.

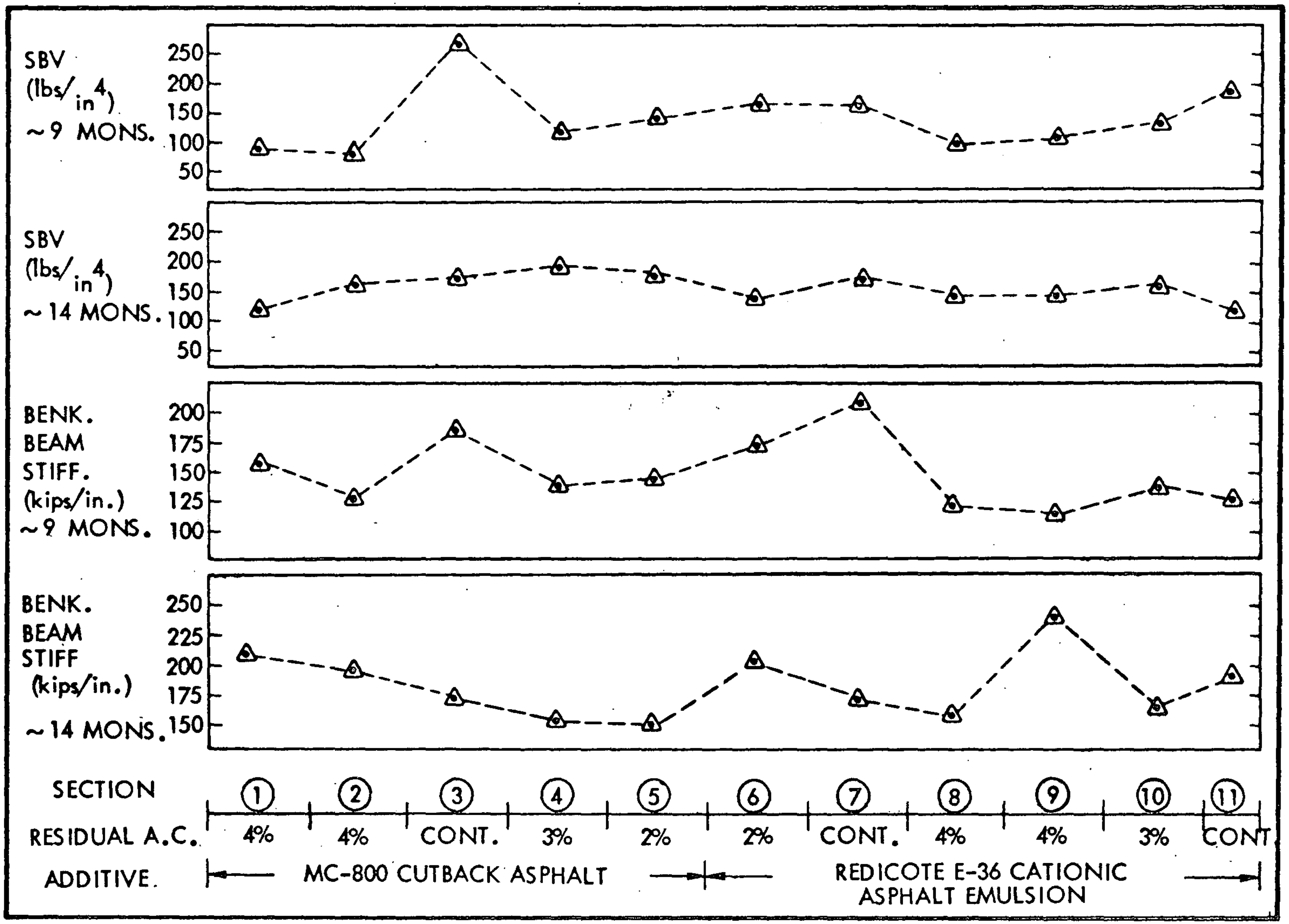


Fig. II-46. Field test results, Poweshiek County test road, 1972.

Above normal rains during the summer and particularly the fall of 1972 did little to improve subgrade moisture conditions as is evidenced by the SBV and beam results, 14 months after construction. Neither of these values increased significantly during the 5 months between tests and in sections 3, 6, and 11 the spherical bearing values were somewhat reduced.

Spherical bearing values (SBV) have been compared to unsoaked CBR and unconfined compressive strengths of soil[26]. For example, an SBV of 100 is approximately equivalent to an unsoaked CBR of 5, while an SBV of 200 is about a CBR of 18 and an SBV of 300 is equivalent to CBR 32. Unconfined compressive strengths for the same three SBV's noted above are respectively 27, 54, and 80 psi.

SBV values of all test sections were in excess of 100, and 8 of the 11 test sections were 150 or greater after 14 months following construction. Unconfined compressive strength test results of the Poweshiek County test road materials (Fig. II-26) were all in excess of 80 psi indicating that SBV's should have been 300 or greater for the treated materials only. Obviously therefore, in-place SBV data is greatly affected by the supporting capacity of the subgrade which is evident in the results presented in Fig. II-46. Since most of the test sections were still maintaining their integrity and performance under traffic after 14 months duration, the MC-800 and Redicote E-36 emulsion must be assumed to have been providing adequate strength, durability, and yet an elastic flexibility over a weak subgrade.

Benkelman beam stiffness is a factor obtained by dividing axle load by average maximum deflection; the more flexible the material, the lower the relative stiffness factor. Using the axle load of the test truck (17,280 lb), maximum stiffness for Navy Department flexible pavement design criteria (limiting deflection of 0.2 in.) would be 86.4 kips/in.[32]. After nine months all sections were in excess of 110 kips/in. of stiffness, while 14 months following construction all sections were in excess of 150 kips/in. stiffness. Such evidence indicates that the design performance of all sections was still quite adequate. This should not be construed that each section was a piece of high performance pavement, since it has been shown for example that stiffness of a 6-in. lime treated subbase plus 7 in. of soil-cement base plus a 3-in. asphaltic concrete surfaced pavement may be in excess of 1000[27]; the minimum Poweshiek County test road data is thus only 10 to 15% of that noted in the example, and far more susceptible to subgrade moisture variations due to thickness of treatment.

As indicated earlier in this report, surface aggregate replacement (a) constitutes a substantial portion of a county maintenance budget and (b) provides a substantial share of dust from unpaved secondary roads due to aggregate degradation. Thus a portion of this total dust control and surface improvement study was to consider in some partial quantitative fashion how well a stabilization treatment might retain aggregate under some form of abrasion. Thus a simple brooming test was used. Photographic examples of the test procedure are shown in Figs. II-47, and III-55 to III-66, Part III Final Report. An area 3 ft wide and 24 ft in length was laid out in duplicate in each section, each covering the bulk of the transverse width of the roadway. A very stiff bristle broom was then used to push and peel every possible piece of aggregate and other particles out of the exposed surface. The quantity

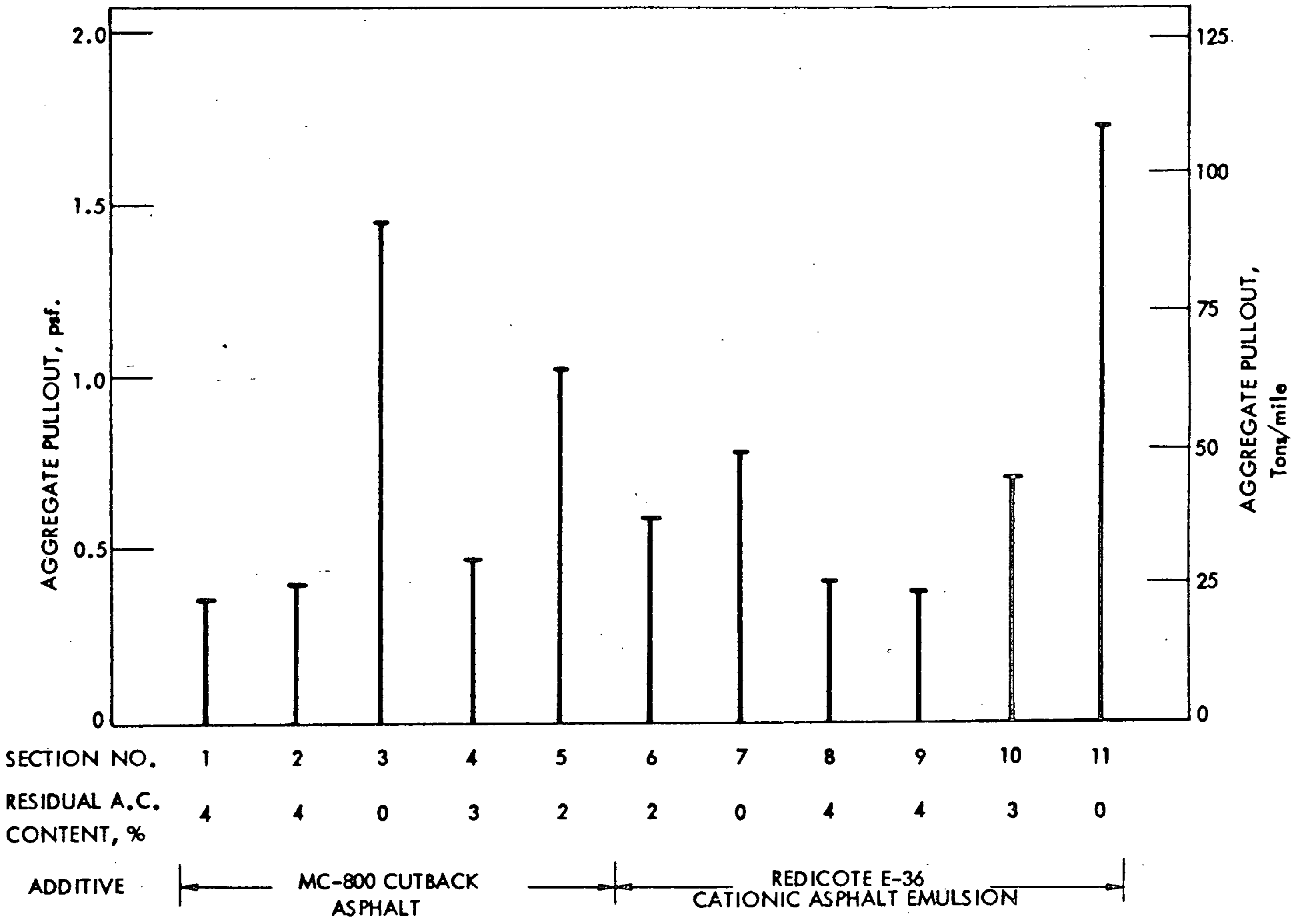


Fig. II-47. Aggregate pullout from brushing test, approximately 14 months after construction.

of materials thus collected were weighed and are related in Fig. II-47 in two fashions: (a) pounds per square foot of broomed area; and (b) tons per mile of 24-ft-wide roadway. Obviously this technique does not predict what occurs due to softening during a rain, what may be bladed off with a snow plow, etc., but is a partial means at least of analyzing the tenacity of a stabilizing agent for holding a particle of soil or aggregate.

From Fig. II-47, all 4% sections are noted as having from 50 to less than 25% as much aggregate pullout as the untreated sections. The 3% sections of each treatment were not as effective as the 4% sections and the 2% treated sections were even less effective. Section 5 (2% MC-800) had greater aggregate pullout than section 7 (control, 0%). It may be reasoned from this study that the 4% treatment levels of either MC-800 or Redicote E-36 emulsified asphalt produce the greatest resistance to surface abrasion and aggregate pullout. The 2% sections would be considered only partially effective.

It is of at least academic interest to note in Fig. II-47 the slight differences existing between sections 1 and 2 and sections 8 and 9. As noted in Table II-8 moisture control for section 1 was to be at the most desirable content, while section 2 was as existed at time of construction. From Table II-10 the amount of liquids lost in mixing operations was 1.2% higher in section 2 than in section 1. Section 8 was to contain a controlled water content during field operations, while section 9 was as existed at time of construction. Yet from Table II-10, section 8 lost 2.6% more liquids during mixing operations than did section 9. Thus both 4% sections that lost the most water during mixing show slightly higher aggregate pullout quantities than their lower moisture loss, 4% treatment, counterparts.

Comparisons of SBV, Benkelman beam, and aggregate pullout data were made with AASHO classification, plasticity indexes, liquid limits, quantity of - 200 sieve fraction, and density of original test section materials. No specific correlations were obtained, with one exception; i.e., Benkelman beam stiffness values at about nine months following construction (Fig. II-46) were nearly identical to the percent passing #200 sieve (Fig. II-35). Though this singular correlation could be of an "I told you so" type, let it suffice to say that further research is probably needed.

General visual observations of the test road from date of construction to date of termination of the project in mid-December 1972 were as follows:

Sections 1 and 2 (4% MC-800) Surface developed a very blocky structure. Clay ball pullout (Fig. II-36) steadily developed with time, creating minor potholes, and was apparently washed away with rain since little or no dusting was observable at speeds up to 50 mph. Small amounts of rutting occurred in wheel tracks. Ride qualities, generally good.

Sections 3, 7, and 11 (untreated) Aggregate pullout was definitive, with much aggregate either side of roadway. Lots of dusting at up to 50 mph. Definite potholing, extremely bumpy ride. Surface transverse cracking developed during winter 1971-72. Occasional washboarding which required blading.

Section 4 (3% MC-800) Slight dusting, slight potholing. Small amount of rutting in wheel tracks.

Section 5 (2% MC-800) Aggregate pullout well defined with some quantity of aggregate at both edges of the roadway. Definite dusting at 50 mph though at least 75% less than untreated sections. Slightly greater rutting in wheel tracks than in section 3. Some potholing.

Section 6 (2% Emulsion) Definite aggregate pullout with some quantities at either edge of roadway. Definite dusting at 50 mph though at least 50% less than untreated sections. Some rutting and potholing. Never quite recovered from loaded cattle truck traversing road shortly after construction and during 3-4 days of drizzling rain.

Sections 8 and 9 (4% Emulsion) Surface texture fairly smooth and tight, occasional slight blockiness. No evidence of clayball pullout. Surface maintained a "peppered" appearance of asphalt. Very minor rutting, though section 9 developed a few minor potholes near the culvert. Dusting, very slight. Driving characteristics, good.

Section 10 (3% Emulsion) Surface fairly smooth and tight, no blockiness. No evidence of clayball pullout. Only slight "peppered" appearance of asphalt. Driving characteristics good, with very slight dusting, very minor rutting.

SUMMARY AND CONCLUSIONS

1. From the unconfined compression study it appeared that the air cured 24 hours-immersed 24 hours test provides an indication of an additives potential as a waterproofer. It does not appear, however, to provide an adequate indication of stability.

2. Traffic simulator results are thought to give valid indications of both fine material retention and waterproofing, as well as

an indication of a materials stability under moving load and imposed environmental conditions.

3. On the basis of the unconfined compression study and the traffic simulator study, Petroset SB cationic latex emulsion, Redicote E-36 cationic asphalt emulsion and MC-800 cutback asphalt appeared to be the most effective in providing waterproofing, fine material retention and stability.

4. In view of the scope of this project for low-cost surface improvement and dust palliation and on the basis of economic considerations, Redicote E-36 cationic asphalt emulsion and MC-800 cutback asphalt were recommended for field trial.

5. Dust data from the test road and a local road near the Iowa State Center in Ames, Iowa, indicate that for limestone surfaced roads about 100 tons of road material, in the form of airborne dust, may be lost per mile per year at a traffic count of 100 vehicles per day. This loss takes place over a 1000-ft corridor, 500 ft on either side of centerline. Indications are that the amount of dust lost is a function of the traffic count and may be estimated if the traffic count is known.

6. Several problems arose during test road construction that required alteration of the proposed construction method and provided a basis for recommendations for future construction. A detailed discussion is presented in the appropriate subsection of the text.

7. The Redicote cationic asphalt emulsion behaved much differently in the field than the laboratory, showing an almost complete lack of color change on incorporation of the additive; the asphalt appearing as small highly dispersed droplets or globules. This is thought

to be primarily due to low field moisture contents of the soil-aggregate mixture and indicates that considerable moisture (above optimum) must be available in a soil-aggregate mixture, at the time of incorporation, for proper distribution of the emulsion. Indications are that surface area and possibly cation exchange capacity of the soil may be additional factors contributing to the droplet formation.

8. The test road has been in service in excess of 14 months at the time of this writing. Dusting has been visibly reduced on all treated sections in varying degrees. Aggregate pullout (predominately + #4 sieve) occurred on the MC-800 sections of low residual asphalt content and resulted in the formation of very small potholes and coarse surface texture. This is not evidenced in the emulsion treated sections which have maintained a relatively tight surface.

With respect to surface improvement it is evident that the stability of the surface for all treated sections (with the exception of the 2% emulsion section whose performance has been unsatisfactory) has been improved. All three untreated sections have had severe potholes and washboarding develop that required maintenance beginning about 2-1/2 months after construction.

9. Of the various applied treatments, the sections containing 4% MC-800 and 4% Redicote E-36 emulsion have shown the highest degree of performance and stability. Of these two products, the 4% Redicote E-36 asphalt emulsion appeared to be somewhat superior from an overall viewpoint.

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PART III

AMMONIUM LIGNOSULFONATES AS DUST PALLIATIVES AND SURFACE IMPROVEMENT AGENTS FOR UNPAVED SECONDARY ROADS

by

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and

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CONTENTS

INTRODUCTION

At the time the research project was initiated in mid-1970, county engineer offices in Iowa were requested to propose locations for test roads within their respective counties. It was explained that all men, material, and equipment would be furnished by the individual counties. Laboratory screenings of additives and field testing of experimental roads would be carried out by the researchers, who would also be present to observe construction and provide technical assistance when and where needed. Four counties responded with interest in lignosulfonate stabilization.

Lignosulfonates are available in almost unlimited quantities from paper mills. Ten years ago lignosulfonates cost 6 cents per gallon at the mill and the price has not increased, although production has. Due to different pulping processes the cation associated with lignosulfonate varies. Sodium, calcium, and ammonium lignosulfonates have all been used in soil stabilization and prices are similar.

The study reported here utilized commercially available lignosulfonate with ammonium as the associated cation. The following paragraphs explain the participation offered for lignosulfonate research by the corresponding county.

The county engineer's office of Clinton County, Iowa made available a section of road 2620 ft long to which crushed limestone had been added at a rate of 2000 tons per mile. This length was naturally divided into three approximately equal sections, a flat upland location, a curve on a hill of about 6% grade, and a level bottomland section on a backwater area of the Mississippi River. The road was located between, and connected with, paved roads terminating at the Clinton and Camanche city limits. Two nearby industries contributed to a 1967 ADT (average daily traffic) of 500 vehicles per day, and maintenance was a problem.

The researchers were invited to perform field tests on several roads within Floyd County, both prior to and following treatment with lignosulfonate. Construction was done by county personnel under the supervision of the lignosulfonate supplier.

A dirt road near Marion, Linn County, was made available by the county engineer for research. Of 6000 ft length, the 1968 traffic survey showed an ADT of 44 vehicles. Prior to treatment, the grade was built up and improved by the ditch clean-out method typical of much secondary road construction.

Using lignosulfonate from a local source, stabilization of nearly 150 miles of roads was begun in Lee County in the late 1950's and early 1960's. Treatment consisted of only light surface blading and occasional application of aggregate coupled with a lignosulfonate surface spray application. Over the years an in-depth treatment was produced. Extensive cost and maintenance records on treated roads were kept and made available to the researchers.

PREVIOUS EXPERIENCES

Lignosulfonates have been used for many years as a soil stabilizer. Replenishment by periodic treatment has been necessary as lignosulfonates dissolve and leach out. More than 60 years ago work was progressing to reduce solubility by mixing with chromic salts to produce an insoluble gel[1]. Scientists and engineers in the United States, Scandinavia, and elsewhere are still researching this reaction. The disadvantage of the chrome-lignin process is high cost of the necessary quantity of salts and their toxic effect in direct human contact.

Recent concern has been directed to the effect of lignosulfonates on persons and animals, as leachates may eventually appear in water consumed by both. Two Federal Food and Drug Administration regulations are directed to these circumstances. One permits inclusion of lignosulfonates into animal feed pellets of up to 4% of the finished pellet[2]. The second permits use of lignosulfonates in certain sprays used on food for human consumption[3]. These regulations illustrate the minimal pollution and toxicity effects of lignosulfonates.

Due to their availability, lignosulfonates have been used for road stabilization, especially in the vicinity of pulp mills, although investigations have been pursued in many geographical areas to evaluate the economics and permanency of such work. In 1939, the State of Washington built a lignosulfonate stabilized road, covered with a bituminous seal coat as a test project[4]. Prior to treatment, the average plasticity index of 10 samples from the section was nine with individual sample variations from 7 to 12. In 1947 a sample taken from the middle of the treated section had a plasticity index of 13 and the average of four samples taken from outside of the treated section was 6.5.

Laboratory tests were made in Canada in the mid-1940's on crushed gravel, clay stabilized gravel, and clay stabilized gravel treated with 1.2% lignosulfonate[5]. A bearing capacity increase was observed with the lignosulfonate. Later tests were made on cylindrical specimens of the same clay-stabilized gravel compacted at optimum moisture. Compressive strength and absorption of water by capillary action were measured on specimens containing 0% to 2% by weight of lignosulfonate. Compressive strength was increased with 2% lignosulfonate at moisture contents from 0% to 5% while water absorption was reduced, provided that initial moisture content was below 50% of the maximum that could be absorbed by the untreated gravel. It was concluded that excellent results could be obtained with compacted gravel-base courses treated with 2% lignosulfonate at optimum moisture content, and would permit the use of gravel otherwise rejected due to large quantity of fines even under cold climatic conditions.

Development of chemical mixtures to decrease undesirable lignosulfonate properties have been hampered by the chemical complexity of lignosulfonates themselves[6-8]. Progress has continued on a trial basis using and evaluating many secondary additives. As soils vary in chemical properties, individual soil mixtures must be thoroughly screened with lignosulfonates and secondary additives in order to indicate relative field performance.

Laboratory studies at Iowa State University in 1956 were made using a loess soil from Northeastern Iowa[9-11]. Several types of lignosulfonates were tested in both powder and solution forms. Each form produced similar results, although it was noted that powder was easier to use. Conclusions were that lignosulfonates alone were not promising stabilizers for loess soils, but indicated more favorable results with granular soils or soil-aggregate mixtures. It was also observed that lignosulfonates increased the plasticity index of soil, being greatest for soils having the highest clay and organic matter contents. Optimum lignosulfonate concentration was considered to be around 6% of the dry soil weight.

In Washington (1958) ammonium lignosulfonate was tested alone and in combination with portland cement and various asphaltic products for stabilizing several soil materials[12]. Laboratory studies used a silty sand, fine sand, crushed basalt rock, and sand-clay. It was observed that lignosulfonate acted as a dispersing agent by decreasing moisture content at maximum standard density. Compressive strength of soil increased with up to 3% addition of lignosulfonate solids. Capillarity was reduced, tending to seal the soil, being quite effective with 4% to 5% lignosulfonate solids. Heat curing increased strength of soil and lignosulfonate mixtures; a practical testing technique since many roadway surfaces commonly reach 140 °F. Experiments with combinations of lignosulfonate, portland cement and fine sand yielded unsatisfactory results. It was also found that lignosulfonates did not combine satisfactorily with liquid asphalts, asphalt cements or asphalt emulsions. To reduce leaching, a bituminous seal coat was recommended on roads stabilized with lignosulfonates.

Concurrently, at Iowa State University it was noted that lignosulfonate acted both as a binder and a dispersing agent[13,14]. As clay swells, pores become plugged reducing evaporation, permeability, and frost action. Treatment was recommended as either surface application or mixed in-depth.

Addition of secondary additives of protein-cation complexes improved compressive strengths of a silty loam soil[15]. Secondary additives of Zein, Armac T, Arquad 2HT and several metallic salts were used.

Lignosulfonate appeared to be most effective in quantities less than 2% when used with large organic cations as secondary additives, and much merit was given for more research on such combinations.

In 1959 work was done at Iowa State using an aggregate mix meeting Iowa State Highway Commission specifications for a surface course[16,17]. The study compared effects of sodium chloride, calcium chloride, lignosulfonates and molasses on moisture-density relationships, strength, moisture retention characteristics, and plasticity of a single material. All additives increased density and strength and only rock salt was ineffective for moisture retention. Lignosulfonates were not noted as superior to any of the other additives although having by far the greatest influence on plasticity index by increasing liquid limit and lowering plastic limit.

Work was also progressing at Iowa State on the addition of secondary additives with lignosulfonates to loess, dune sand, glacial till, and gumbotil[18-20]. Samples of lignosulfonates from several suppliers were tested and included both calcium and ammonium types. Secondary additives used were a fatty amine acetate and a quaternary ammonium chloride, Armac T and Arquad 2HT. Using either with any of the lignosulfonate samples, it was found preferable to mix the secondary additive with the soil before adding lignosulfonate. The amount of stabilizing agent required to adequately stabilize the soil increased with increasing clay content. Considerable waterproofing action occurred with increasing quantities of additives although 2% lignosulfonate was the maximum tested. It was noted with Arquad 2HT that calcium lignosulfonate was more beneficial than ammonium lignosulfonate. Supposedly, chemical reactions between additives occurred although no speculations were made toward their identification. It was concluded in all tests that the combination of additives was beneficial to stabilization.

Investigations at Iowa State were also made on the effect of aluminum sulfate as a secondary additive with calcium lignosulfonate[21,22]. Combinations of additives varied from 0-5% aluminum sulfate, 0-8% calcium lignosulfonate, and up to 8% of both. Soil used was a calcareous loess. Specimens were compacted, cured, and immersed in water for 24 hours prior to testing for unconfined compressive strength. An optimum combination of 6% calcium lignosulfonate and 8.5% aluminum sulfate was observed on the basis of densities and immersed strengths. An explanation of the possible formation of basic aluminum lignosulfonate and its composition was set forth.

In 1969 several roads were treated in Davidson County, South Dakota, to evaluate additives under actual field conditions[23]. One road was treated with a commercially produced stabilizer, ClaPac, which was found to be corrosive to equipment, cost $12.50 per gallon, and was therefore designated as undesirable for future work. Lignosulfonate was supplied at a cost of 25 cents per gallon plus freight charges to Davidson County (compared to a cost direct at paper mills of 6 cents per gallon). An untreated section was included adjacent to a lignosulfonate treated section. Both sections were primed with MC-70 and it was felt that the lignosulfonate section better absorbed the prime. This was followed with an armor coat of MC-800 and 5/8 in. maximum size pit-run aggregate. The performance of these sections in the years following is not known as no later reports were found in the literature search.

In 1970 research was done in Japan which shed some light on the stabilizing mechanism of lignosulfonate[24]. Solutions of measured quantities of sodium lignosulfonate were placed in flasks containing known weights of sodium montmorillonite and the mixture was permitted to establish adsorption equilibrium. The suspension was centrifuged to determine the concentration of sodium lignosulfonate in the supernatant liquid. Difference between this value and original concentration was the amount of adsorption. Original solutions were varied as to pH and amounts of sodium lignosulfonate and sodium chloride contained therein. It was concluded that adsorption is controlled by concentration of sodium ions existing in solution and pH (the lower the pH, the better the adsorption).

In 1970 Linn County, Iowa, stabilized a very sandy soil with lignosulfonate and applied a 4-in. asphaltic concrete surface. Estimated traffic was around 8000 vehicles per day. After two full years of traffic, performance was excellent.

Lignosulfonate stabilization must compete with other processes which have been tried and found successful at dust control. Shelby County, Iowa, used SC250 with a sand blotter for more than 20 years[25]. Lee County, Iowa, used lignosulfonate from a local source as a surface treatment for almost as many years (private communication, Lee Co. Engr. Office, June 1970). Linn County, Iowa, began several years ago to do in-depth treatment with ammonium lignosulfonate and also has utilized secondary additives in attempting to further improve stabilized roadways (private communication, Linn Co. Engr. Office, June 1970). Examples of the need for stabilization techniques for immediate use in dust palliation and surface improvement of secondary roads are shown in Figs. III-1 and III-2. Examples of lignosulfonate surface and in-depth treatments are noted in Figs. III-3 and III-4.

Fig. III-1. Dust from unpaved secondary road, Clinton County, prior to treatment with local industrial residual by-product.

Fig. III-2. Frost boil on unpaved secondary road, Linn County.

Fig. III-3. Vehicle traveling 40 mph on unpaved secondary road, Lee County, after several years of annual lignosulfonate surface treatment.

Fig. III-4. Vehicle traveling 40 mph on unpaved secondary road, Linn County, after lignosulfonate in-depth treatment.

LABORATORY INVESTIGATIONS

Materials

Previous work at Iowa State University[21,22] indicated aluminum sulfate as the most promising and economical secondary additive to reduce solubility of lignosulfonates. The Washington laboratory studies of 1958 suggested a chemical reaction when lime was added to ammonium lignosulfonate[12]. It was decided to consider these two secondary additives in powder form for experimental sections; aluminum sulfate, $Al_2(SO_4)_3 \cdot 18\ H_2O$, hereafter referred to as alum, and lime, calcium hydroxide, $Ca(OH)_2$. Lignosulfonate refers to ammonium lignosulfonate, commercially available in this area in a concentrated solution consisting of approximately 50-55% solids, with remainder as water. Distilled water was used in all laboratory work.

Samples of in-situ material were obtained from proposed test road sites to a depth of 4-6 in. Materials used in various investigations by previous researchers at Iowa State were available in quantity for laboratory studies. Selected were a limestone of the Pennsylvania System sampled from near Bedford, Iowa, and a weathered glacial till from the Cary Drift of the Wisconsin Glacier, obtained as overburden from Cooks Quarry near Ames, Iowa.

Specimen Preparation

Field material was passed through a 3/8-in. sieve which permitted the largest particle to be no greater than 1/5 of the final specimen diameter. A calculated quantity of additive was then added to a measured quantity of soil and thoroughly mixed by hand since quantities involved were small, and a mechanical mixer would not substantially improve the uniformity of the mixture.

The desired amount of lignosulfonate was usually diluted with 1 or 1.5 parts water prior to mixing with soil thus providing better dispersion of the lignosulfonate and simultaneously increasing the moisture content. Additional quantities of water were added and thoroughly mixed when necessary to arrive at final desired moisture content.

Cylindrical specimens, 2-in. diameter by 2-in. height, were compacted using field material and additives. Compaction followed a procedure developed by Chu and Davidson[26], similar to that used in laboratory work at Iowa State with lignosulfonates[17-21]. A 2-in. diameter cylindrical mold was filled with sufficient soil-chemical mixture to produce a 2-in.-high specimen after compaction. Compaction was achieved by a 5-lb weight falling a distance of 1 ft five times on each end of the specimen. Each specimen was extruded, weighed, height and diameter measured, marked for identification, and allowed to air cure at room conditions. Identical specimens were prepared from the same batch mix. Densities and moisture contents reported are averages of three or more specimens. Thus moisture contents and dry densities stated in following pages refer to average conditions at molding.

Test Procedures

Unconfined compressive strength of each 2 x 2 in. cylindrical specimen was measured at a constant deformation rate of 0.1 in./min. Strength of a specimen was determined by observing maximum deflection of a proving ring and converting deflection to load by means of a calibration chart.

The water immersion test performed on the cylindrical specimens required only a container of water greater than 2 in. in depth. Specimens were submerged in water and visual observations made and noted at intervals up to four hours. A specimen was considered to have failed when the 2-in. diameter top had disintegrated and only a cone of loose material remained. If after four hours a specimen had not failed it was immediately removed and tested for unconfined compressive strength. Strengths presented in the following pages are the average of at least two specimens.

Proctor density and plastic limit tests were performed in accordance with ASTM designation D 698-70 and ASTM designation D 424-59 respectively.

Evaluations

Unconfined Compressive Strength

Clinton County   Specimens of material from Clinton upland (glacial till) and bottomland (silty clay alluvium) were molded using several combinations of lignosulfonate, lime, and moisture contents. It was noted that dry densities of specimens at optimum moisture content differed by 10 pcf between these two sections. This indicated variability of untreated field material which had to be considered in evaluating relative performance of sections after treatment.

Unconfined compressive strength tests were made on specimens containing 1.5-3.0% lignosulfonate and 0-2.0% lime after air curing one and seven days (Figs. III-5 and III-6). Tests using upland material (Fig. III- showed decreasing strengths with increasing amounts of lime and lignosulfonate after both 1 and seven days air curing. Tests on specimens prepared from bottomland material indicated that treated specimens were not stronger after two days of air cure than untreated specimens; treated specimens after seven days of air cure showed a substantial improvement in strength (Fig. III-6).

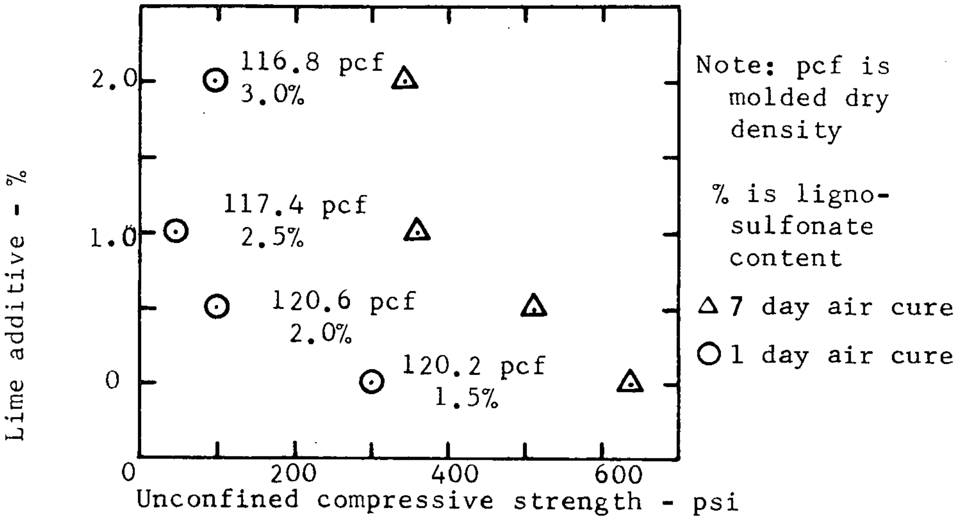


Fig. III-5. Effect of lignosulfonate and lime on unconfined compressive strength, Clinton upland material.

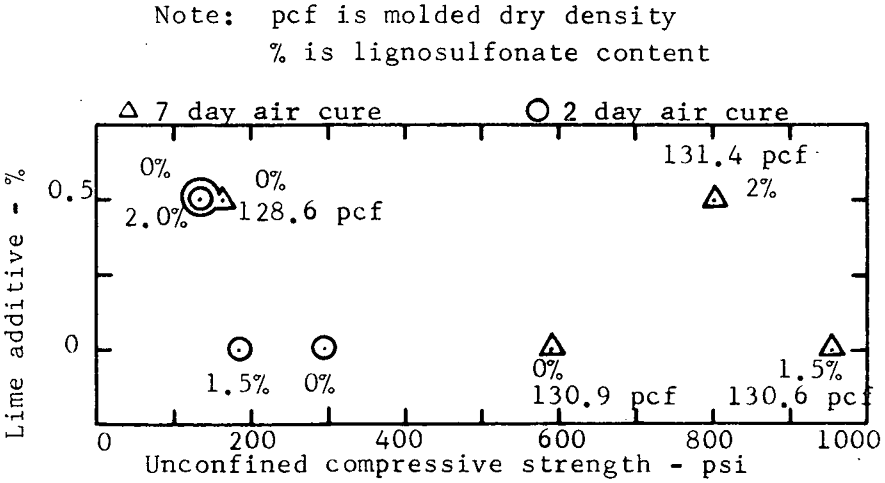


Fig. III-6. Effect of lignosulfonate and lime on unconfined compressive strength, Clinton bottomland material.

Economics were considered at this point. Calculations were made and will be considered in detail later, but in considering a concentration of 2% lignosulfonate solids by dry soil weight, cost of the lignosulfonate only, at the available commercial price, was over $6000 per mile. Inclusion of construction costs and cost of any secondary additive could have brought total cost above $10,000, and it was questionable that excess cost could be justified by increased benefits.

On the basis of cost analysis and test results the decision was made to continue investigations using a maximum concentration of 2% lignosulfonate solids both alone and with secondary additives. Drainage being best on the hill section made this most suitable for lignosulfonate alone as less leaching would occur. Material sampled from the hill section (a sandy loam) was used to compact 2 X 2 in. specimens at concentrations of lignosulfonate of 0, 0.5, 1.0, 1.5, and 2.0%. Specimens were compacted using several moisture contents at each concentration.

A plot was made of moisture content at molding versus lignosulfonate concentration. Points of equal dry density were connected with a line and the density contours in Fig. III-7 were formed. The dashed line locates moisture content at maximum density for given concentrations of lignosulfonate.

Examination of the contours indicated that when compacting below optimum moisture content, the effect on density by up to 1.5% lignosulfonate was slight. A concentration above 1.5% lignosulfonate sharply reduced moisture content required to produce a given density with constant compactive effort.

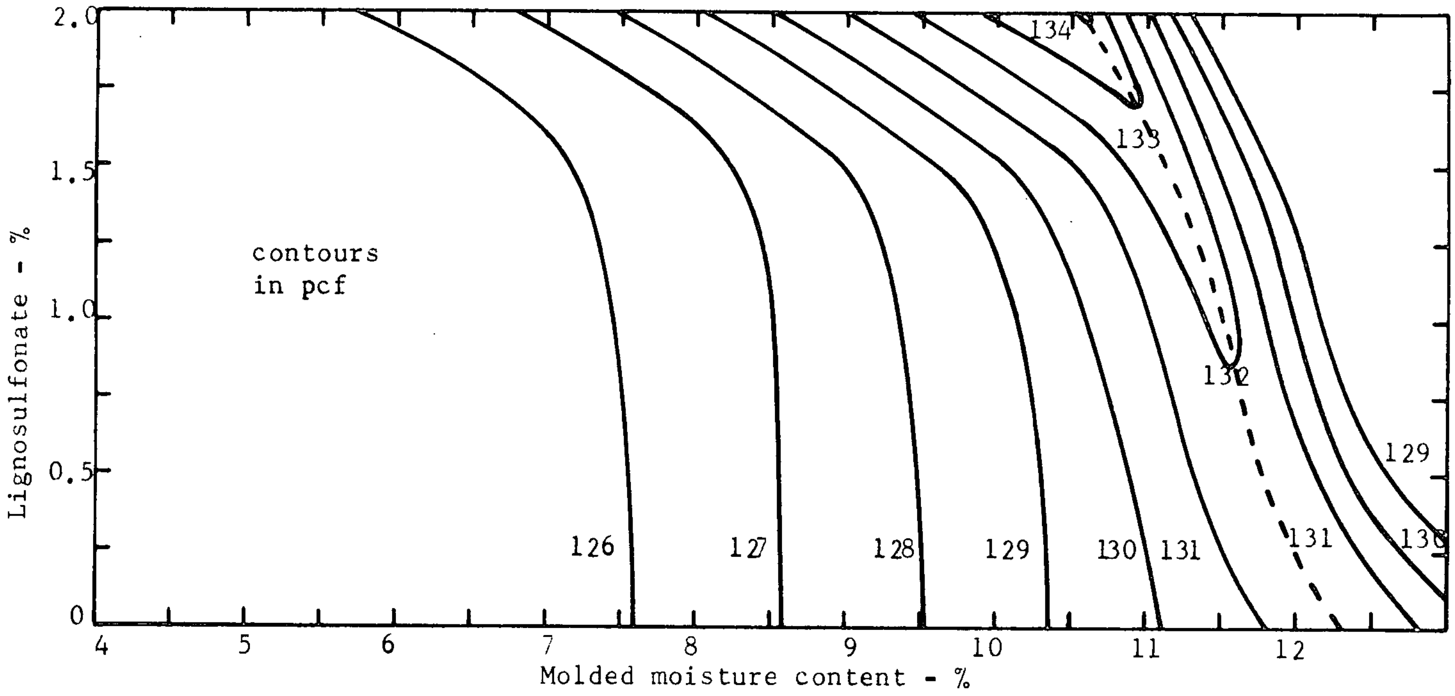


Fig. III-7. Dry density contours, Clinton hill material.

Further study of the contours in the area of optimum conditions showed that addition of 0.5% lignosulfonate reduced optimum moisture by 0.5%. An additional 1.0% lignosulfonate reduced optimum moisture another 0.7%. Greatest effect was by the first 0.5% addition although contours appeared to begin showing another large effect above 1.5% lignosulfonate content. Full extent of this trend was not shown since the highest lignosulfonate concentration investigated was 2%.

Lignosulfonate-moisture content-dry density relationships were demonstrated in an alternate manner in Fig. III-8. This was accomplished by selecting in Fig. III-7 a constant lignosulfonate concentration and at the intersection with each contour, plotting associated moisture content versus dry density values. Points for a given lignosulfonate content were connected and a family of curves formed. The peak of each curve represented optimum conditions for a given lignosulfonate concentration.

In Fig. III-8 (as in Fig. III-7) there were sharp changes in moisture-density relations at 0.5% and 1.5% lignosulfonate. However, in Fig. III-8 effects could be separated as to moisture content and/or dry density at optimum moisture-density. Less than 1.5% concentration of lignosulfonate affected mostly moisture content. Increase from 1.5% to 2.0% lignosulfonate increased dry density by 2 pcf and decreased moisture content at optimum by 0.6%. The magnitude of this decrease was comparable to the optimum moisture decrease resulting as lignosulfonate concentration was increased from 0-0.5%.

Contours were drawn in a manner similar to Fig. III-7 using 1- and 7-day air cured unconfined compressive strengths instead of dry densities

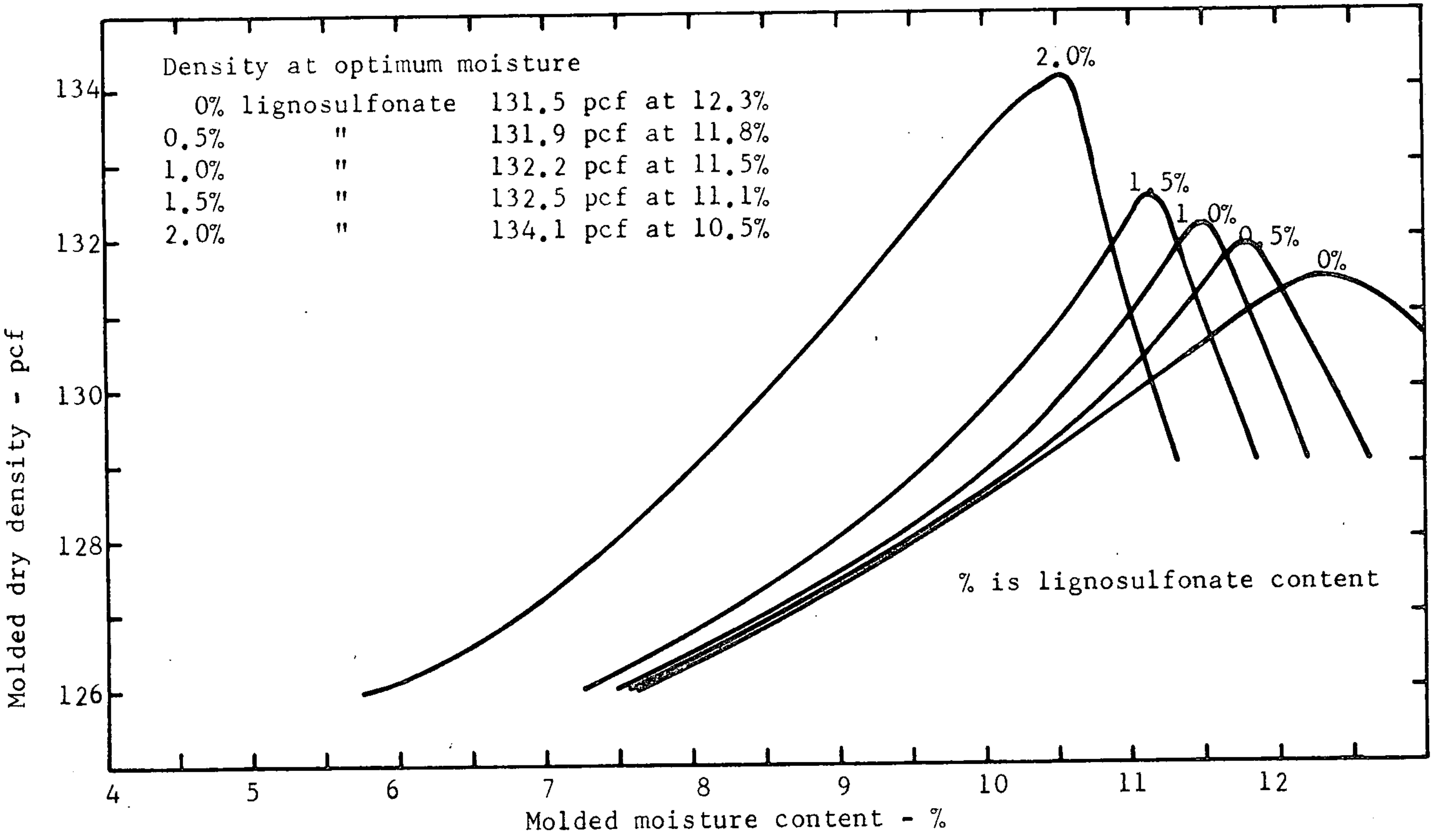


Fig. III-8. Effect of lignosulfonate content on moisture-density curves, Clinton hill material.

(Figs. III-9 and III-10). The dashed line indicating optimum moisture-density conditions from Fig. III-7 was superimposed on each.

Specimens with highest strengths were those molded below optimum moisture-density and contained lowest amounts of lignosulfonate. It was noted that after a 7-day air cure, strongest specimens were at slightly higher lignosulfonate and molded moisture contents than specimens producing highest strengths after a 1-day air cure. It was reasoned that with increased curing time specimens with higher lignosulfonate contents might have had higher strength gains.

Economics prevented using a large concentration of lignosulfonate, so a compromise decision was made to use 1% lignosulfonate concentration in test sections. Tests using lime as a secondary additive indicated decreases in strength and dry density with increasing amounts of lime. Recommendation was therefore made to use only 0.5% lime by dry soil weight as a secondary additive in the bottomland section.

Standard tests had been performed at the request of the Clinton County Engineer by the Iowa State Highway Commission on samples of a local industrial waste by-product, and showed that the material did not meet standard specifications for asphaltic cements. However, this tar-like residual material created disposal problems in light of enforcement of pollution control regulations. Laboratory tests at Iowa State University showed the strength of specimens compacted at 4% moisture and 4% residual to be 127 psi compared to 318 psi strength of specimens compacted with only water to optimum density. As the material was available in quantity and located less than a mile from the test road it was agreed to treat the upland section with this residual for observations of dust palliation and waterproofing capabilities.

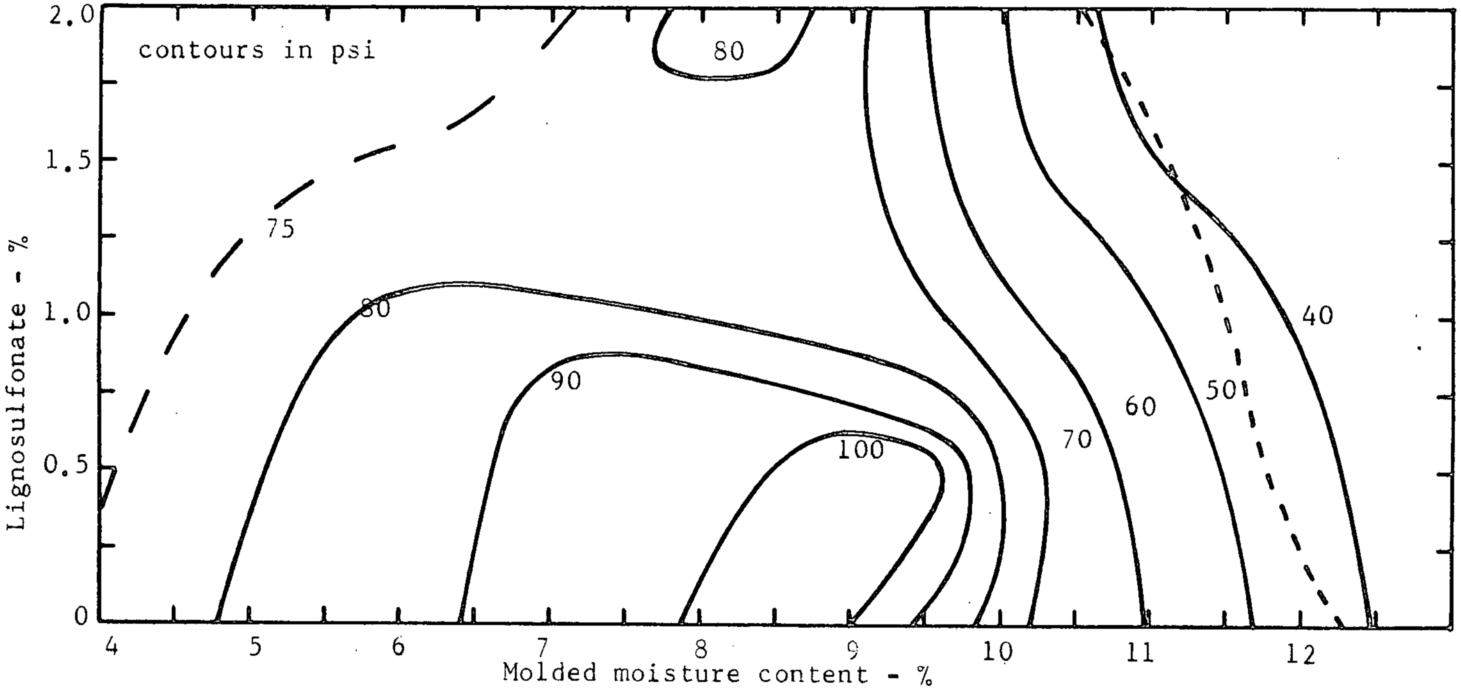


Fig. III-9. Unconfined compressive strength contours, 1-day air cure, Clinton hill material.

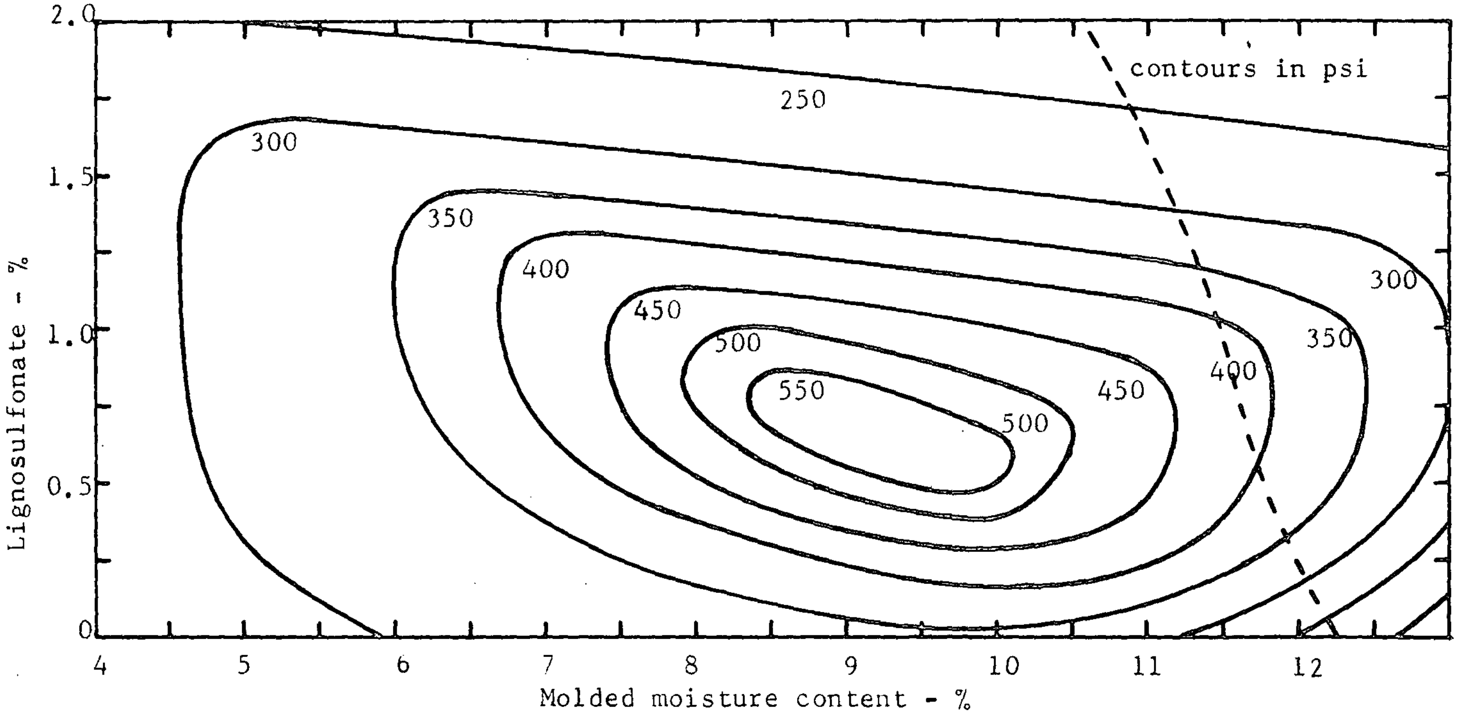


Fig. III-10. Unconfined compressive strength contours, 7-day air cure, Clinton hill material.

Floyd County Samples of in-place material (a sandy to silty loam terrace soil) were taken from a single Floyd County project near the YMCA Camp southeast of Charles City. Although no provisions included a secondary additive in field construction, laboratory investigations considered the effect of lime for comparison with materials from other counties.

Lignosulfonate concentrations and moisture contents used resulted from typical field treatment procedure which began with the concentrate solution consisting of about 50% lignosulfonate solids as provided by the commercial supplier. Further dilution was usually at the rate of 1 gal. concentrate to 1.5 gal. water and reduced the concentration of lignosulfonate solids in solution to 20%. Solution was then applied to, and mixed with, roadway material until the desired concentration of lignosulfonate in the soil mixture was attained.

A solution containing 20% lignosulfonate solids in water was prepared for laboratory use. Specimens, 2 x 2 in., were compacted at 4, 6, 8, and 10% concentrations of the solution on a dry-soil weight basis, producing specimen lignosulfonate contents of respectively 0.8, 1.2, 1.6, and 2.0%. Two additional series of specimens at the same concentrations were compacted with the inclusion of 0.5% and 1.0% lime respectively, added to the soil prior to the lignosulfonate solution.

As shown in Fig. III-11, at a given lime content, dry density increased with increasing lignosulfonate. Density increase was possibly due to two factors; namely increased lignosulfonate content lowered moisture content necessary to achieve a given density which simultaneously increased actual moisture content as lignosulfonate was added in a water solution. The fact that lignosulfonate and moisture contents

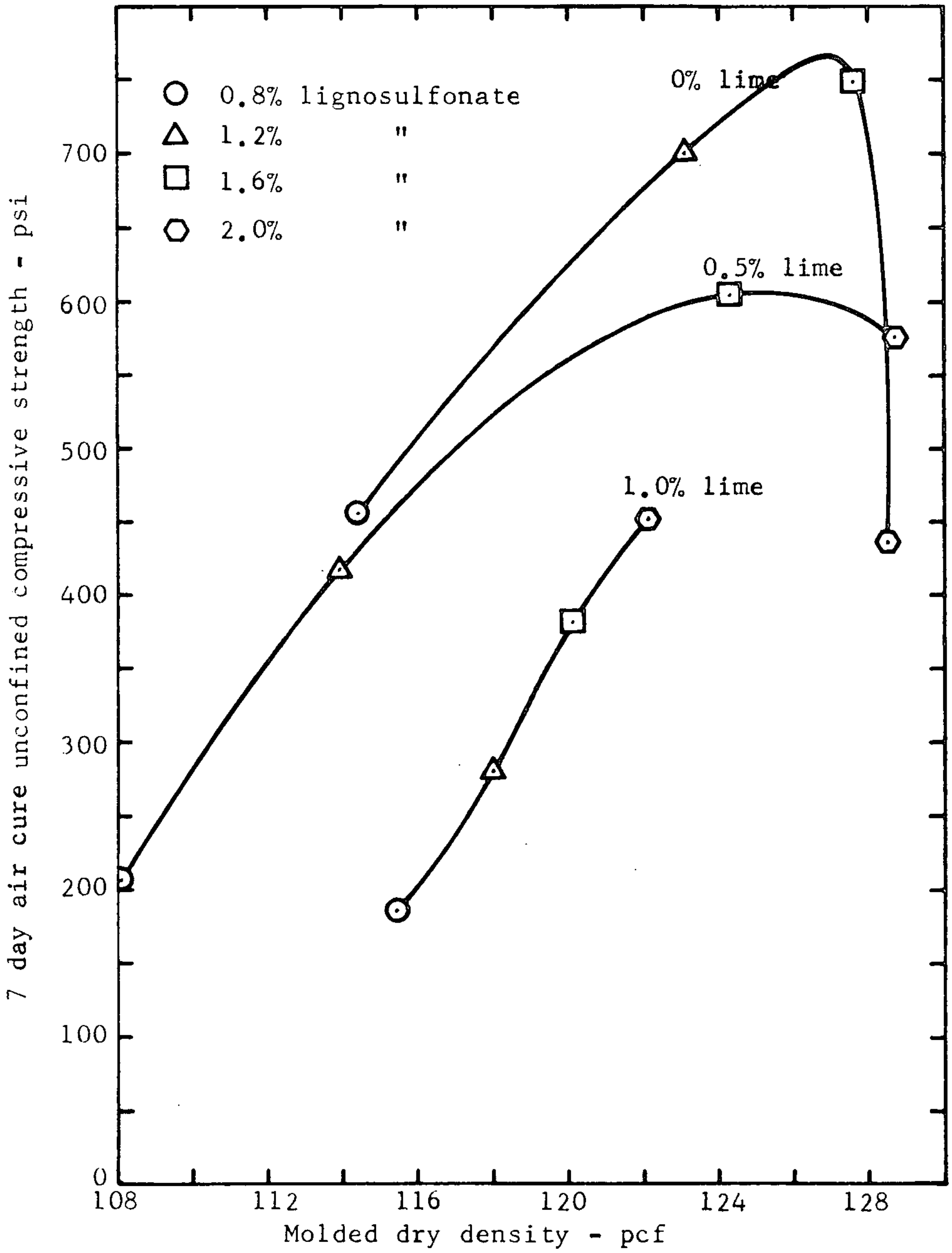


Fig. III-11. Relationship of lignosulfonate and lime contents, strength, and density, Floyd County YMCA road material.

increased together had to be considered when evaluating cause of variations in performance of specimens.

Specimens were tested for unconfined compressive strength after 1- and 7-day air cures. Strengths after one day were between 34 and

128 psi with no apparent effect due to increasing lime content. After seven days all specimens had several times as much strength as 1-day air-cured, and higher lime contents produced lower strengths, Figs. III-11 and III-12. Contradictory behavior of speciments with 2.0% lignosulfonate

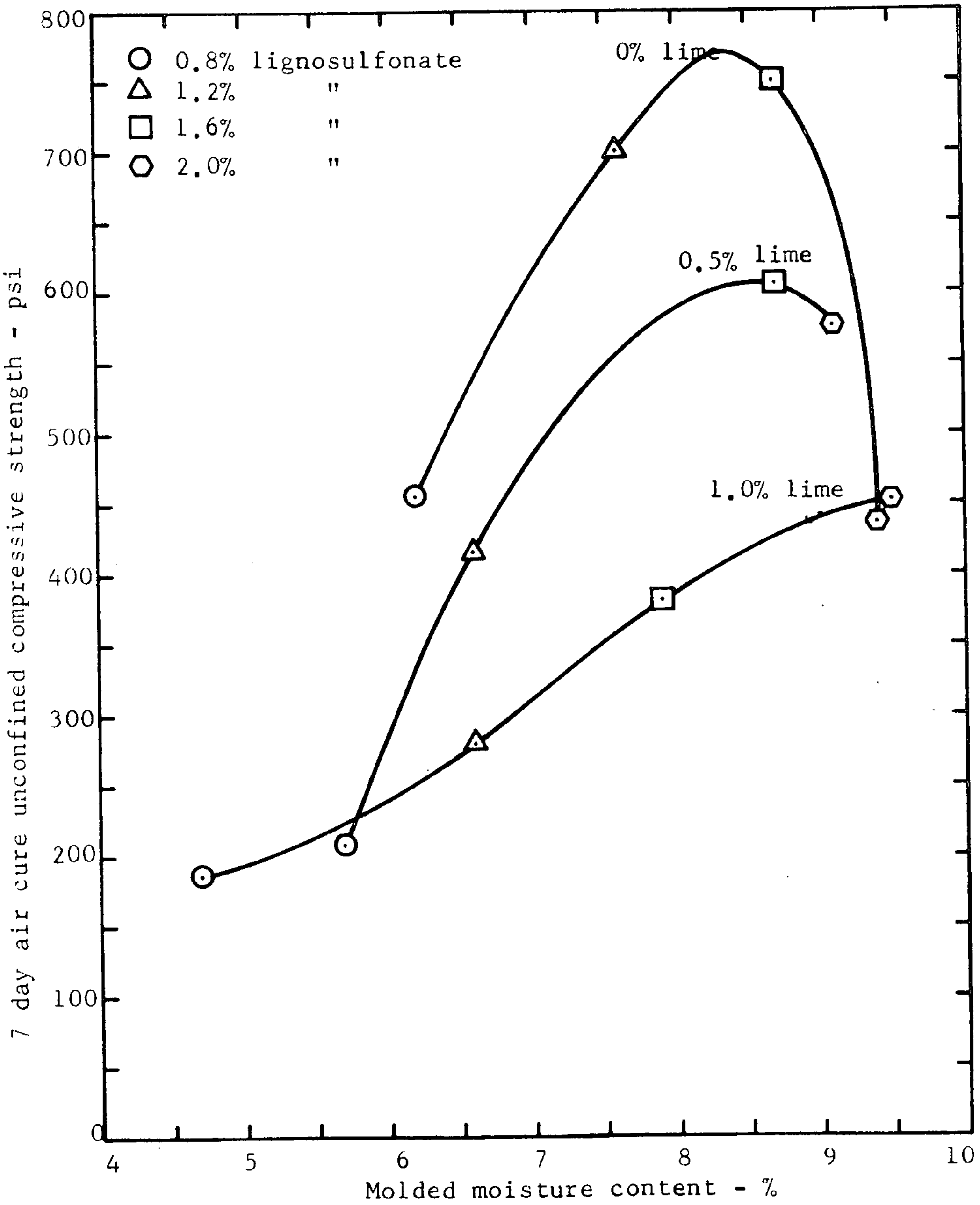


Fig. III-12. Relationship of lignosulfonate and lime contents, strength, and moisture content, Floyd County YMCA road material.

reflected necessity of a longer curing time for attainment of maximum strength due to high moisture content and lower rate of evaporation.

In Fig. III-11, a significant relationship was the occurrence of what appeared as a point of "optimum" conditions (i.e., the peak of each curve). For 0% and 0.5% lime specimens, maximum densities occurred with 1.6% lignosulfonate, and with 1% lime maximum density was indicated at greater than 2.0% lignosulfonate. However the curves varied as the moisture content at time of molding varied (i.e., higher density and strength at moisture contents near optimum). Therefore the amount of existing moisture in roadway material during construction would influence final compacted density.

In Fig. III-12, optimum 7-day air cure strengths appeared to occur at 1.6% lignosulfonate with 0% and 0.5% lime and at greater than 2.0% lignosulfonate for 1.0% lime.

Strength and density contours were drawn for specimens just discussed and are shown in Fig. III-13. Highest density specimens were at high lignosulfonate and low lime contents. An unexplained anomaly existed at low lignosulfonate contents above 0.5% lime concentration where an increased density resulted with increasing amounts of lime. Strongest specimens contained 1.5% lignosulfonate and 0% lime.

Better strengths were achieved at less than maximum lignosulfonate contents and corresponding lower moisture contents (Fig. III-13). As previously mentioned, this might be due to the effect of a reduced rate of evaporation and the need for longer curing times to reduce moisture content and produce higher strengths. Presence of moisture was clearly indicated by air-dry densities at testing being 3 pcf to 10 pcf greater than molded-dry densities with no change in specimen volumes. Least

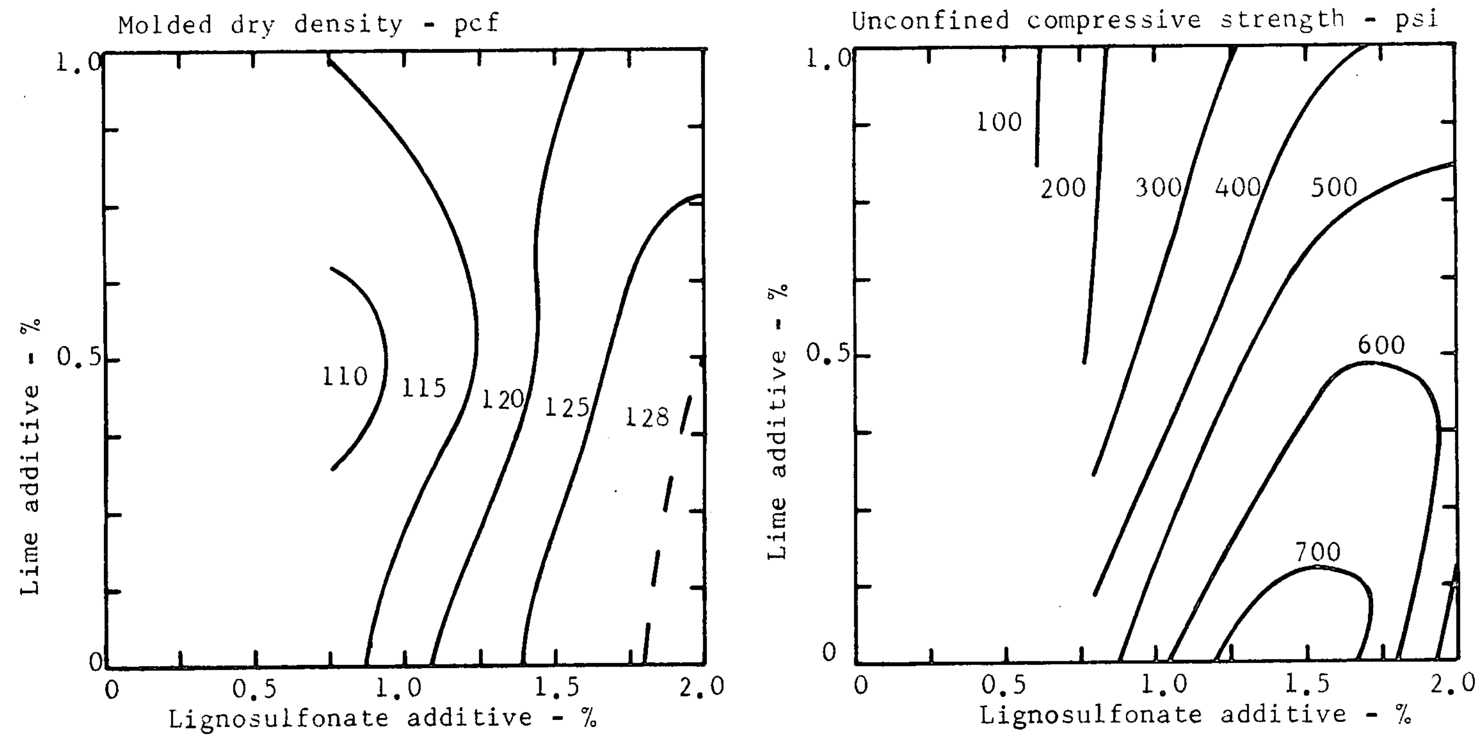


Fig. III-13. Dry density contours and 7-day air cure strength contours, Floyd County YMCA road material.

differences between molded-dry densities and air-dry densities were by specimens containing higher lime contents.

These studies with material from a single Floyd County project indicated general relationships between additive contents and specimen properties. Reduction in dry density at compaction was an expected response upon addition of lime, and this study showed a corresponding decrease in unconfined compressive strengths determined by specimens air cured for seven days. Indications were that increasing lignosulfonate content increased strength of specimens. However increasing lignosulfonate content also appeared to increase curing time required for attainment of maximum strength, which was previously observed in tests on specimens using material from Clinton County.

Linn County Based on results with Floyd County material, four sets of 2 X 2 in. specimens were prepared from original Marion roadway material (a gritty clay loam to silty clay) without any aggregate added. Each set contained 1% lignosulfonate with two of the sets containing 0.5% secondary additive as lime or alum. The fourth set was molded at a moisture content exceeding optimum.

Specimens were tested for general comparison with prior tests. Other roadway materials consisted mostly of aggregate which had been applied during original construction and maintenance. The Linn County soil contained no aggregate. Test results are shown in Fig. III-14. Densities were lower than most specimens from Clinton and Floyd County material by about 10 pcf although strengths were comparable. Similarity of the specimens molded at the same moisture content with varying additives may be noted. Densities varied less than 2 pcf and strengths were nearly identical. Thus addition of alum or lime did not significantly

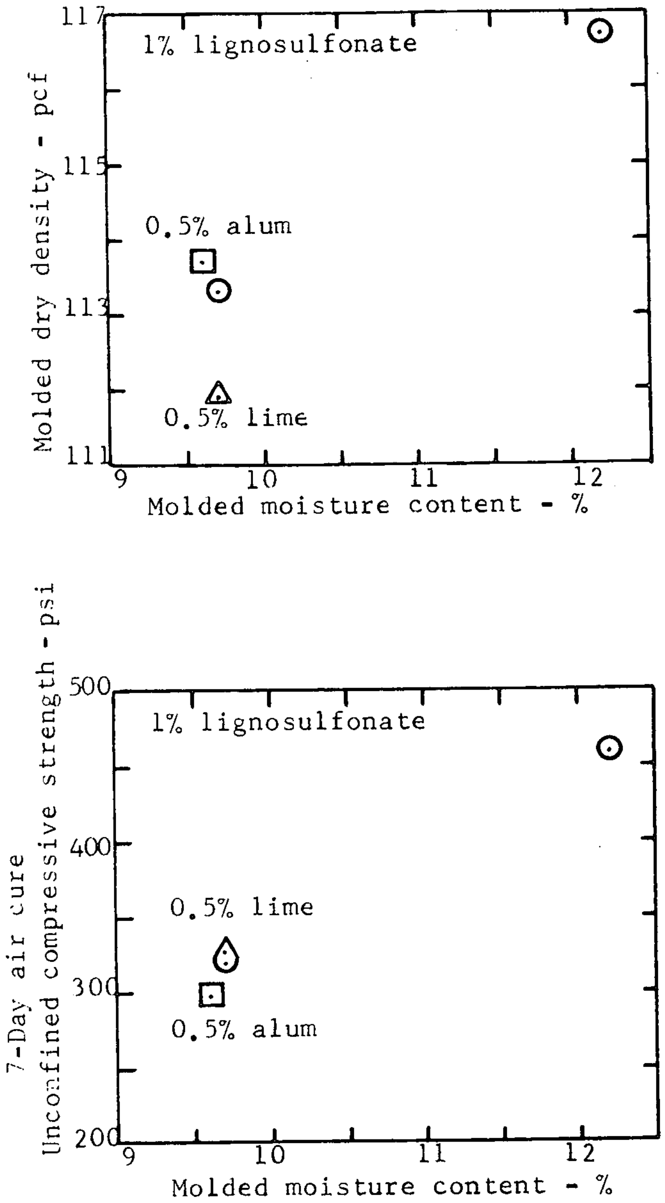


Fig. III-14. Relationships of additive and moisture contents, strength, and density, Marion test road material.

affect either density or strength. The specimen molded above optimum moisture content had higher density and strength. Therefore strength depended on density as controlled by moisture content when molded at constant compactive effort.

Water Immersion

An important part of the research carried out concerned the deleterious effects of water into such a system[13-15]. The result is a structure better able to perform satisfactorily in the presence of surface and subsurface water. However, as lignosulfonates themselves are water soluble, improvements have been somewhat temporary. Thus the presence of secondary additives to reduce solubility of ammonium lignosulfonate would be desirable.

Photographs of specimens during and after immersion are shown in Figs. III-15 and III-16. Durability to water immersion of various

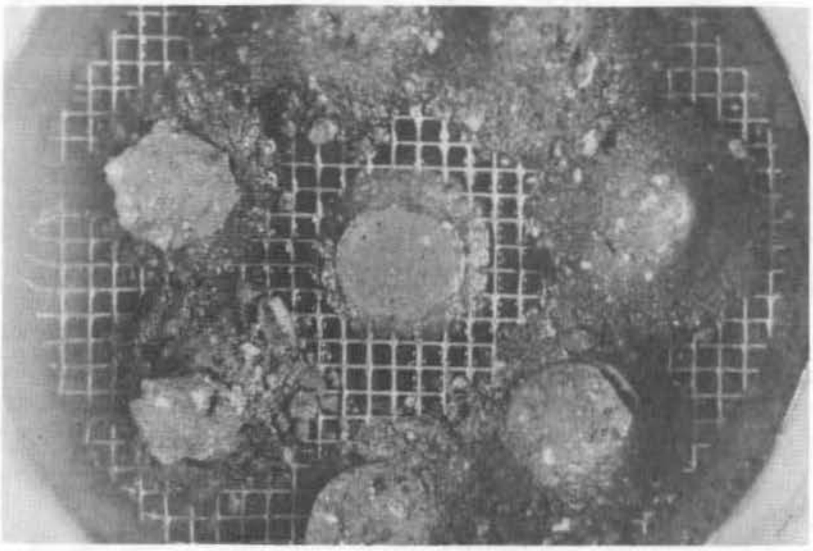


Fig. III-15. Water immersion, 4 hours, various additive contents, 1-year air cure, Floyd County YMCA road material.

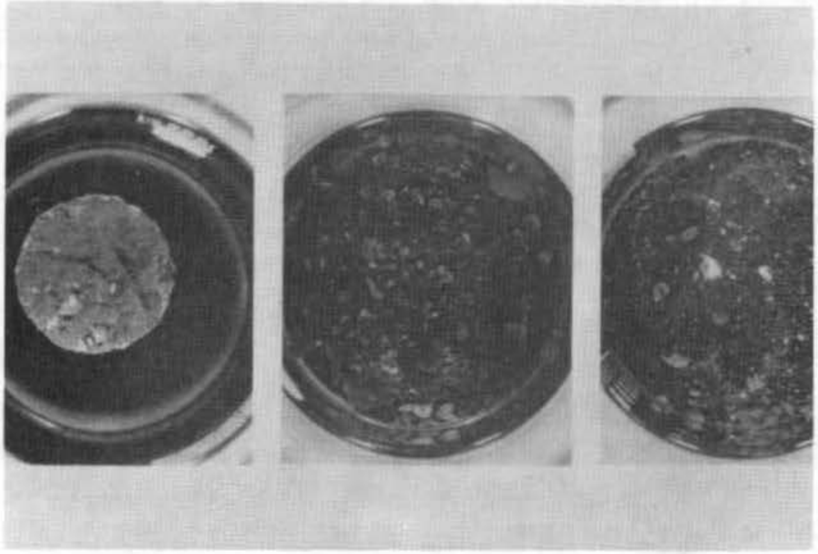


Fig. III-16. Water immersion, 90 minutes, decreasing additive contents, left to right, Clinton upland material.

specimens are plotted in Figs. III-17-21. The ordinate on each figure was time required for disintegration under water immersion and the abscissa was moisture content at molding. Similar figures could have

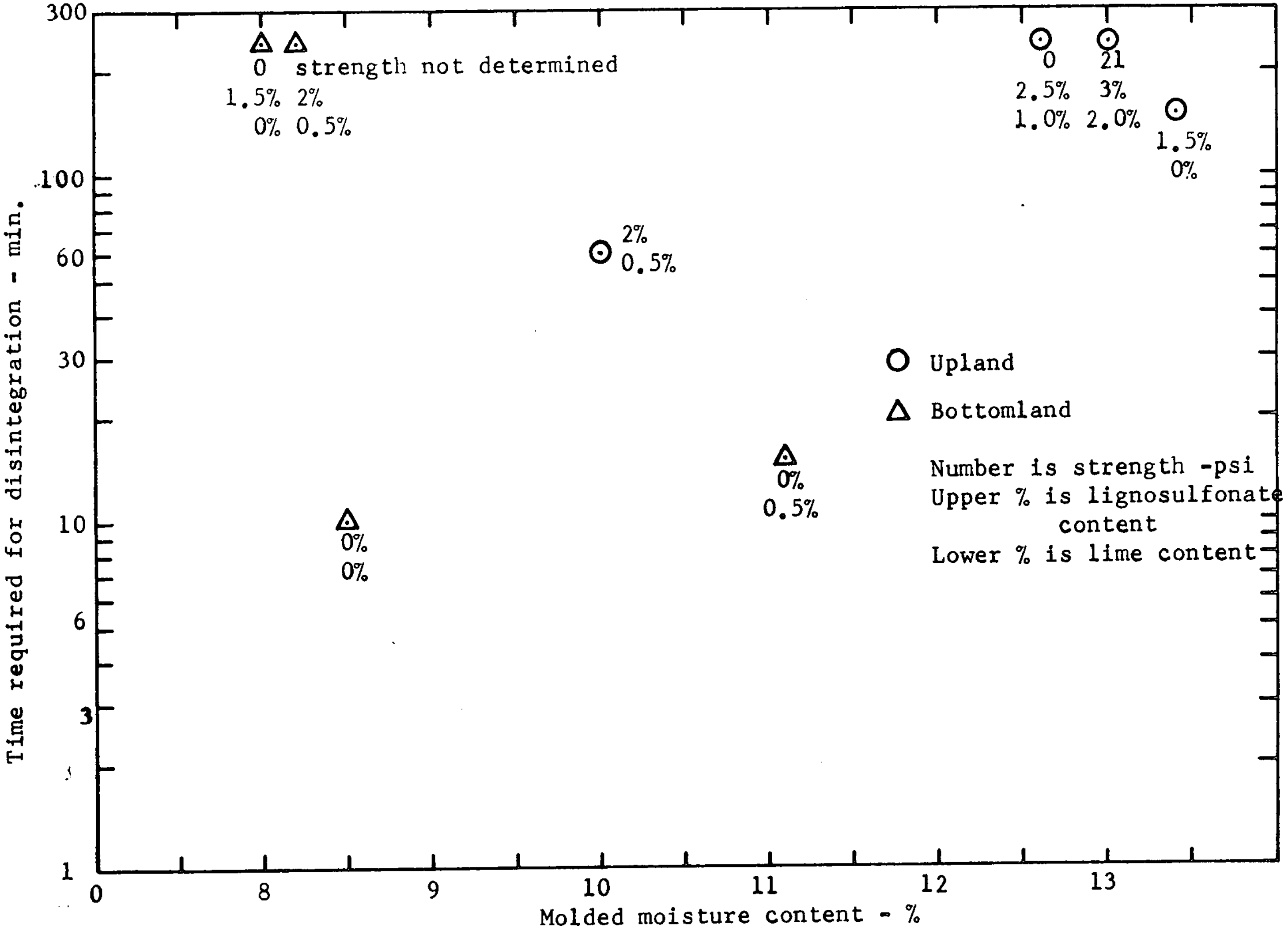


Fig. III-17. Water immersion, 1-year air cure, Clinton upland and bottomland material.

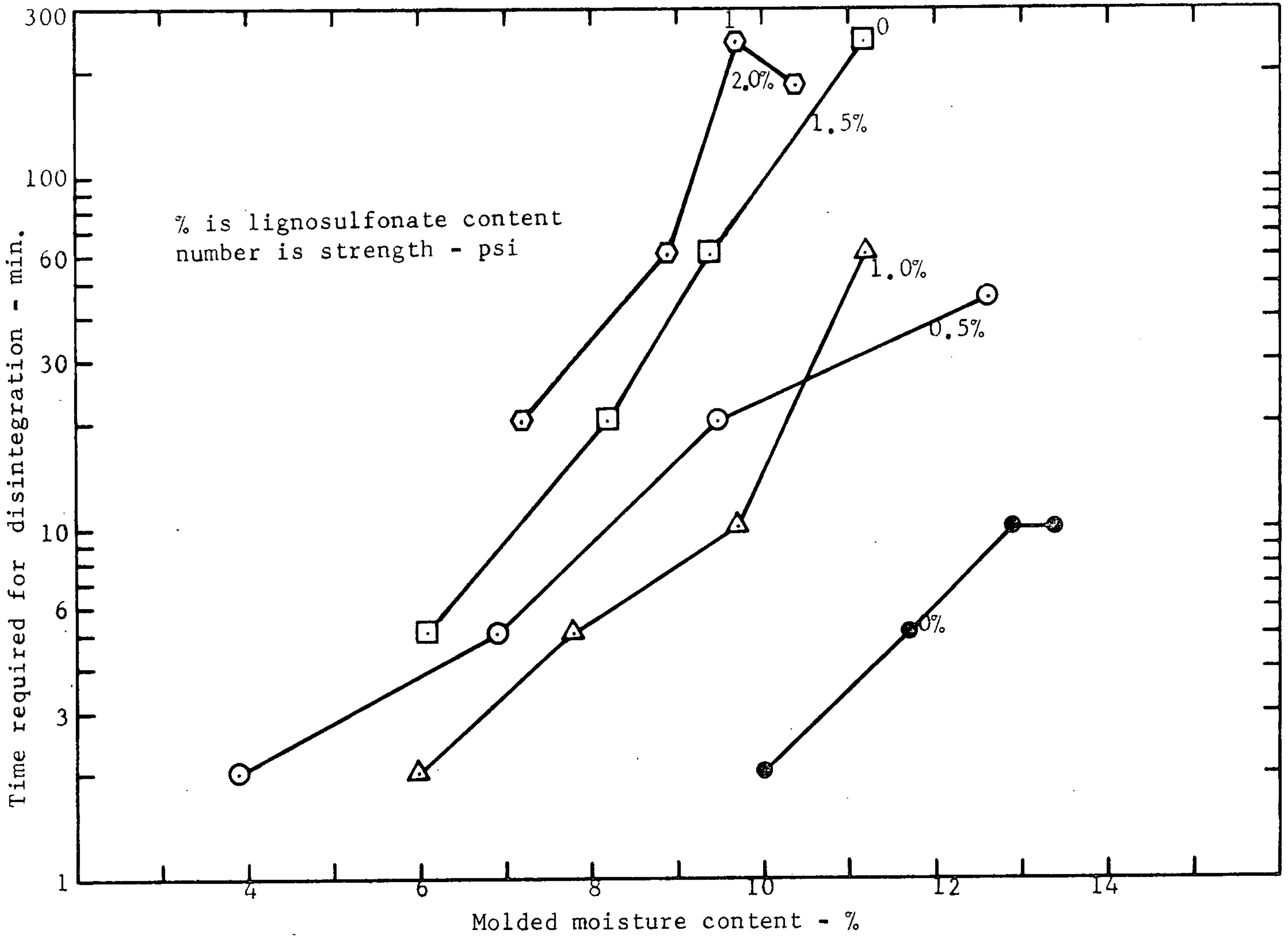


Fig. III-18. Water immersion, 1-year air cure, Clinton hill material.

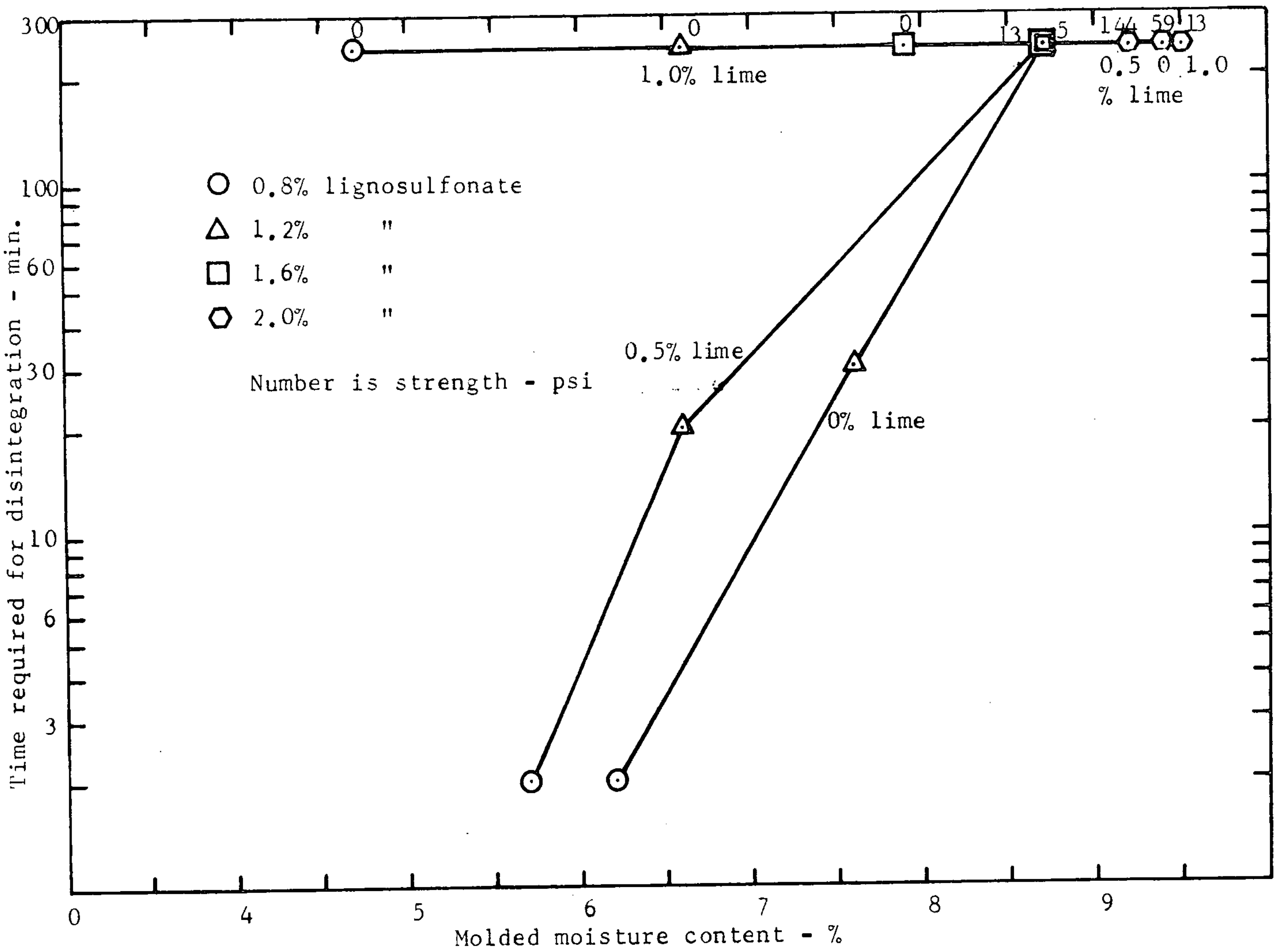


Fig. III-19. Water immersion, 1-month air cure, Floyd County YMCA road material.

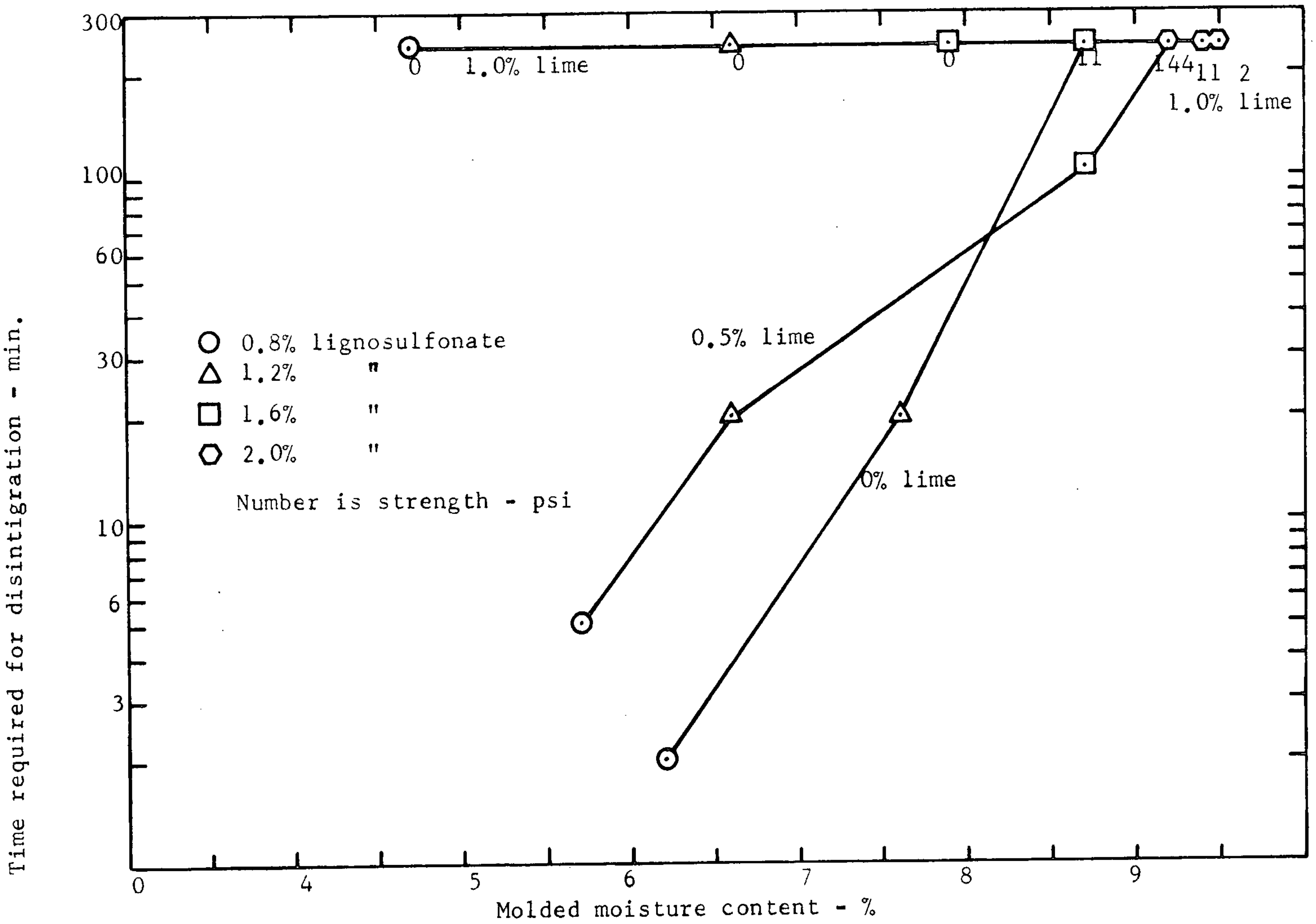


Fig. III-20. Water immersion, 1-year air cure, Floyd County YMCA road material.

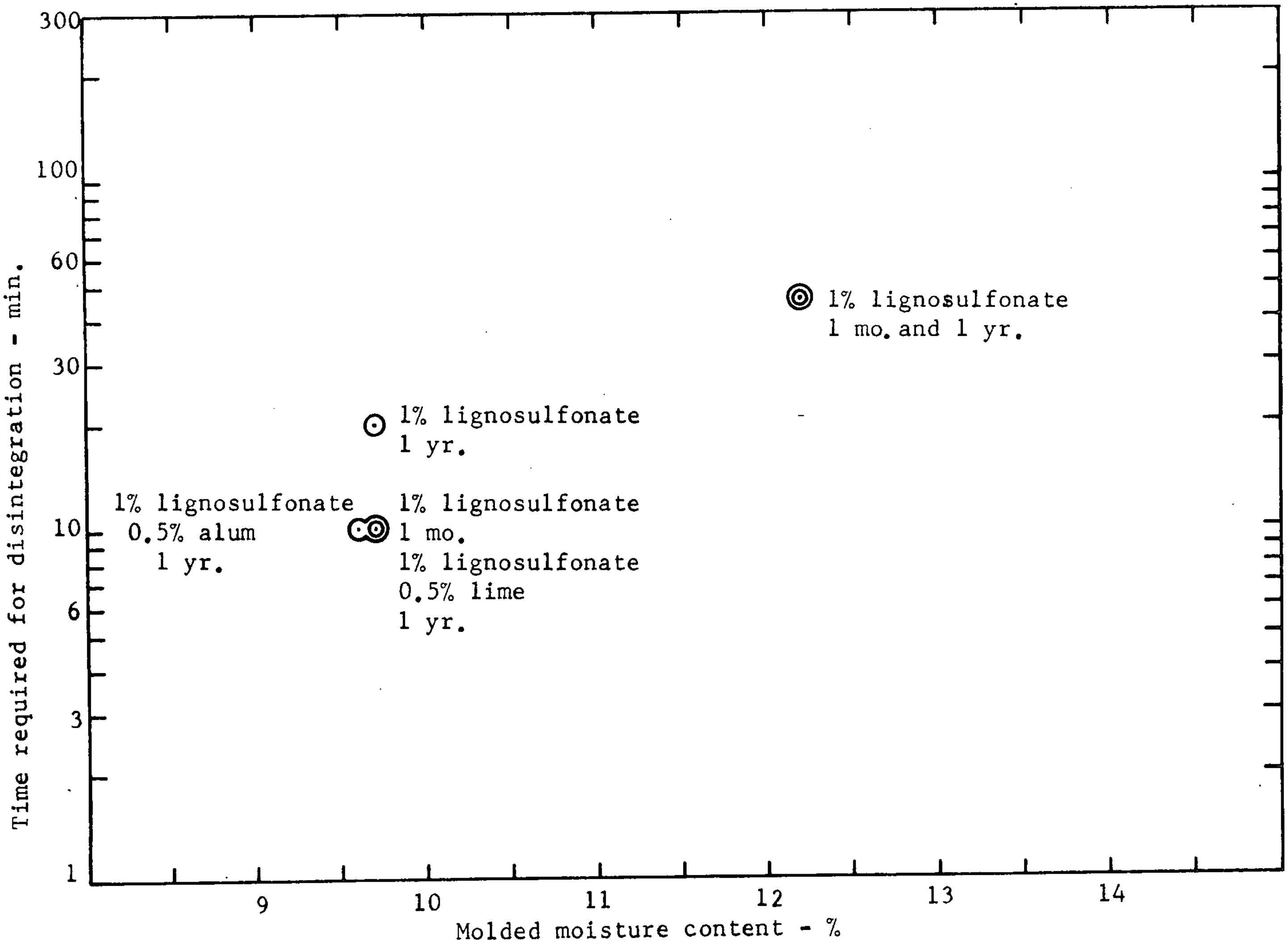


Fig. III-21. Water immersion, 1-month and 1-year air cure, Marion test road.

been produced by using dry density at molding as the abscissa instead of moisture content, due to density being dependent on moisture content at the time of molding.

A cursory examination of these figures generally showed that durability was improved by the addition of lignosulfonate, lime, or combinations of both. The capability of untreated specimens to resist disintegration was small and most untreated specimens failed within 10 minutes. Performance of treated specimens was improved to the extent that in several instances considerable strength remained after 4 hr and was noted on the figures.

Even though increased additive content generally increased durability to water immersion, greatest resistance to disintegration under water occurred with specimens containing 0.5% lime and 2.0% lignosulfonate (Figs. III-19 and III-20). Nearly identical specimens were tested after curing for 1 month and 1 year and the same strength was produced by both specimens after immersion for 4 hr.

Recognition was made in the same figures to specimens containing lignosulfonate and 1.0% lime. At all concentrations of lignosulfonate these specimens lasted 4 hr, although little strength remained. This was a good indication that the presence of both lime and lignosulfonate was superior to either one alone. Further evidence of this was seen in Fig. III-17 according to which the specimen containing 3.0% lignosulfonate and 2.0% lime had greater strength after immersion than any other treated or untreated specimen.

Specimens from the original soil of the Marion test road, which contained no aggregates as did the other field materials, did not seem to bear previous observations (Fig. III-21). Specimens containing lignosulfonate, lignosulfonate and lime, and lignosulfonate and alum all behaved similarly when molded at the same moisture content. It seems reasonable that individual characteristics could have been more clearly seen if additional specimens had been compacted using several moisture contents and an optimum density had been achieved.

For each curve in Fig. III-18, the specimen which was compacted closest to optimum moisture content (see Fig. III-8) was the most durable. This agrees with the intuitive deduction that the most dense specimen would contain fewest void spaces, and have least number of pore openings through which water could pass. Thus, restricting movement of water into a specimen, the ability to maintain strength for a longer period of time is a logical expectation.

An alternate explanation of improved resistance to immersion considered the cementing action of lignosulfonate. Instead of, or in addition to reducing water movement, lignosulfonate glued together adjacent soil particles. In order for specimen disintegration to occur, lignosulfonate had to be dissolved, which required time, and thus increased the time required for a specimen to fail.

This could be further supported by referring to Fig. III-10 and observing that addition of lignosulfonate increased strength. An enlightening area for further research would be the investigation of the effect of lignosulfonate and secondary additives on cohesion and angle of internal friction of soils as determined by triaxial shear tests. At this point, it is not yet possible to completely understand the precise stabilizing mechanisms of lignosulfonate. However results of investigations in Japan in 1970 did show adsorption of lignosulfonate on clay terminal surfaces and studied the factors affecting this activity[24].

Plastic Limit

Plastic limits were found for Bedford limestone using several combinations of lignosulfonate and lime (Fig. III-22). Results showed only small changes in plastic limit at 0% and 1% lime content even when amount of lignosulfonate was increased. However the effect with 0.5% lime content was quite large, going from over 21 at 0% lignosulfonate down to 16 at 1% lignosulfonate which could possibly be an optimum condition.

Determinations of plastic limits were made using lignosulfonate and lime or alum with till. This work showed the largest effect on plastic limit of varying amounts of lignosulfonate when lime was the secondary additive. Increase in plastic limit with lime and 0% lignosulfonate was due to the flocculating effect of the lime. Destruction of the flocculating effect of lime by the dispersant effect of lignosulfonate was shown by decreasing plastic limit with increasing lignosulfonate at a constant 0.5% lime content. The addition of 0.5% alum resulted in only minor changes in plastic limit with increasing lignosulfonate contents.

The stabilizing process of lignosulfonate and secondary additives appeared to be different between till and crushed limestone as indicated by different response of plastic limit to additives. Additive response by a combination of these two types of soils would be expected to vary as soil proportions varied. As will be shown later with data from the Marion test road, performance of sections containing low amounts of aggregate definitely was not the same as that of sections with a high aggregate content.

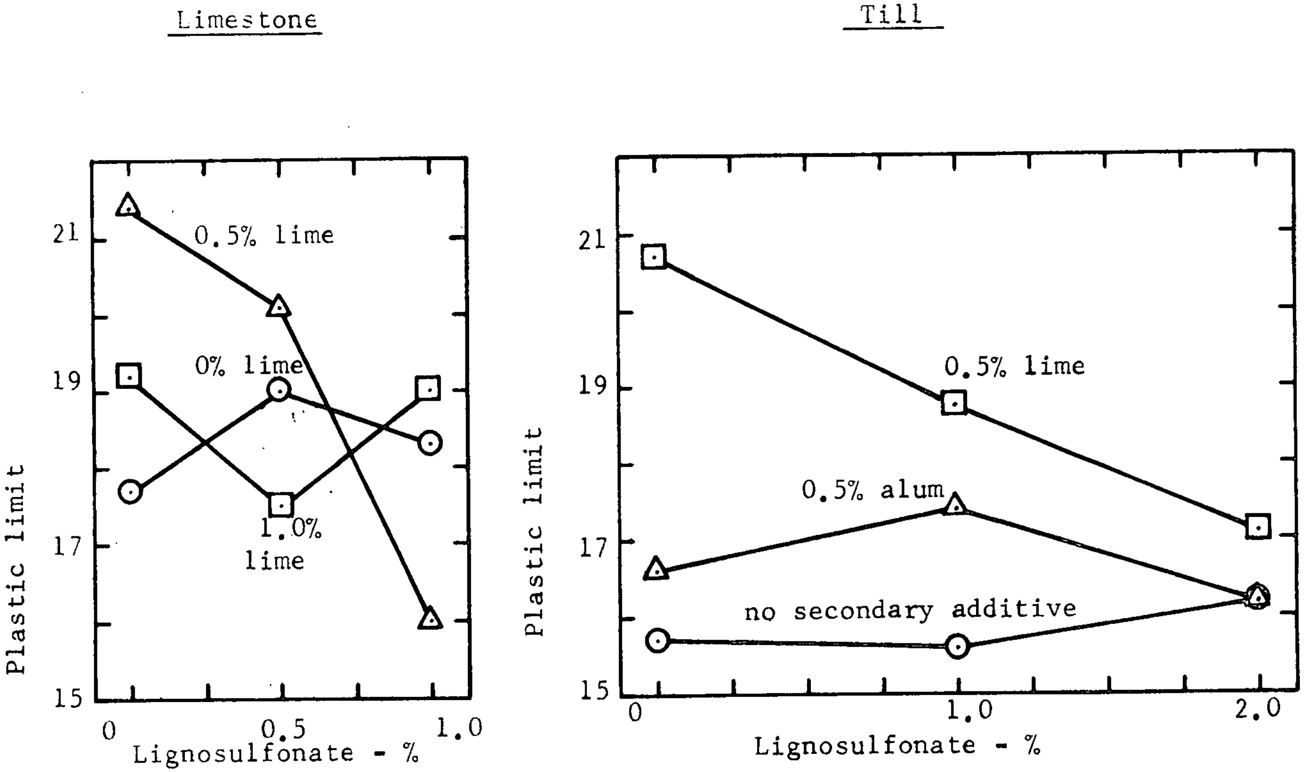


Fig. III-22. Plastic limit, Bedford limestone and Cary glacial till.

Proctor Density

Proctor density investigations consisted of performing moisture-density studies at lignosulfonate contents of 0.5, 1.0, 1.5, and 2.0% as well as 1% lignosulfonate plus 0.5% lime or alum. Data considering only lignosulfonate were used to produce the density contours (Fig. III-23). At 1% lignosulfonate, maximum density was achieved at just over 8% moisture content. Going any direction from that point resulted in a density decrease. Above 1.5% lignosulfonate content densities again began increasing.

A similar type of behavior was noted in previous density contours (Fig. III-7) which had two different areas of influence, one below 0.5% lignosulfonate content and another above 1.5%. Again here were indications of a possible ion saturation stabilization[24] with very small amounts of lignosulfonate and an adhesive stabilizing mechanism with larger concentrations.

A family of moisture-density curves was constructed from Fig. III-23 and is presented in Fig. III-24. Proctor density information for alum and lime with 1% lignosulfonate is also noted on these curves. Lime densities were lower than the 1% lignosulfonate curve by several pcf. The alum curve almost coincided with the 1% lignosulfonate curve near the peaks.

Based on this density study it was expected that field trials might result in 1% lignosulfonate producing highest treated dry densities and 1% lignosulfonate plus 0.5% lime producing lowest dry densities. Also the moisture content at maximum density for 1% lignosulfonate would be increased 2% by the addition of 0.5% lime as a secondary additive. No change would be expected in either dry density

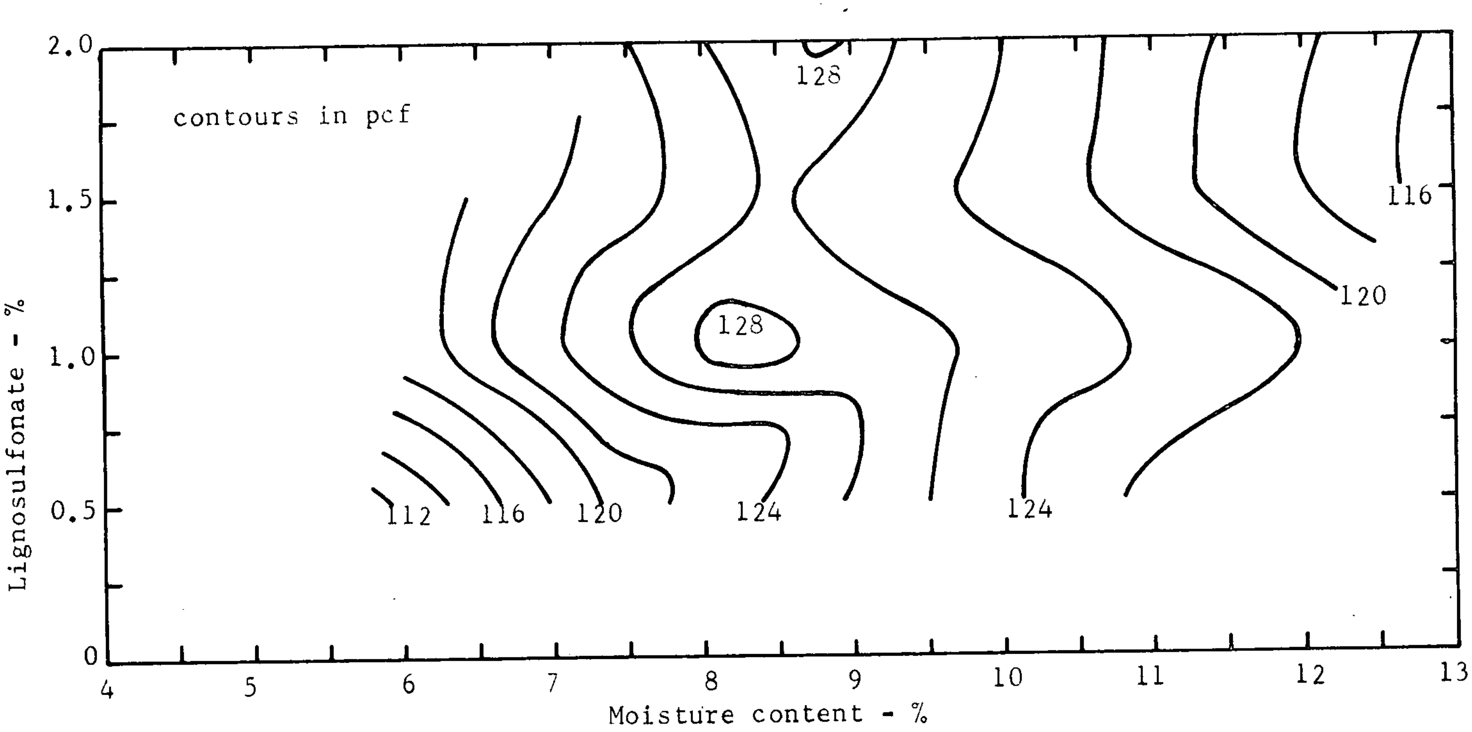


Fig. III-23. Density contours, Bedford limestone.

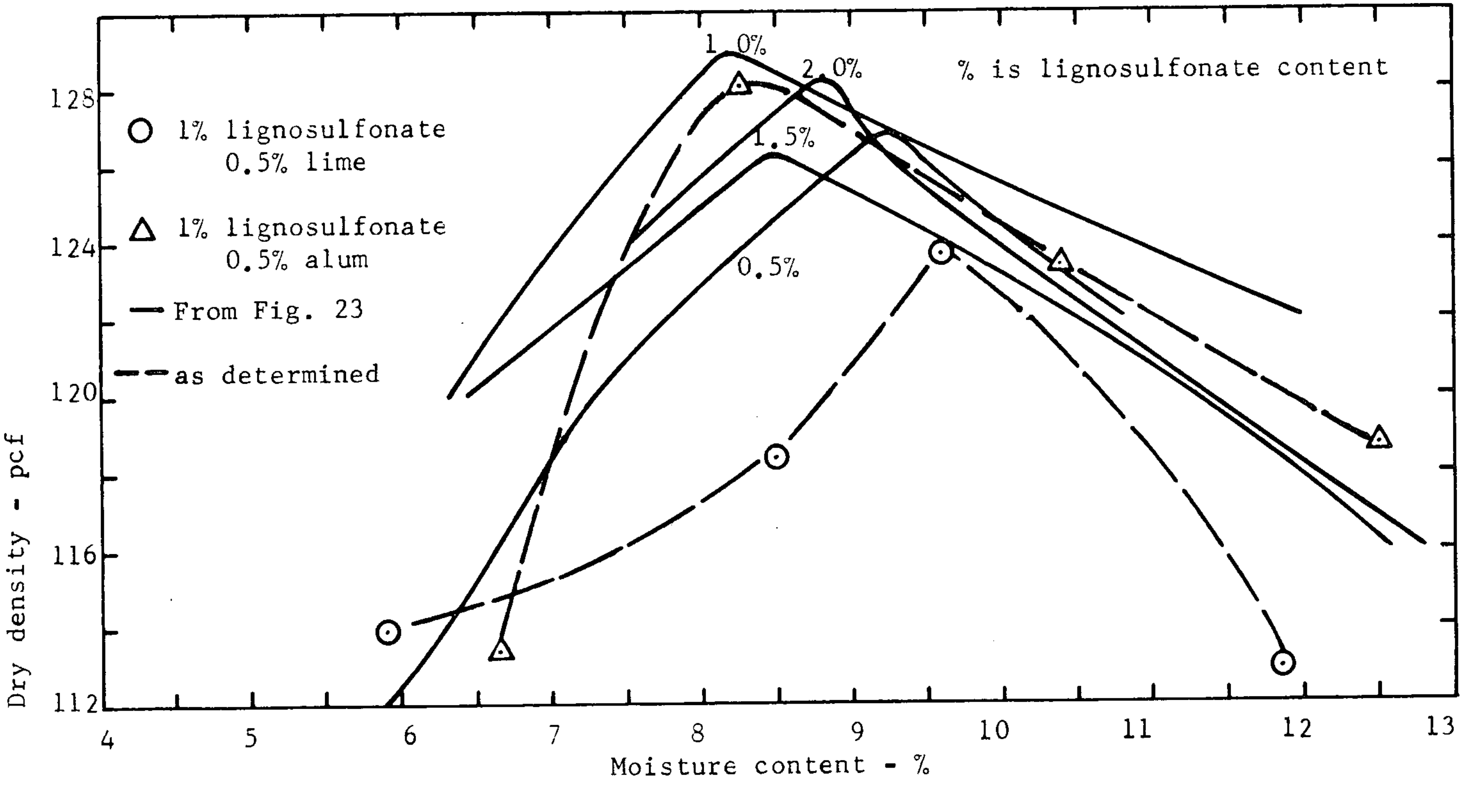


Fig. III-24. Effect of lignosulfonate and secondary additive content on moisture-density curves, Bedford limestone.

or optimum moisture content by the use of 0.5% alum as a secondary additive to 1% lignosulfonate. Results of density and moisture content determinations made on the Marion test road will be presented later. Let it be noted at this point only that these field results did not verify the tendencies indicated by the Proctor density study.

TEST ROAD CONSTRUCTION

Laboratory screenings previously discussed provided the basis for additive recommendations made to participating counties. Background experience from projects which were completed anywhere from one to 20 years ago adequately demonstrated construction procedures and made available several practical methods for the treatment process. Generally similar procedures were used in constructing all test roads with individual counties using existing equipment of comparable capability. A multipass mixer was reasonably successful in obtaining a homogeneous mix. All construction was accomplished by the county using its own men, materials, and equipment.

County personnel cooperated fully and often provided services in excess of their obligations. Their skill in operating equipment and understanding project goals helped to provide careful control during construction. Success of this research was greatly facilitated by their willing assistance.

Clinton Test Road

As previously indicated it was recommended to treat the hill section (sandy loam) with 1% lignosulfonate, the bottomland section (silty clay alluvium) with 1% lignosulfonate plus 0.5% lime, and the upland section (glacial till) with the residual material. Recommendations were approved by Clinton County. Construction was accomplished in August 1970 over a period of several days, finishing one section before the next was begun. The sequence of procedure was: (1) scarify to desired depth and pulvimix; (2) add powdered secondary additive if required and pulvimix; (3) spray on lignosulfonate and pulvimix; and (4) compact with sheepsfoot roller and tighten surface with light shot of lignosulfonate followed by rubber tired rolling. Procedure for the upland section was similar except that the residual material instead of lignosulfonate was used and the light top shot was omitted. Examples of the construction procedure are shown in Figs. III-25 and III-26.

Two methods of lime distribution were tried. Dumping the contents of a bag at intervals and dragging with a spike tooth harrow was not as successful as using a sand spreader. Figures III-27 and III-28 show

Fig. III-25. Lime spreading during construction, 1970, Clinton test road.

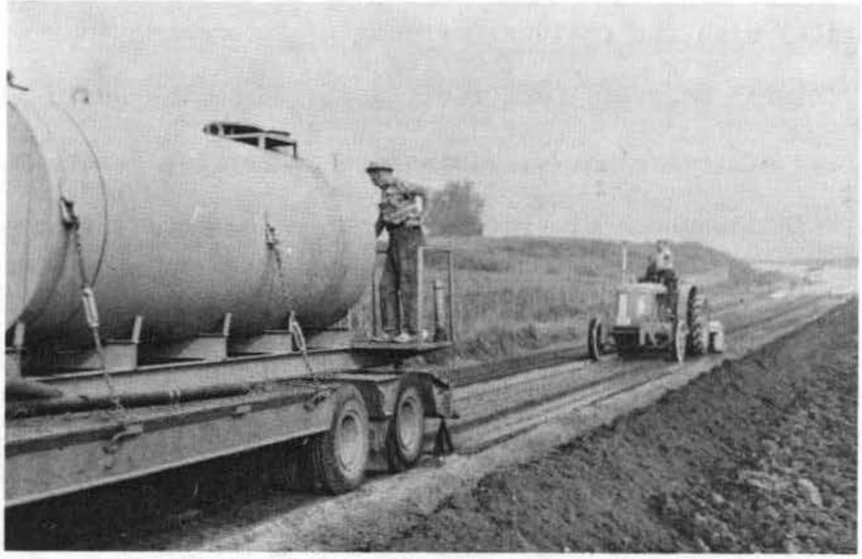


Fig. III-26. Spray application and mixing of lignosulfonate, 1970, Clinton test road.

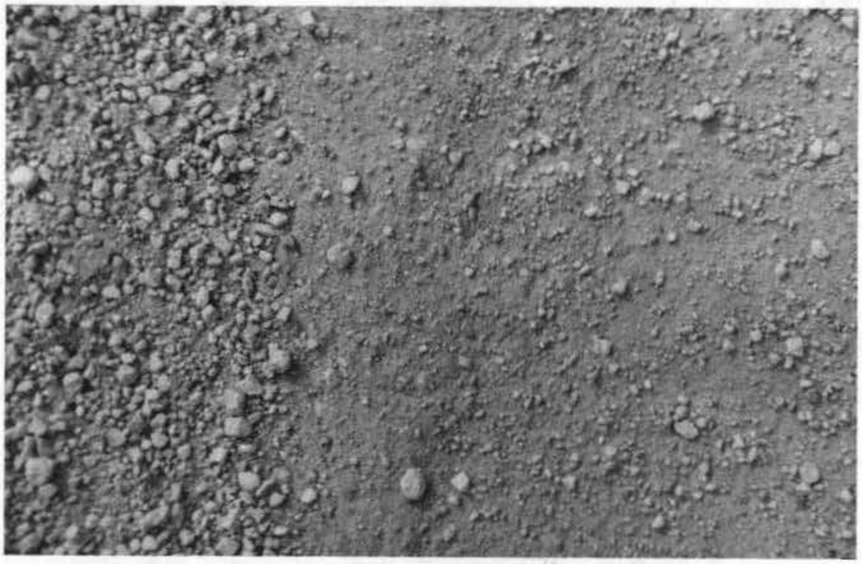


Fig. III-27. Close-up view prior to construction of 1% lignosulfonate plus 0.5% lime additive section, 1970, Clinton test road.

close-up views of the bottomland section before treatment and immediately after compaction. Inclement weather interfered with the construction of the residual section. Before recommended density had been achieved

Fig. III-28. Close-up view immediately following compaction, 1970, Clinton test road.

heavy rain saturated the section and traffic produced deep rutting and nearly impassable conditions. This section was then torn up and turned over continuously for several hours until a workable moisture content made suitable recompaction possible. Final depth of treatment of all sections was 6 in.

Floyd County YMCA Road

The roads constructed in Floyd County were used merely for observation as no provisions for controlled construction were obtainable. Therefore testing was planned to be minimal and was further reduced when rainy weather delayed construction.

Marion Test Road

The opportunity to build a number of controlled test sections provided the ideal situation to more fully evaluate factors influencing dust palliation and surface improvement. Not only could different additive combinations be investigated but several aggregate proportions could be accomplished. Relative performance of sections could hopefully be extended to provide recommendations for extensive treatment of unpaved secondary roads throughout the state.

Even with the 6000-ft length available, only a limited number of sections could be reasonably constructed. The first decision concerned the aggregate proportions and was based on the general application rate by counties of about 1800 tons of aggregate per mile. The opposite extreme would have been no aggregate whatsoever. However the dispersing properties of lignosulfonate are such that some aggregate is necessary to provide traction in wet weather. An intermediate amount of aggregate was also considered worth investigating, so 400, 1000, and 1800 tons per mile were selected as the applied rates providing minimal, intermediate, and near standard application rates.

On the basis of laboratory studies and previous experiences it was shown that 1% lignosulfonate solids improved stability. Doubling this amount did not double the improvement although the material cost was twice as much. Calculations showed total cost using 2% lignosulfonate to approach $10,000 per mile. Due to the higher cost of 2% lignosulfonate stabilization, it was decided to consider 1% lignosulfonate concentration for the majority of the sections. In order to observe the effect of varying lignosulfonate content on low aggregate soils, 2% lignosulfonate was used on one section containing 400 tons per mile of aggregate.

Laboratory studies showed favorable improvements by the addition of 0.5% lime as a secondary additive with a 1% lignosulfonate concentration. This proportion of additives was used in one section of the Clinton test road which had performed satisfactorily since completion. Therefore the same additive contents were used in sections of the Marion test road at various aggregate contents. For comparative purposes between alum or lime as a secondary additive, separate test sections were constructed using 0.5% of each secondary additive with 1% lignosulfonate and various aggregate contents.

Section lengths of 500 ft were felt to be a practical length both for equipment during construction and for later testing. Even at this length dust collectors could not extend very far transversely to roadway centerline before dust from adjacent sections would interfere with measurements. Complete sectioning of the Marion test road is shown in Fig. III-29.

Figures III-30 and III-31 show typical road conditions before construction in fall 1970. Prior to scarification the required amount of aggregate was trucked in and blade spread. As a matter of construction convenience as many sections were worked on simultaneously as were practicable. Excess water during construction of the 2% lignosulfonate section made necessary extra blading until a moisture content low enough for satisfactory compaction was achieved. Final depth of treatment of all sections was 4 in.

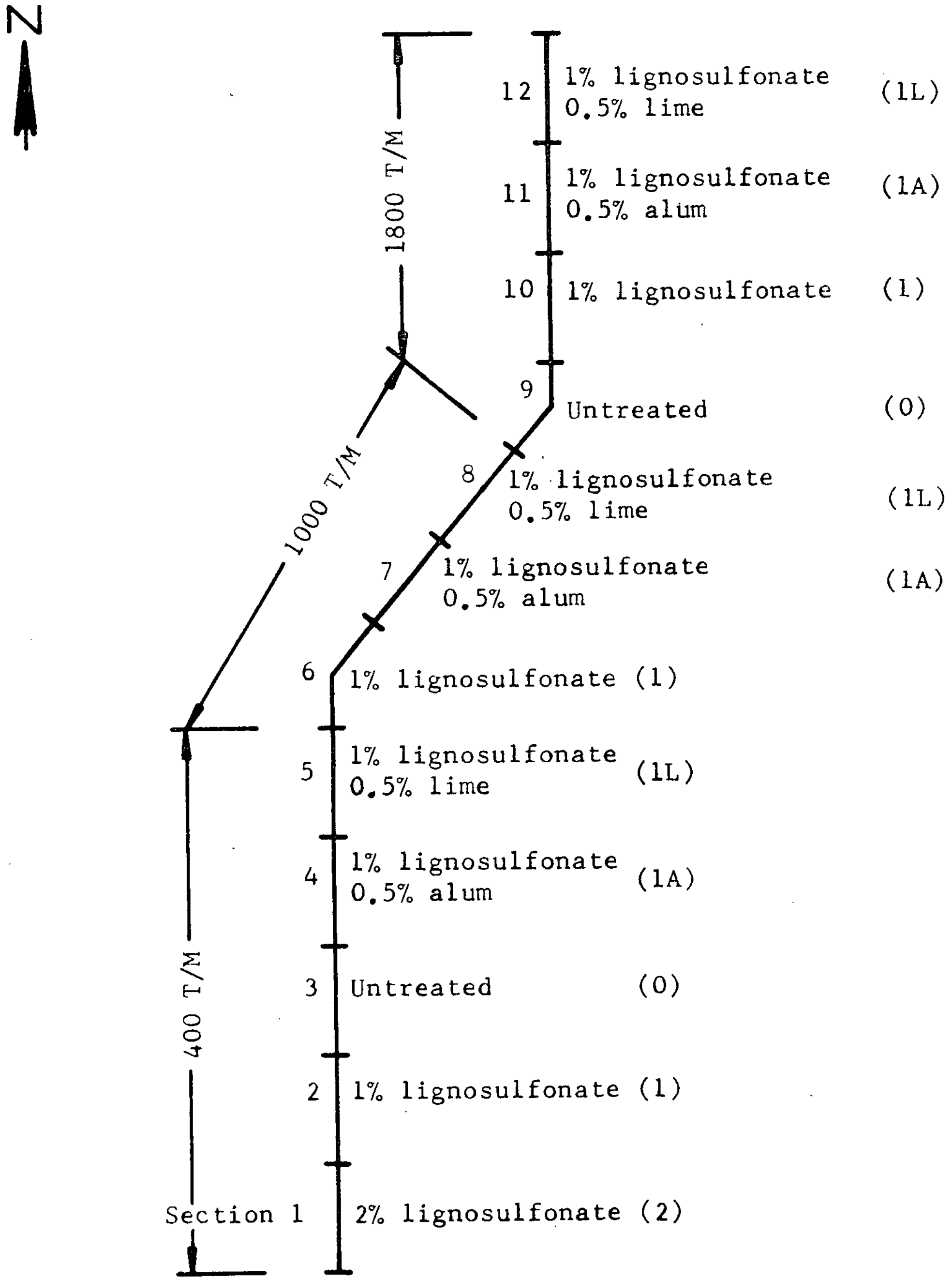


Fig. III-29. Section description, plan view, Marion test road.

Fig. III-30. Vehicle traveling 40 mph on finished grade prior to treatment, 1970, Marion test road.

Fig. III-31. Close-up view, finished grade prior to treatment, 1970, Marion test road.

FIELD TESTING

Several field testing procedures were used to evaluate and analyze performance of test roads in addition to qualitative observations and conversations with persons who used the roads. Tests will be described here and results presented in the discussion of individual test roads.

Testing Procedures

Dusting of roadways was measured by a method similar to standard provisions contained in ASTM designation D 1739. Containers with measured cross section were partially filled with water and placed on stakes at intervals perpendicular to the roadway at relatively constant elevation with roadway centerline. Representative portions of dust raised by passing vehicles settled in containers. After a period of weeks containers were carefully covered and returned to the lab where organic matter was removed, water was evaporated, and accumulated dust was weighed. Pounds of dust per acre per day which settled on areas adjacent to the road could be determined by knowing the weight of the dust, cross section of the container, and exposure time.

To determine amount of loose aggregate on the roadway a strip 3 ft long and the width of the road was swept with a broom and the amount of loose aggregate obtained was weighed. These weights provided an indication of the degree to which aggregate was held in place under the abrasion of traffic and climate. Two measurements were made on each section and an average calculated in tons per mile.

In-place densities were determined using a balloon density apparatus[26]. Holes were carefully dug in the roadway to the bottom of the treated layer and material from the hole double sealed in plastic bags. Volume of hole was measured with the balloon apparatus (Fig. III-32). Sealed material was returned to the lab, weighed, oven-dried, and moisture content and dry density computed. Two or more determinations were made and average density calculated for individual sections.

Fig. III-32. In-place balloon density apparatus.

The Benkelman beam provided a means for measuring deflection of a roadway by an axle load of known weight. An axle weight of 17,300 lb and dual tire pressure of 75 psi were maintained on the test truck. Deflection probe was initially placed 5 ft in front of either the right or left set of rear duals and deflection dial set to zero. As the truck slowly moved forward a reading to the nearest 0.001 in. was taken at maximum deflection as the rear axle passed over the probe, and three more readings were taken at 5-ft intervals (Fig. III-33).

Wheel load divided by maximum deflection determined the relative stiffness of the roadway with dimensions of kips of load per inch deflection[27]. Several determinations were made on each section and an average value calculated. This test was performed on traveled wheelpaths

Fig. III-33. Benkelman beam test.

of the roadway which were usually quite evident. If four wheelpaths were present the two inside paths were the locations for testing. Usually only three wheelpaths existed with one being in the center in which instance the two outside paths were tested. This location procedure was used on all test roads.

Spherical bearing value[28,29] was the result of a stress-strain test in which loads were applied hydraulically to a 6-in.-diameter spherically shaped loading head and deflections recorded at increments of load (Fig. III-34). Data was plotted with total load as the ordinate and a function of deflection and diameter of sphere as the abscissa. To simplify function calculations, hydraulic pressure in the loading piston instead of total load was used for calculations. These two values differed by a constant factor which was equal to the area of the piston inside the hydraulic cylinder. Slope of the line thus formed was defined as the spherical bearing value. Several tests were made

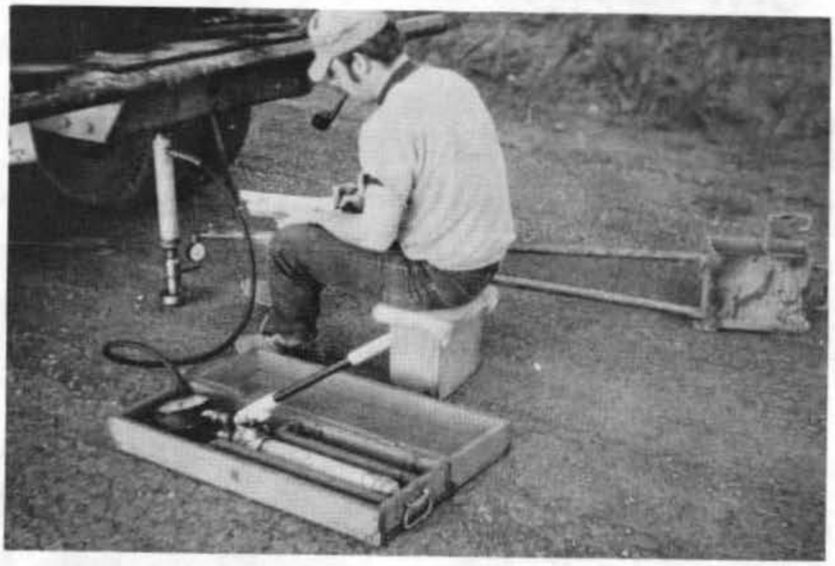


Fig. III-34. Spherical bearing test.

per section and the averages were calculated. Tests were made in the same traveled wheelpaths as with the Benkelman beam.

Temperature recorders were placed in four of the 12 sections of the Marion test road. Four identical recorders were used, each with a 7-day spring driven recording chart and dual individual mercury filled thermal elements. These probes were placed 4 in. from the surface at the bottom of the treated layer, and at 8 in. or 4 in. below the treated-untreated interface in natural subgrade material (Figs. III-35 and III-36). In one section the lower probe was not buried but instead exposed to measured ambient air temperature.

Results

Clinton Test Road

Measurements of dusting were made during the summer of 1971 about 10 months after construction. Dust collectors were set up on both sides

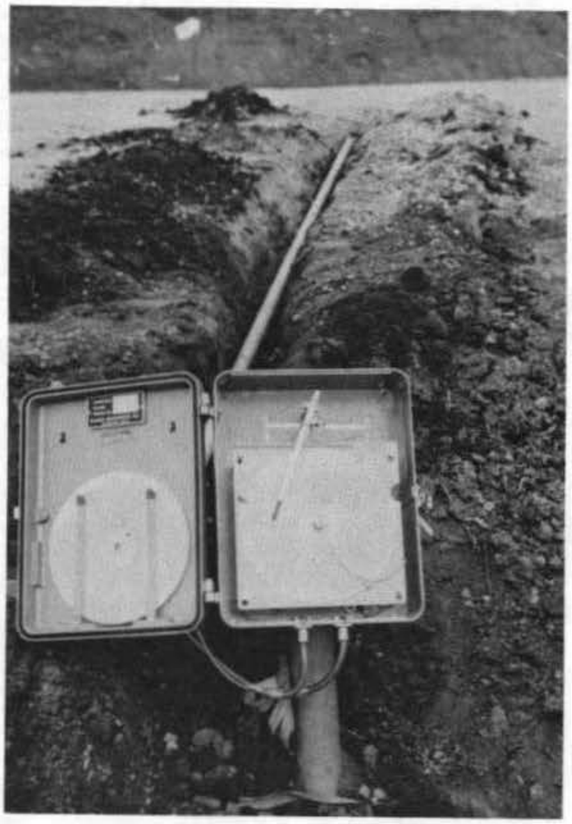


Fig. III-35. 7-day, dual element temperature recorder installation, 1971, Marion test road.

Fig. III-36. Close-up view, dual thermal elements, 1971, Marion test road.

of the road in residual and lignosulfonate plus lime sections, 25 ft and 50 ft from centerline. On the lignosulfonate section collectors were set up on the outside of the curve 50, 75, 100, and 125 ft from centerline. Collectors were picked up 45 days later and only seven were left as the other five had either been knocked down or stolen.

Data from seven remaining collectors is shown in Fig. III-37 although curves could not be drawn to demonstrate dusting for each section. Points indicated general dust levels at various distances from centerline of the three treated sections carrying approximately

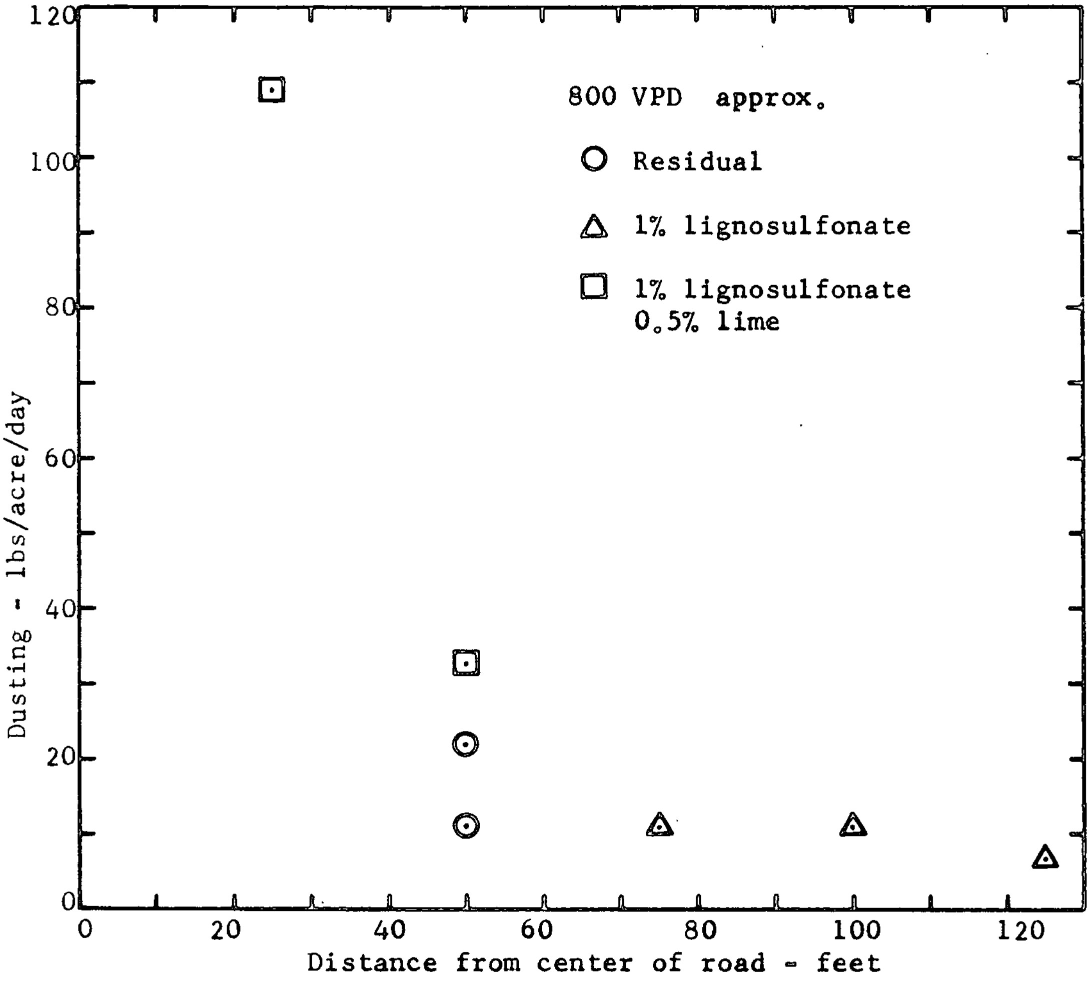


Fig. III-37. Dust measurements, summer 1971, Clinton test road.

800 vehicles per day. Dust from these sections was about the same as was found from untreated roads reported by Bergeson and Hoover elsewhere in this report but which carried only 1/8 as much traffic. All three treatments thus substantially reduced dusting to approximately 15% of original.

A good indication of dusting is the amount of loose aggregate on the surface of a road. Looking at Figs. III-27 and III-28, the difference between treated and untreated material is visible. Figures III-38

Fig. III-38. Close-up view, 1% lignosulfonate plus 0.5% lime additive section, 2 weeks after construction, 1970, Clinton test road.

Fig. III-39. Close-up view, 1% lignosulfonate additive section, 2 weeks after construction, 1970, Clinton test road.

and III-39 show close-up views taken about two weeks after completion of the sections containing lignosulfonate and lignosulfonate plus lime.

It can be seen that there was no loose aggregate present and consequently little or no dusting.

Photographs of the sections taken in spring 1971, are shown in Figs. III-40-43 and indicates that the road had undergone much weathering

Fig. III-40. 1% lignosulfonate plus 0.5% lime additive section, looking south, spring 1971, Clinton test road.

Fig. III-41. Rutting, 1% lignosulfonate plus 0.5% lime additive section, looking north, spring 1971, Clinton test road.

Fig. III-42. Surface runoff channels, 1% lignosulfonate additive section, spring 1971, Clinton test road.

Fig. III-43. Residual section, looking west, spring 1971, Clinton test road.

and traffic abrasion. These sections received no maintenance except for snow removal during winter. The truck visible in Fig. III-40 was not unusual traffic and was being driven close to the shoulder to avoid numerous 1- 2-in. deep potholes. Strength decreased in the proximity of the shoulders due to reduced lateral support in the subgrade. As a result of traffic loads, ruts developed in which rainwater ponded near the shoulder on one side of the road (Fig. III-41).

The hill section treated with lignosulfonate only had better drainage and did not develop potholes, but 10 months after construction did have small drainage channels caused by runoff (Fig. III-42) in this sandy loam soil. Longitudinal cracks developed in the residual section under traffic following construction. Water may have infiltrated into cracks, reducing subgrade support and resulting in limited alligator cracking visible in the months following construction. Potholes appeared in the east portion of the residual section during the spring of 1971 (Fig. III-43) and may have been initiated in wet subgrade spots which resulted from the wet conditions existing in this portion at the time of construction. Rainwater collected in potholes and aggravated the condition similar to the lignosulfonate plus lime section.

In the fall of 1971, more than a year after construction, all sections were covered with a double armor coat. This was followed by very hot weather and peeling occurred as asphalt stuck to tires of heavier vehicles (Fig. III-44). Additional chips were spread which worked into the armor coat and satisfactorily prevented further stripping (Fig. III-45) but was still tacky to the feet during the summer of 1972.

As of late summer 1972, the three sections were in very satisfactory condition. Riding qualities throughout were good, although slight surface ravelling was occurring over approximately 2-5% of the total surface, primarily in the lignosulfonate plus lime section and predominantly in the central portion of the road. Little to no surface ravelling was

Fig. III-44. Peeling of armor coat during hot weather, fall 1971, Clinton test road.

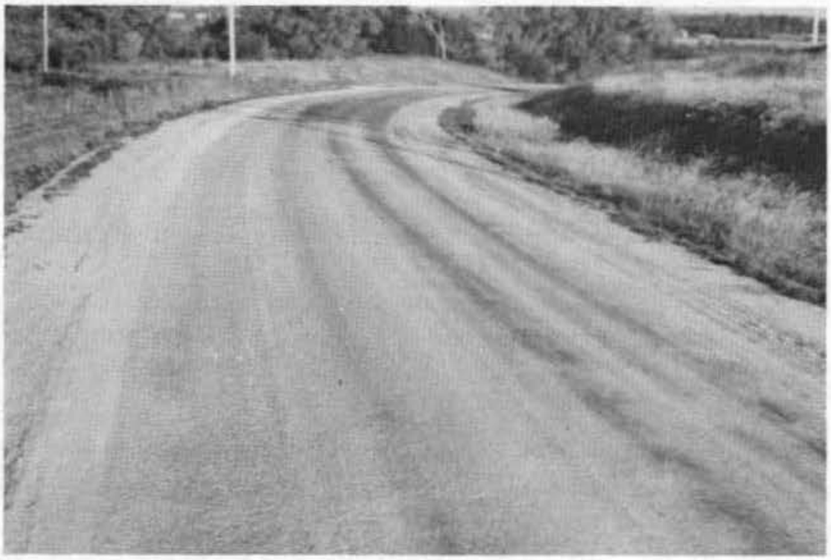


Fig. III-45. Extra chips spread to prevent further stripping, fall 1971, Clinton test road.

observed in the residual section. It appeared that most of the surface ravelling was occurring due to inadequate penetration of the seal into the base. As has been noted previously in this report, lignosulfonates tend to plug soil voids, an action which would allow little or no penetration of a cutback asphalt into the pores. Use of a cationic asphalt emulsion seal may overcome such a problem. Some surface patching was apparent.

It has been observed from the Clinton County project that lignosulfonate, lignosulfonate plus lime, and the residual waste material when used as dust palliatives/surface improvement agents under high traffic volume, do not readily withstand such abrasion. With application of at least a seal coat, performance is quite satisfactory.

Benkelman beam and spherical bearing tests were performed on all sections in the summer of 1970 prior to and about 2 weeks following construction, in 1971 in late spring, during the summer, and several weeks after application of the armor coat and last during the summer of 1972. Figure III-46 summarizes the results of these tests, discussed in the following paragraphs.

Prior to treatment the hill section showed higher strengths by both tests than either of the flat sections which was expected due to better subgrade drainage on the hill. Relative stiffness of the residual section was about 300 kips/in. prior to construction, two weeks following treatment, and during the summer of 1971. The section treated with 1% lignosulfonate decreased in relative stiffness from 500 kips/in. before construction to 350 kips/in. or less. Treatment with 1% lignosulfonate plus 0.5% lime increased relative stiffness from 300 kips/in. to almost 400 kips/in. Spherical bearing values indicated all treatments decreased strength of sections by about 30%.

From fall 1971, to summer 1972, both stiffness and bearing value increased in the residual section while small increases in bearing value

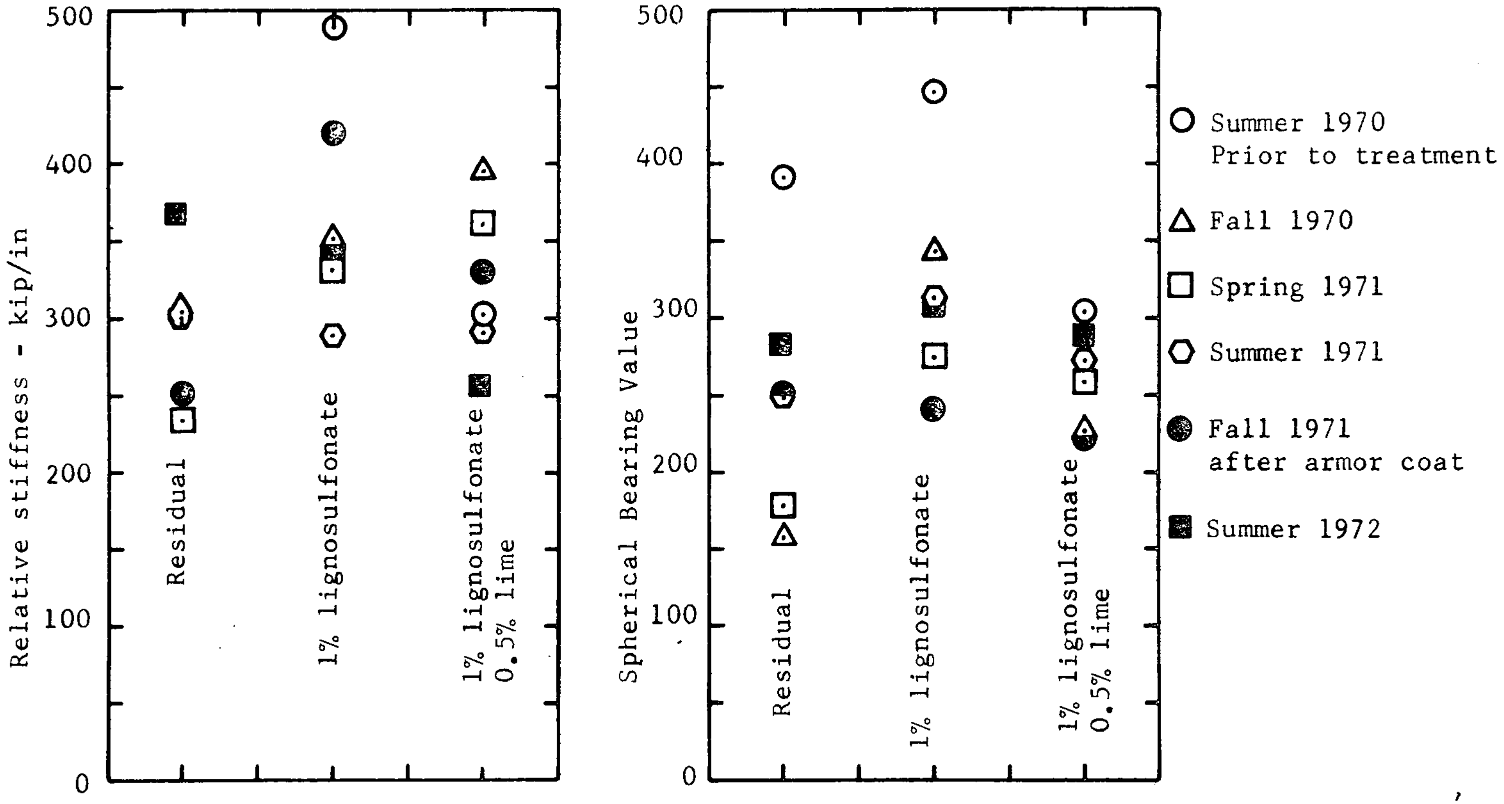


Fig. III-46. Relative stiffness and spherical bearing values, 1970-1972, Clinton test road.

occurred in the lignosulfonate and lignosulfonate plus lime sections. The latter two sections decreased in stiffness values over the same period of time. In general, it is felt that the surface armor coat was beneficial to the overall stability and performance of the three test sections.

Spherical bearing values have been compared to unsoaked CBR and unconfined compressive strengths of soil[28]. For example, an SBV of 100 is approximately equivalent to an unsoaked CBR of 5, while an SBV of 200 is about CBR of 18, and SBV of 300 is equivalent to CBR 32. Unconfined compressive strengths for the same three SBVs noted above are respectively 27, 54 and 80 psi.

SBV values of all test sections after two years were in excess of 250 and at no time were less than 150. Thus, in-place CBR of the three sections over two years ranged from above 10 to in excess of 32.

Unconfined compressive strength of the lignosulfonate sections was in excess of 250 psi after 7 days air curing (Figs. III-5 and III-6) while strength of the residual was 127 psi, indicating that SBVs should have been well in excess of 300 for the treated sections. Thus in-place SBV data is greatly affected by the supporting capacity of the subgrade as evidenced in the results of Fig. III-46.

Relative stiffness values of all test sections were in excess of 225 kips/in. over the two years of study. Maximum stiffness for Navy Department flexible design criteria is 86.4 kips/in. While evidence thus indicates design performance of each section was adequate, this should not be construed that each section was a high performance pavement, since it has been shown that stiffness of such pavements may be in excess of 1000[27]; the minimum Clinton County test road data is thus approximately 1/4 of a high type pavement and far more susceptible to subgrade moisture variations and thickness of treatment.

Floyd County YMCA Road

Dust measurements were not made in Floyd County although visual inspections in 1971, one year after construction, showed reduced dusting on treated roads.

Benkelman beam and spherical bearing tests were performed on the YMCA road before and after treatment. Relative stiffness dropped from 185 kips/in. prior to treatment to 153 kips/in. determined several weeks after construction. Pre-construction spherical bearing values on the YMCA road, and two other lignin treated projects (one northwest, the other southeast of Rockford, Iowa) were 232, 233 and 263 respectively. These and relative stiffness values were much less than those found on the Clinton test road. After treatment the spherical bearing value on the YMCA road was only 127 and also much lower than post construction test values on sections of the Clinton test road. Average dry density on the YMCA road several weeks after construction was 112 pcf.

Each of the three observed Floyd County lignosulfonate projects have performed less satisfactorily than the Clinton County test sections in that aggregate pullout, potholing, and dusting after one year were visually much higher than the Clinton County sections (although at least 50% less than prior to treatment). A large contributing factor to the performance in Floyd County was the initial silty to sandy characteristics of the in-place soils which with the quantity of lignosulfonate utilized did not appear to provide a tightly bonded surface.

In early October 1971, each of these roads were seal coated, very unsatisfactorily, due primarily to a faulty control valve system in a new asphalt distributor. As of late summer 1972, it was visually apparent that an estimated 15 to 20% of the seal had already been patched and much more was in the process of ravelling and potholing.

Though the above problems are noted herein, it was still obvious that after two years of performance each Floyd County lignosulfonate project was still providing an all-weather serviceability.

Marion Test Road

Density and strength tests were made several weeks after construction in 1970 and at intervals during 1971. Delays in shipping caused temperature recorders to arrive too late for installation before winter 1970. Temperature recorders were placed in the spring of 1971 in sections 6, 7, 8 and 9. Dust and loose roadway aggregate measurements were made during a dry spell in late summer (Figs. III-47 and III-48).

Observations during the year after construction showed that the performance of section 3, the low aggregate untreated section, was the worst of the 12 sections. A soft area with ruts developed during the spring thaw in 1971 (Fig. III-49). A month later the ruts had begun to heal somewhat, Fig. III-50, but there was no maintenance except for usual snow removal in winter. A very light, sparse, spread of limestone chips was placed on sections 4 and 5 in spring 1971, due to extremely slippery conditions.

In the fall of 1971, a year after construction, there were several days during which a light, misty rain fell continuously. The soft spot in section 3 again appeared (Fig. III-51). All sections containing

Fig. III-47. Dust collectors, summer 1971, Marion test road.

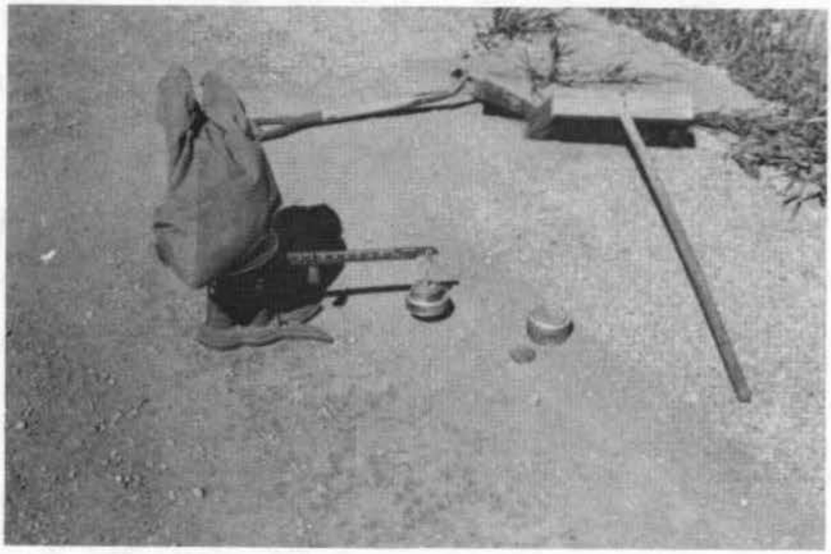


Fig. III-48. Loose roadway aggregate determination, summer 1971, Marion test road.

low and medium amounts of aggregate had about 1/4 in. of lignosulfonate dispersed clayey mud on the surface (Fig. III-52. Section 1, containing 2% lignosulfonate, had a much lighter mud layer (Fig. III-53). Sections containing the most aggregate were almost unaffected by the rain and not

Fig. III-49. Untreated, low aggregate content, section 3, March 1971, Marion test road.

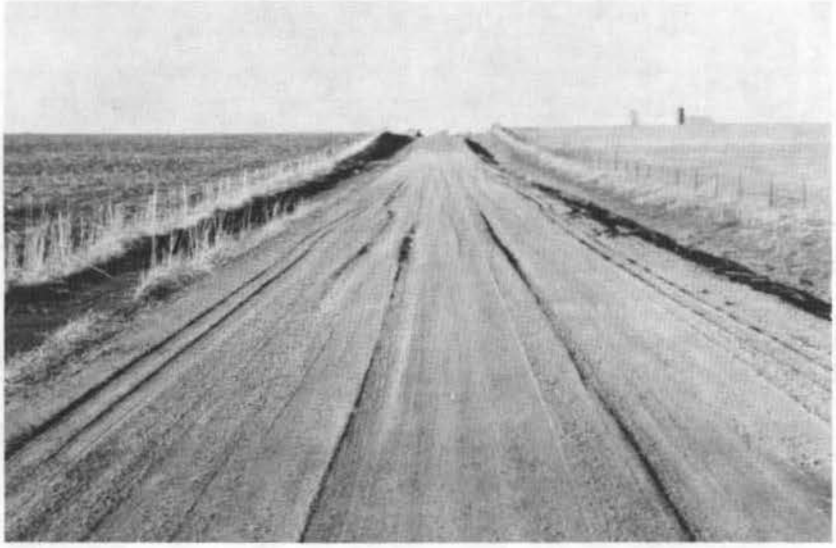


Fig. III-50. Untreated, low aggregate content, section 3, April 1971, Marion test road.

the least bit muddy. Several days later in low and medium aggregate sections, ponded water visibly contained dissolved lignosulfonate (Fig. III-54).

Fig. III-51. Untreated, low aggregate content, section 3, October 1971, Marion test road.

Fig. III-52. Lignosulfonate dispersed surface after drizzle on low and intermediate aggregate content sections, October 1971, Marion test road.

At the time loose aggregate was measured a series of pictures were taken for comparisons among the sections. Figures III-55-66 were representative observations of typical dry conditions of several sections.

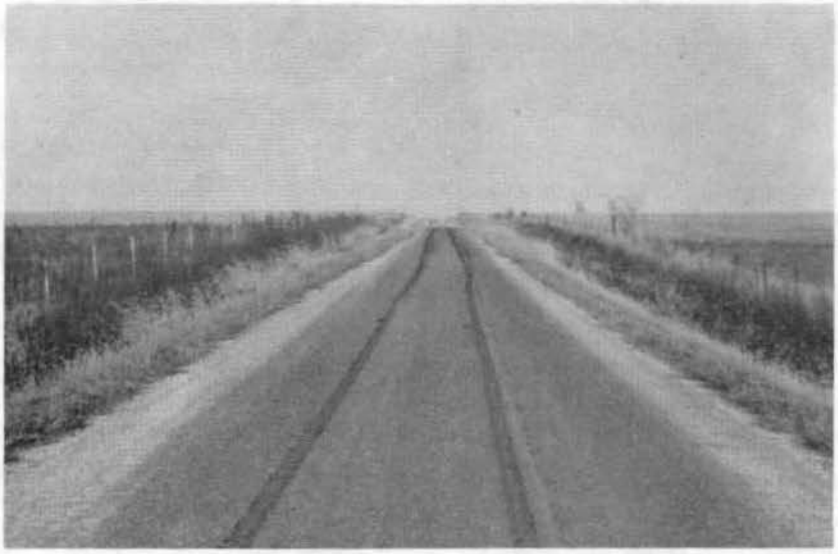


Fig. III-53. Lignosulfonate dispersed surface after drizzle on 2% lignosulfonate, low aggregate content, section 1, October 1971, Marion test road.

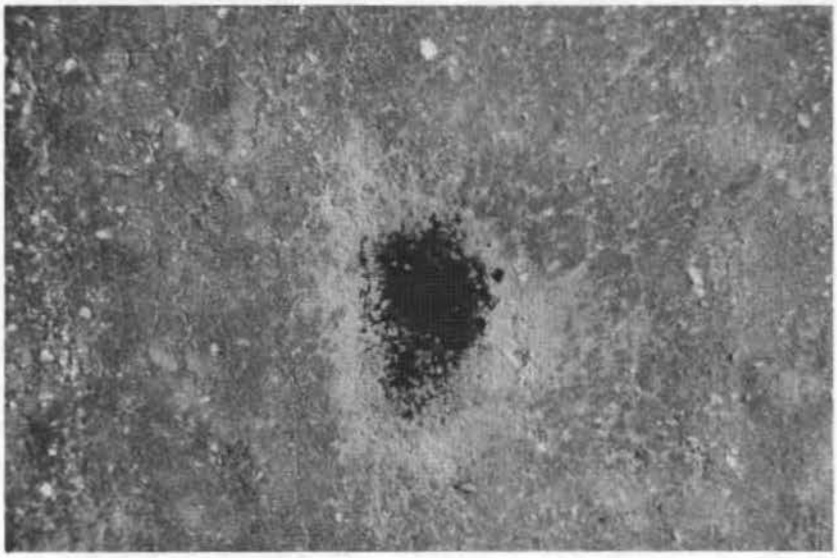


Fig. III-54. Roadway depression containing ponded rain water with leached lignosulfonate, October 1971, Marion test road.

Fig. III-55. Broomed strip across section 1, 2% lignosulfonate, low aggregate content, summer 1971, Marion test road.

Fig. III-56. Close-up view, section 1, 2% lignosulfonate, low aggregate content, summer 1971, Marion test road.

Each pair of pictures showed a broom-swept path across the section for visual indication of loose aggregate present and a close-up view of the normal traveled portion of the road. The six sections were the 2% and 1% lignosulfonate-400 tons per mile (sections 1 and 2), 1% lignosulfonate-1000 tons per mile (section 6), 1% lignosulfonate-1800 tons per mile (section 10), and the two untreated sections at 400 and 1800 tons per mile (sections 3 and 9). Sections not shown contained alum or lime and visually were very similar to the 1% lignosulfonate section at the same aggregate content.

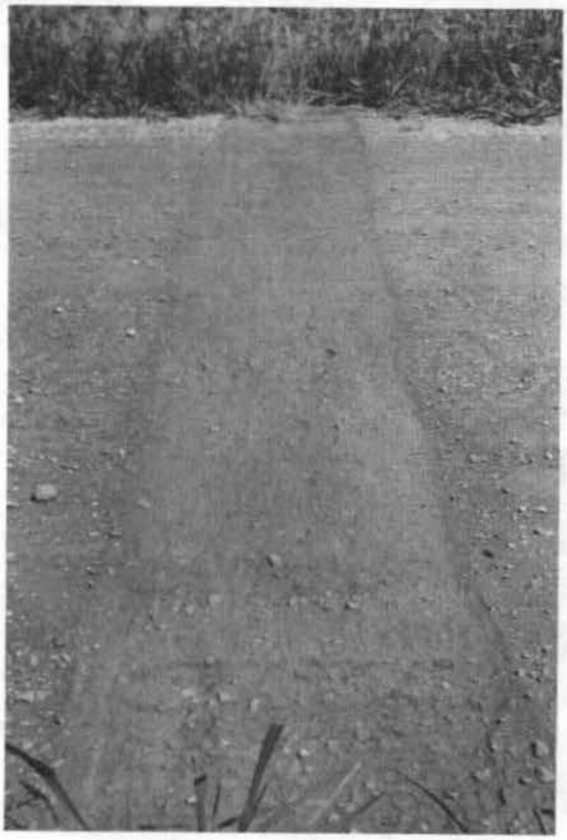


Fig. III-57. Broomed strip across section 2, 1% lignosulfonate, low aggregate content, summer 1971, Marion test road.

Fig. III-58. Close-up view, section 2, 1% lignosulfonate, low aggregate content, summer 1971, Marion test road.

The difference between sections 1 and 2 appeared to be more loose aggregate near the shoulders on section 1, with a surface that had a greater amount of fine alligator cracks (Figs. III-55 and III-56). In comparing 1% lignosulfonate, sections 2, 6, and 10, increasing aggregate content was seen (Figs. III-57-62). Increasing aggregate content increased amount of loose aggregate near the shoulders, but not in the traveled wheelpaths. On untreated sections there were larger amounts of loose aggregate present across full roadway widths

Fig. III-59. Broomed strip across section 6, 1% lignosulfonate, intermediate aggregate content, summer 1971, Marion test road.

Fig. III-60. Close-up view, section 6, 1% lignosulfonate, intermediate aggregate content, summer 1971, Marion test road.

(Figs. III-63-66) and particularly more fine dust producing particles to be raised by passing vehicles than on treated sections.

Dust measurements were taken on seven sections during the summer 1971. Collectors were placed on both sides of the road 12, 23, and 30 ft from centerline except for one side of section 8 where distances were 12, 28, 44, and 60 ft. Individual curves were drawn for both sides of the road for each section and the average curve was then drawn for each section (Fig. III-67).

Fig. III-61. Broomed strip across section 10, 1% lignosulfonate, high aggregate content, summer 1971, Marion test road.

Fig. III-62. Close-up view, section 10, 1% lignosulfonate, high aggregate content, summer 1971, Marion test road.

There appeared to be three groupings of sections, which produced high, medium, and low amounts of dust. The high group consisted of only one section, untreated section 3. For the sake of calculations the amount of dust 20 ft from the centerline of section 3 was assumed to be 100%. By comparison the medium group of the 400 and 1000 tons per mile sections had average dusting of 59% of the untreated section. The low dusting group of sections at high aggregate content produced only 18% as much dust as section 3. This indicated that treatment of high aggregate content sections reduced dusting by about 80%.

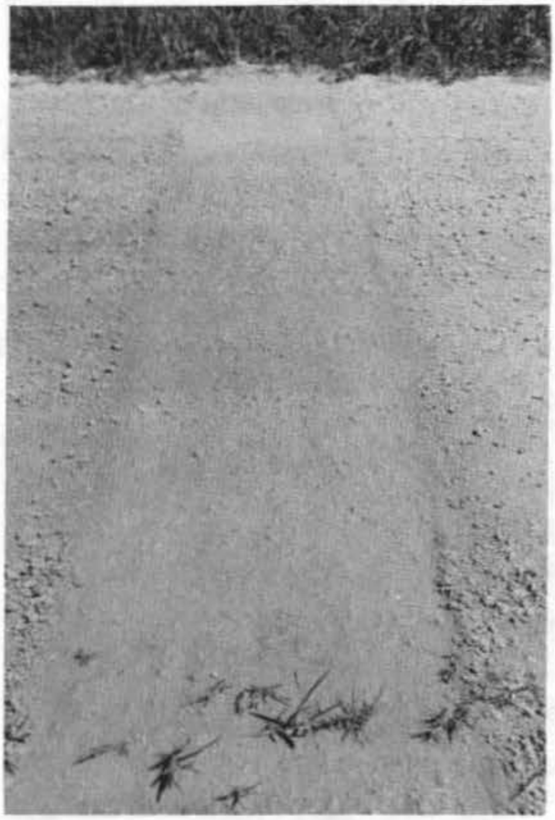


Fig. III-63. Broomed strip across section 3, untreated, low aggregate content, summer 1971, Marion test road.

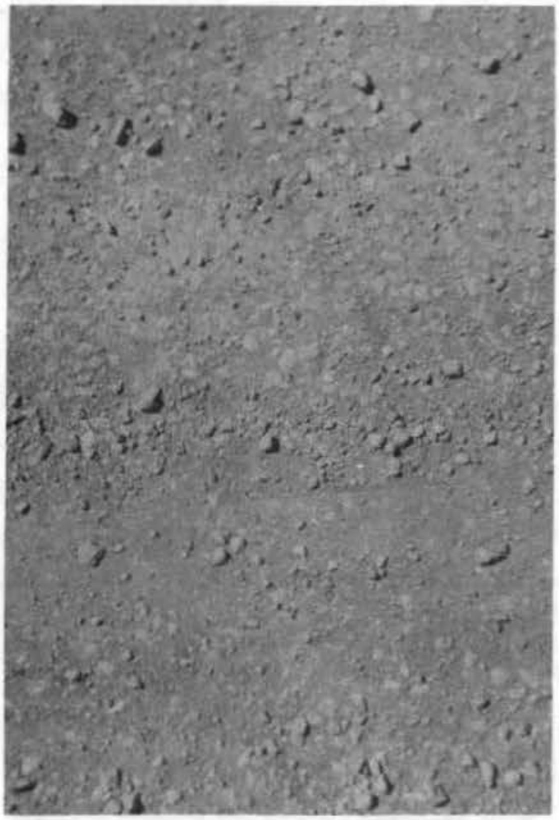


Fig. III-64. Close-up view, section 3, untreated, low aggregate content, summer 1971, Marion test road.

Loose material on a roadway reduces a driver's visibility and control of his vehicle. An indication of the amount of dust and rocks which could be raised by a passing vehicle was found by measurements of loose aggregate.

Measurements of quantity of loose material on each section of the Marion test road were made during the summer 1971, and amounts presented as tons of loose aggregate per mile of length (Fig. III-68). In general, quantities increased with increasing aggregate content which was the reverse of the dusting trend. A reasonable explanation for

Fig. III-65. Broomed strip across section 9, untreated, high aggregate content, summer 1971, Marion test road.

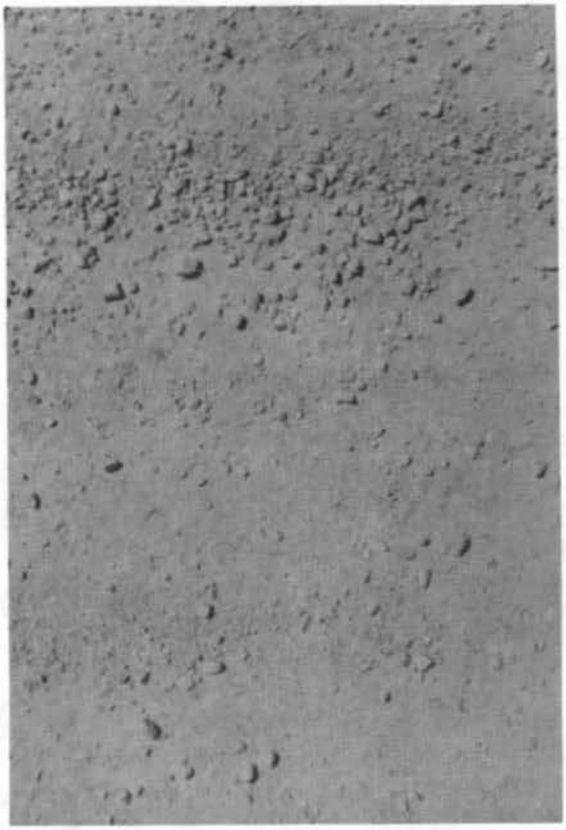


Fig. III-66. Close-up view, section 9, untreated, high aggregate content, summer 1971, Marion test road.

these reverse trends was that most loose aggregate was located near the edge of the road, instead of in the traveled portion which was where dust originated. This observation was visible in photographs which showed the road in sufficient detail (Figs. III-55-66).

Surface runoff had carried aggregate off of the roadway of section 10 and limestone rocks were present in large numbers in small erosion channels running down the foreslope into the ditch. This was not seen on any of the other sections. Therefore the quantity of loose aggregate on section 10 should have been greater than was actually weighed.

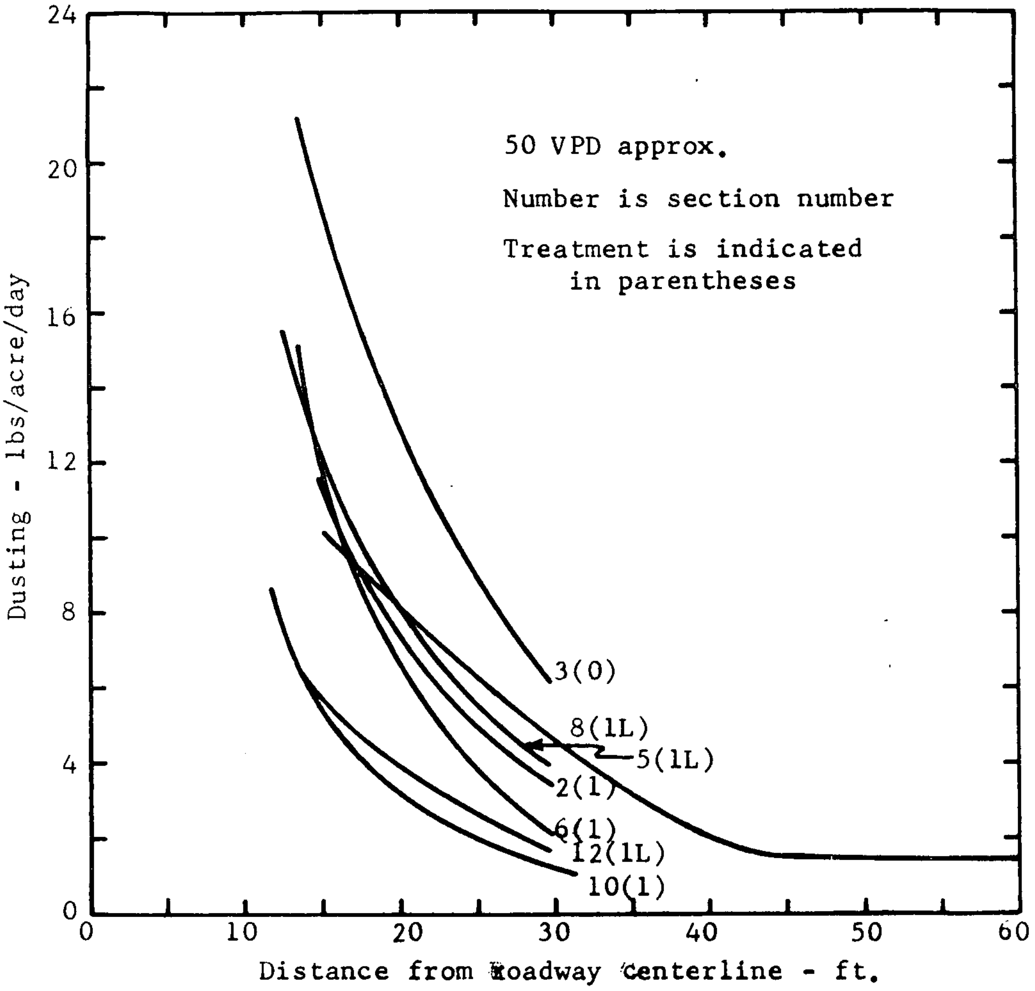


Fig. III-67. Dust measurements, summer 1971, Marion test road.

As a result of this adjustment a consistent observation can be made within each of the three aggregate contents, 1% lignosulfonate with 0.5% lime as a secondary additive had the least amounts of loose aggregate. Sections with 1% lignosulfonate and 1% lignosulfonate plus 0.5% alum had higher quantities of loose aggregate with no obvious difference between these two treatments. The section with 2% lignosulfonate had less loose aggregate than all 1% lignosulfonate sections and 1% lignosulfonate with alum sections. Untreated sections had the most loose aggregate.

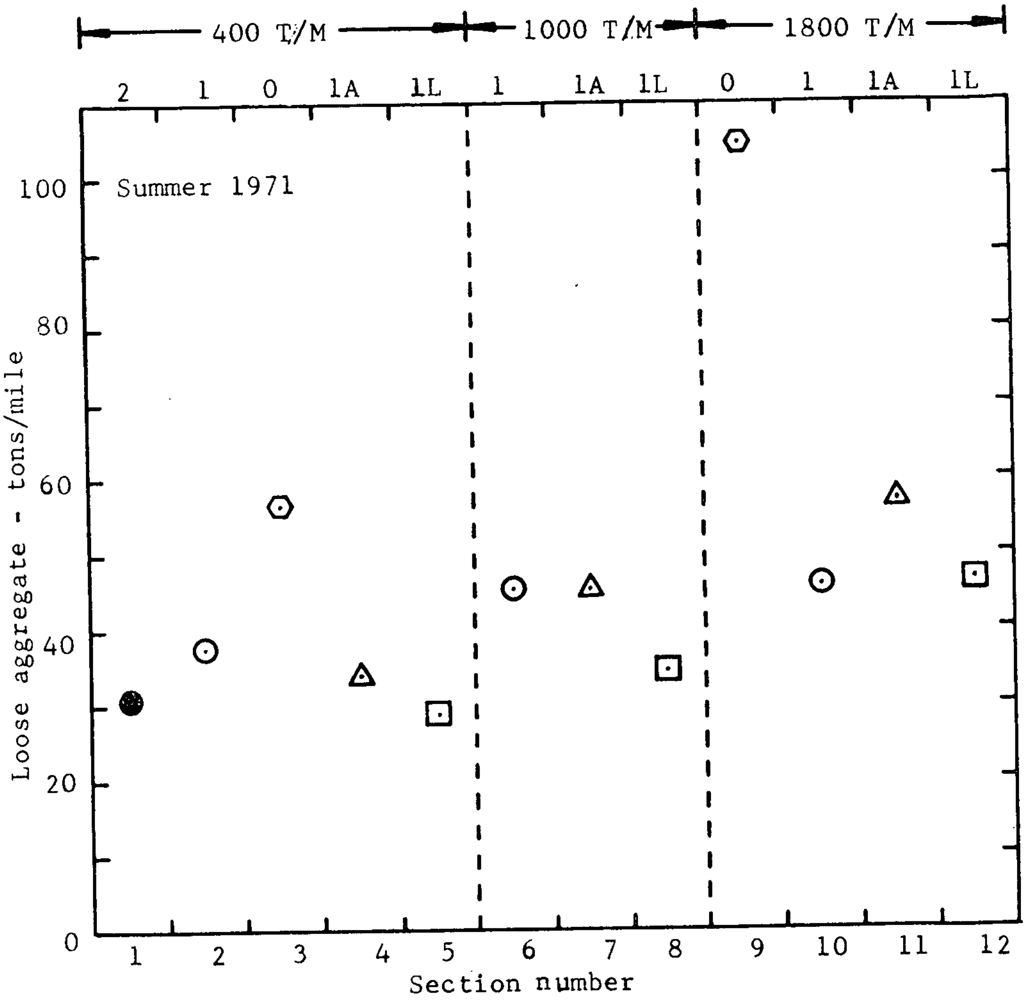


Fig. III-68. Loose roadway aggregate measurements, summer 1971, Marion test road.

In-place density tests and moisture content determinations were made in the fall 1970, shortly after construction, and the following summer and fall. Values found were very scattered (Figs. III-69 and III-70) but several observations could be made. Highest dry densities and lowest moisture contents occurred in the untreated section at high aggregate content. The low aggregate untreated section usually had lowest moisture content of the 400 and 1000 tons per mile sections. Within the low and intermediate aggregate content sections, density

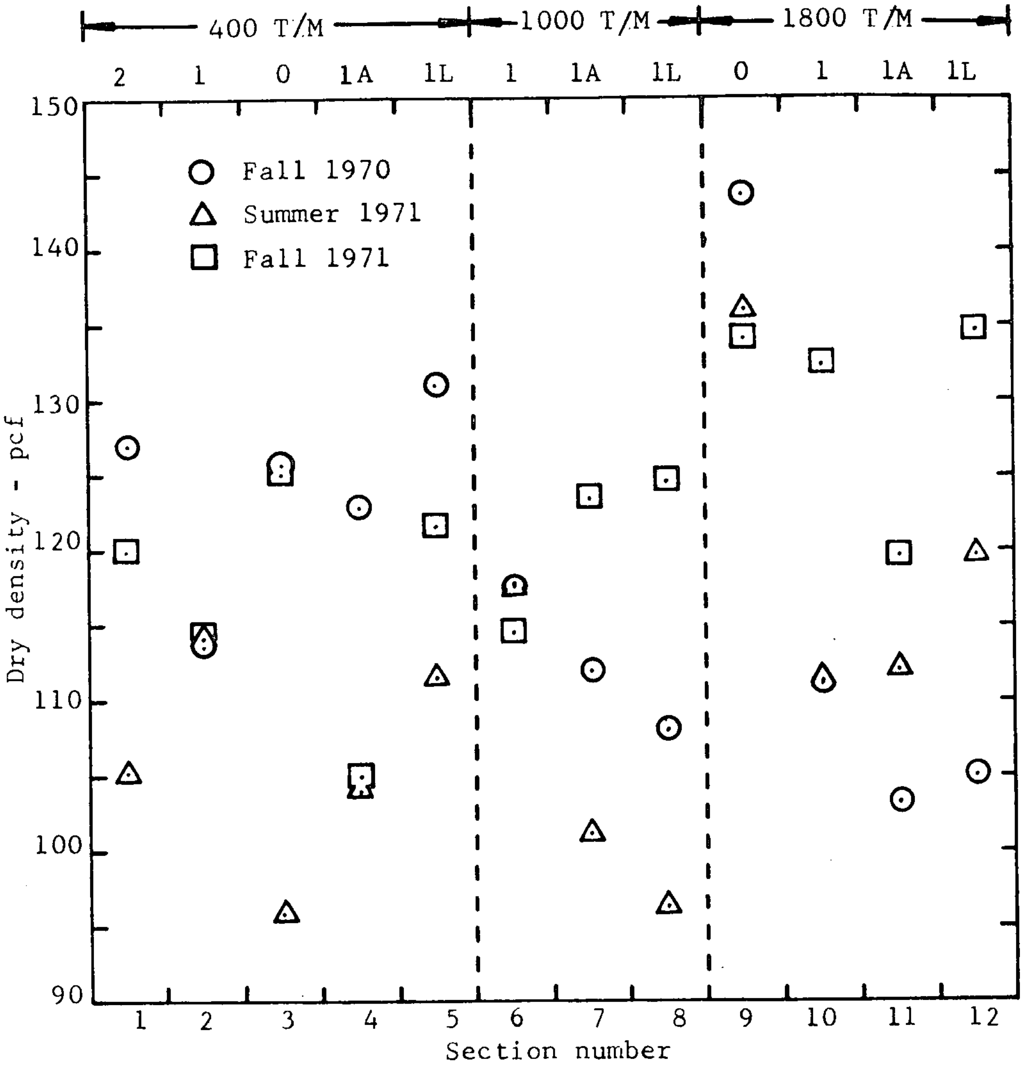


Fig. III-69. Dry density determinations, 1970-1971, Marion test road.

of the untreated section varied from highest to lowest depending on the season.

Moisture contents shortly after construction were the highest which occurred. In high aggregate sections, low moisture contents indicated that those sections dried out more rapidly and maintained a lower moisture level. In all sections lowest moisture content was found in the summer 1971.

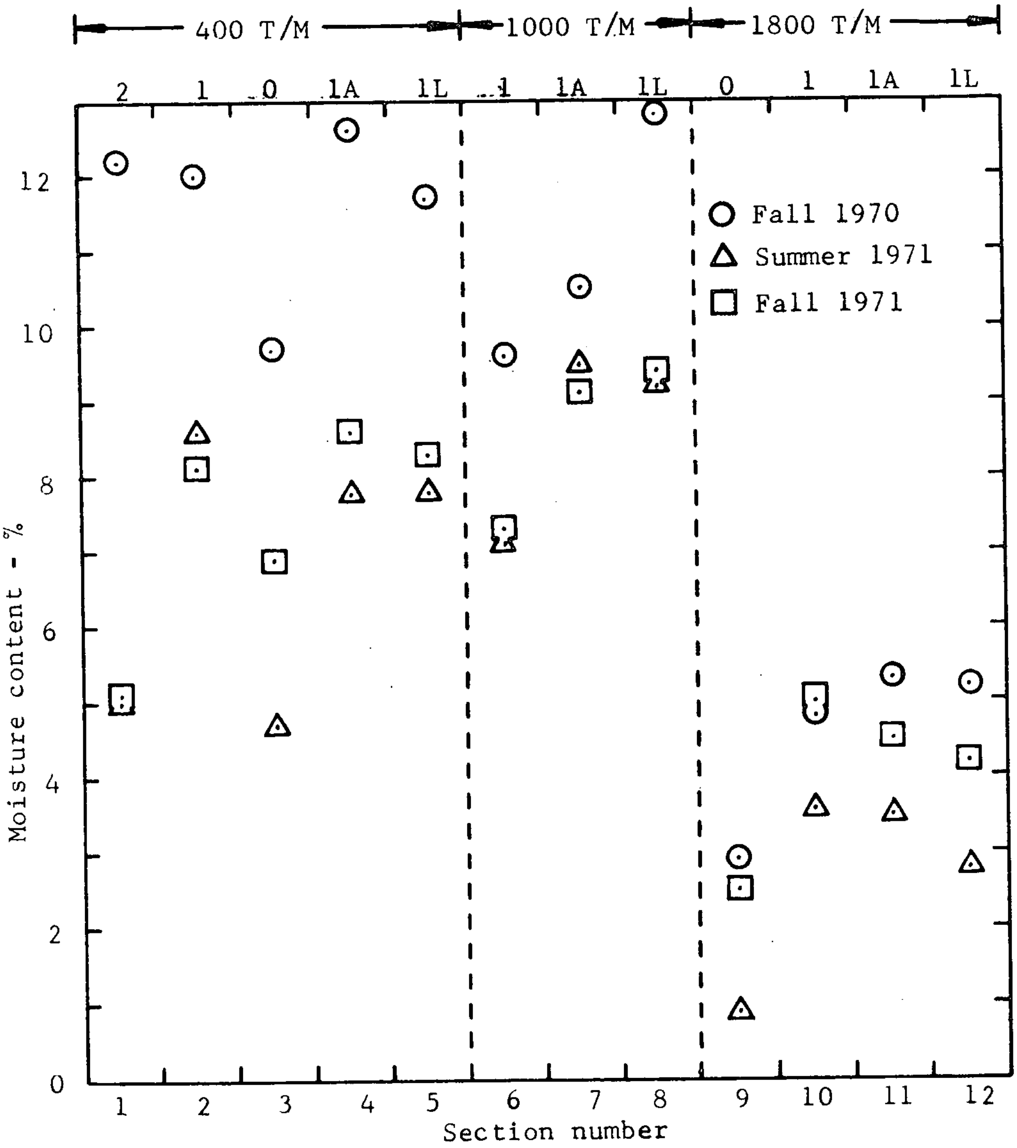


Fig. III-70. Moisture content determinations, 1970-1971, Marion test road.

An observation could be made concerning aggregate contents by comparisons of seasonal variations of dry densities of treated sections. Densities found soon after construction in the 400 tons per mile treated sections were highest of the seasonal determinations made. Densities right after construction were intermediate values for the 1000 tons per mile sections and lowest values in the treated sections with 1800 tons of aggregate per mile. Another way to state this was to observe that during the year following construction, dry densities decreased in low aggregate content sections, remained about the same in intermediate aggregate content sections, and increased in treated sections with highest aggregate content. This might be interpreted that treated sections were all approaching a common density. Averages of the three seasonal densities were all between 109 and 122 pcf except for untreated section 9 which had an average seasonal density of 138 pcf (Fig. III-71).

Densities of high aggregate treated sections increased during the months following construction. Untreated high aggregate content section densities showed a slight decreasing tendency. Perhaps all of the high aggregate sections were approaching a common density which would be greater than that of any of the sections containing less aggregate.

Strength tests showed seasonal variations for sections, and both relative stiffness and spherical bearing values were lowest in spring tests, 1971 (Figs. III-72 and III-73). Strength values increased in the summer and were higher still in the fall 1971. This was an annual cycle to be expected and had been observed for many years for roads in general since designers anticipate lowest stability during the wettest season, which is the spring thaw, and an increase in stability as subgrade materials drain reaching a peak stability in late fall. However, summer 1972, tests saw a definite increase in SBV and stiffness values of all sections.

Trends observed with relative stiffness values, were also found with spherical bearing values. Duplication of results by two different

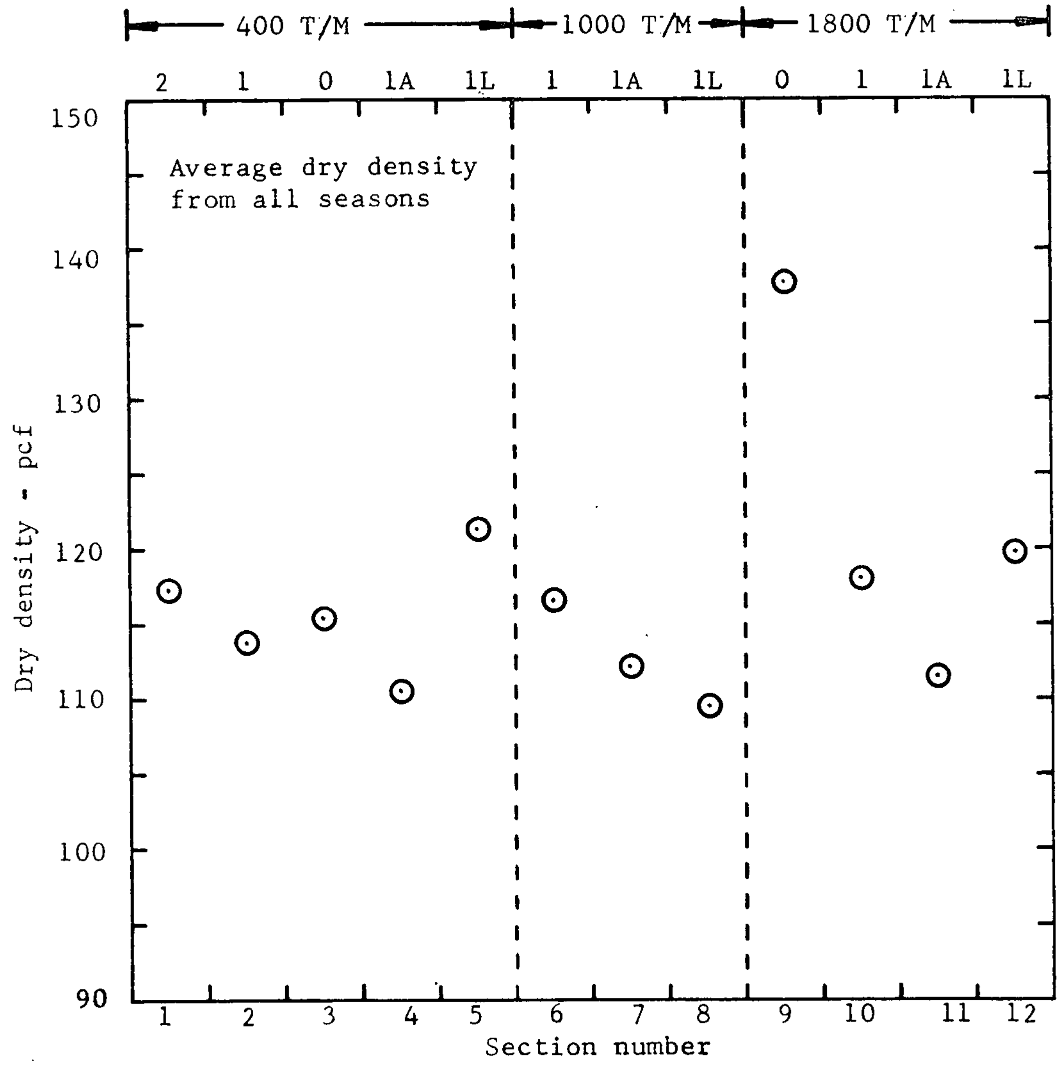


Fig. III-71. Average dry density, 1970-1971, Marion test road.

tests added credibility to the data and made evaluations based on such data more significant.

One such observation concerned density-strength relationships shown by the three lignosulfonate plus lime sections 5, 8, and 12. Spherical bearing and Benkelman beam tests made one year apart in the fall on these sections increased in strength except for low aggregate section 5 which decreased very slightly in relative stiffness. Section 5 decreased in density and the other two increased in density.

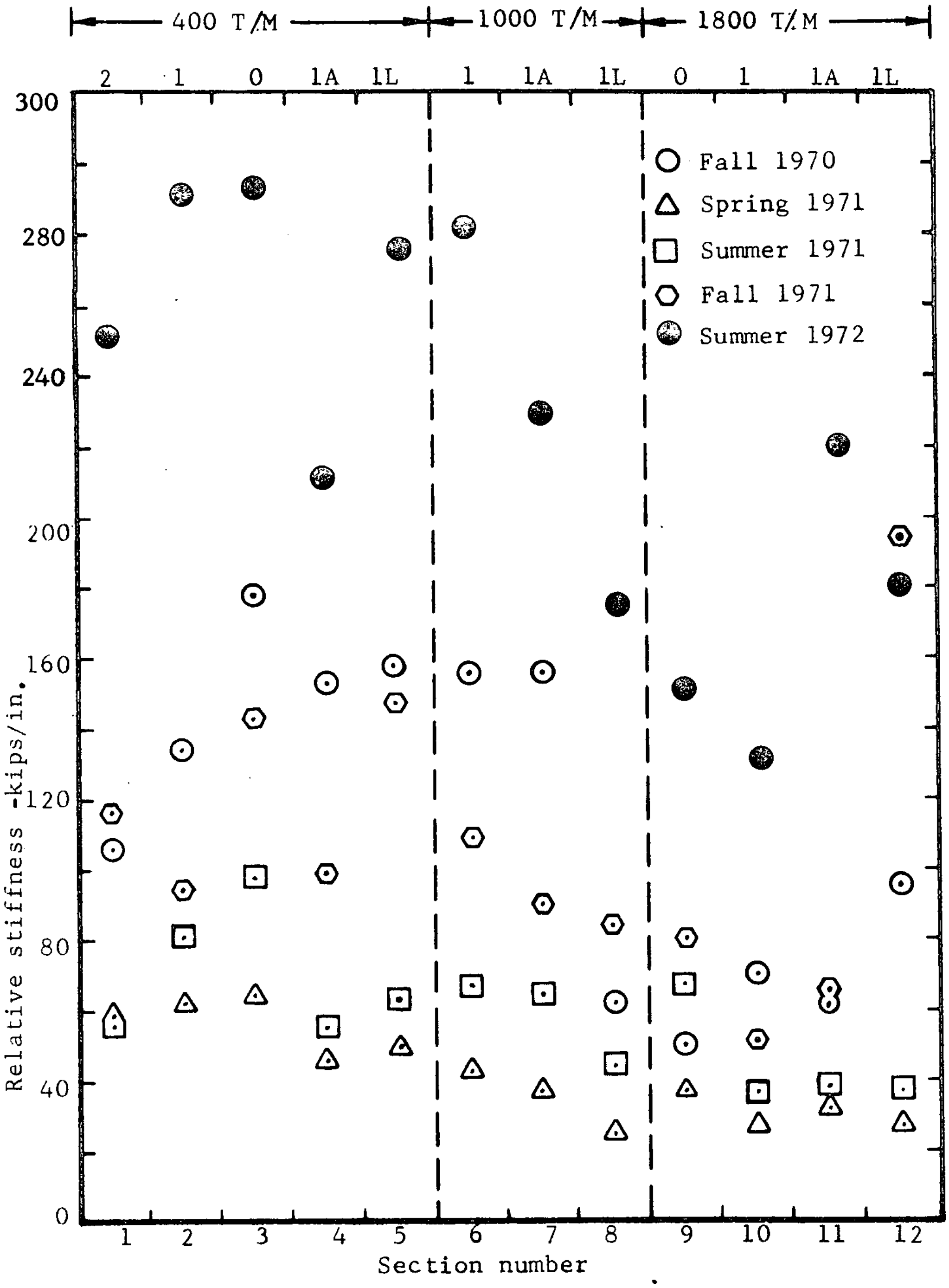


Fig. III-72. Relative stiffness values, 1970-1972, Marion test road.

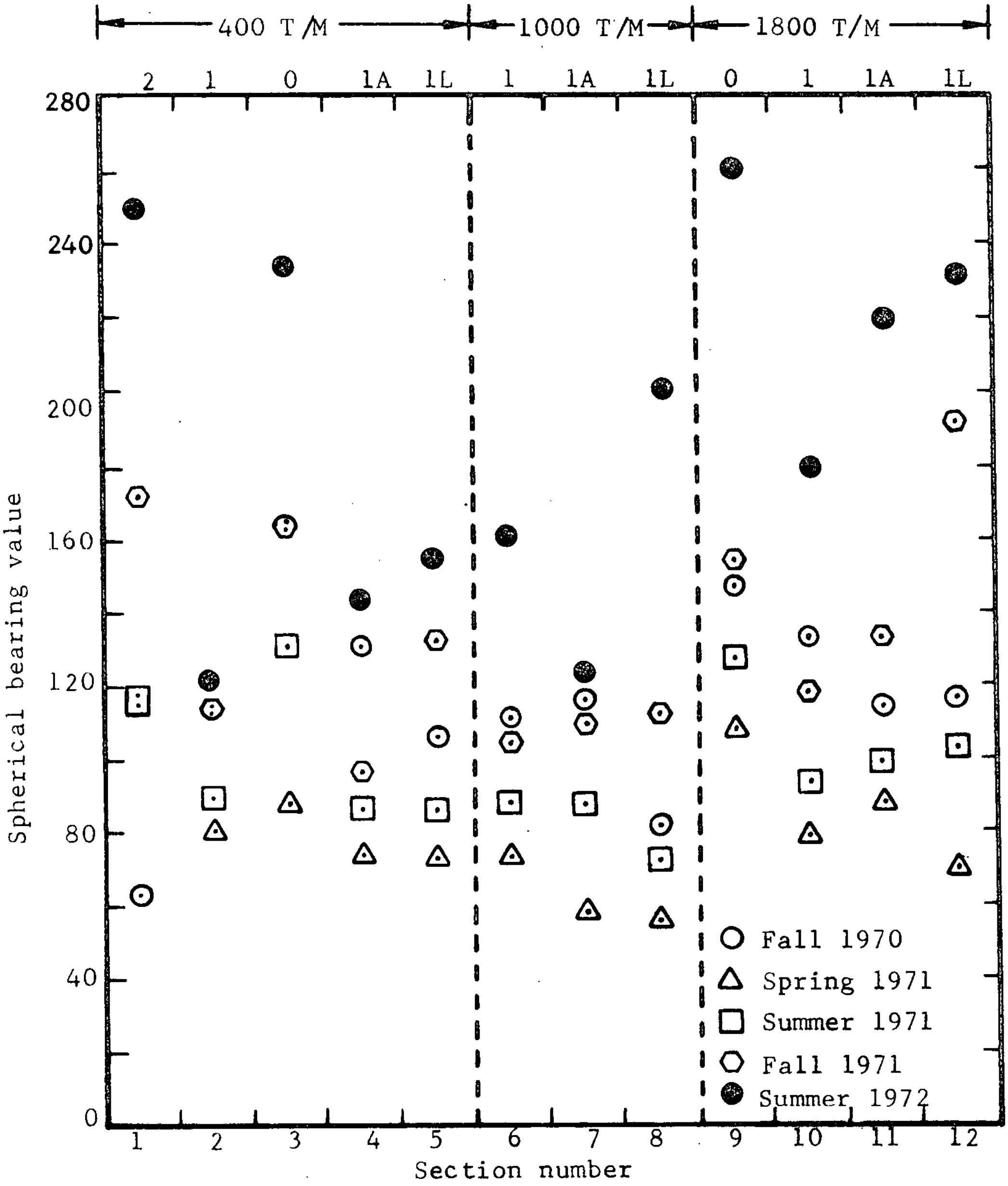


Fig. III-73. Spherical bearing values 1970-1972, Marion test road.

Tests made in the fall 1971, one year after construction, showed that lignosulfonate plus lime sections of low and high aggregate contents generally had higher densities and relative stiffness values than other treated sections at the same aggregate content; a reversal was noted with the intermediate aggregate section. At the same time the highest spherical bearing value was also by the high aggregate lignosulfonate plus lime section.

Fall tests in 1971 showed that the 2% lignosulfonate section was stronger and more dense than the adjacent 1% section. The 2% section did not have superior strength nor was it as dense as the 400 tons per mile untreated section. Therefore strength and density of the 1% lignosulfonate section were less than the untreated section, a relationship which was also shown in the 1800 tons per mile sections. Similar trends of these sections was also noted for the summer 1972 tests.

There did not appear to be any significant relationships evident between strength and aggregate content. Though all sections improved in both SBV and relative stiffness values with time, the summer 1972 tests indicated (a) generally lowest SBV results in the 1000 tons per mile sections, while (b) generally lowest relative stiffness values in the 1800 tons/mile sections. Perhaps if densities became fairly constant future strength tests would indicate a trend. Future tests might also include strength tests immediately after a soaking rain to determine if additives improved strength retention in the presence of high moisture content. Tests in the spring 1971, and summer 1972, were made when the roadway surface at least was relatively dry.

Comparison between identical treatments versus aggregate content is also somewhat anomalous for the summer 1972 tests. SBV values of the 1% lignosulfonate sections show a general increase with increasing aggregate content while the reverse is noted in relative stiffness. SBVs of the lignosulfonate plus alum sections were lowest with the 1000 tons per mile section, higher at 400 tons per mile, and highest at 1800 tons per mile, while a near reversal was noted in relative stiffness. SBVs of the lignosulfonate and lime sections increased with increasing aggregate while a near reverse was noted in relative stiffness.

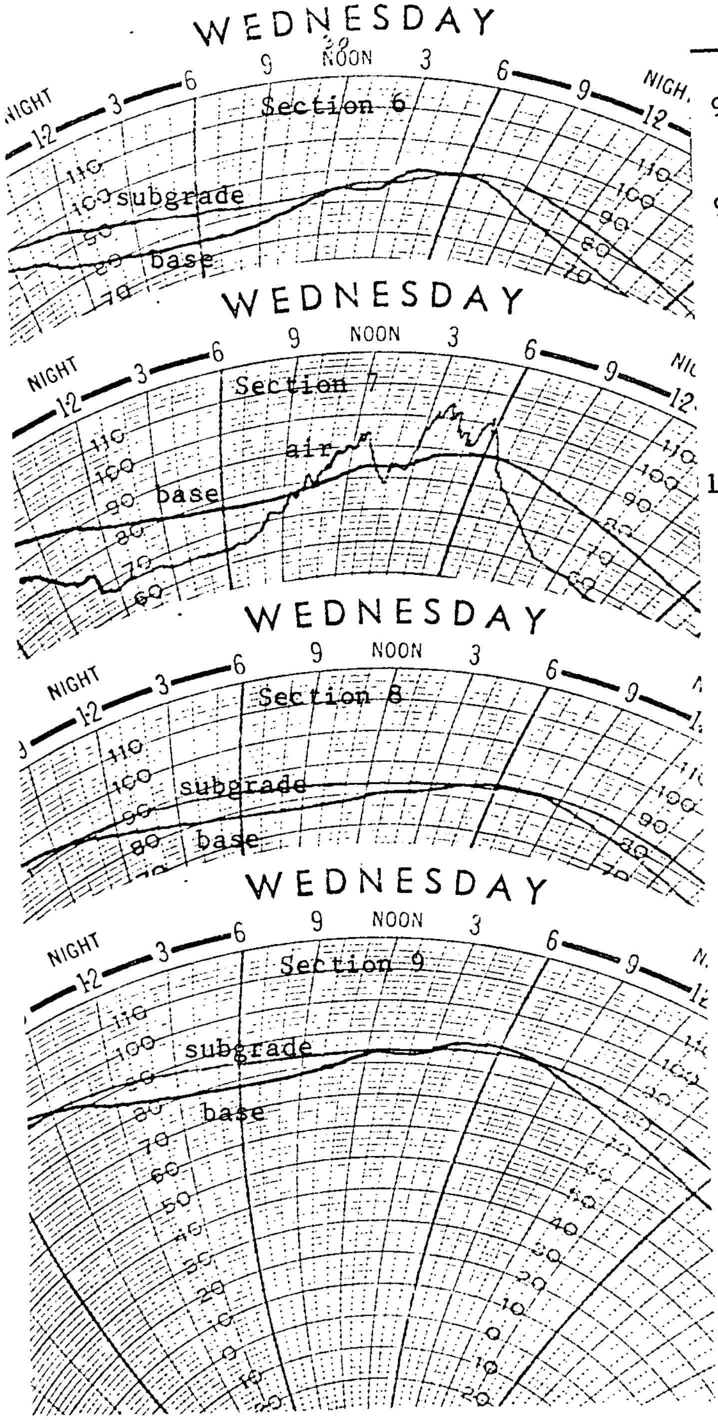
The greatest percentage of strength gain over post-construction tests during the two-year study period was in the 400 tons per mile aggregate content section containing 2% lignosulfonate. As previously noted strengths increased with increasing additive percentages in the laboratory tests. Studies by Makoto[24] indicated that adsorption of a lignosulfonate to a montmorillonite clay was dependent on pH, with lower pH values creating better adsorption. Adsorption may be indicative of stabilization. Greater quantities of limestone may have increased the soil pH. Thus a larger soil concentration of lignosulfonate in a lower pH regime may improve adsorption/stabilization characteristics and may, with considerable time, polymerize. Much further research is needed in this lignosulfonate stabilization area. However montmorillonitic clays may hold a key to the mechanism.

Temperature of base and subgrade of sections 6-9 were continuously recorded on charts since installation in the spring 1971. As previously described these sections are the three intermediate aggregate content sections and the untreated section with high aggregate content. Air temperature instead of subgrade temperature was measured by one of the thermal elements in section 7.

Observations were made on summer temperatures since similar thermal conductivity characteristics should have occurred whether heat flow was upward as in winter or downward as in summer. Examination of temperatures for a day in June (Fig. III-74) showed that during the afternoon air temperature decreased and then increased. Temperature in base and subgrade changed accordingly with change most noticeable in section 6 and least noticeable in section 8. Section 6 had the highest and lowest base temperature of the day. Section 8 had the lowest high base temperature and the highest low base temperature. Therefore section 6 was most susceptible to temperature change and section 8 was least susceptible.

Temperature change in section 8 was slower than in section 6. This was shown by section 8 reaching its maximum or minimum temperature two hours after section 6 respectively reached its maximum or minimum temperature (Fig. III-74). High temperature in the base of section 8 was 5 °F less than high temperature in the base of section 6. Minimum temperature of section 8 was 2 °F higher than the low base temperature of section 6, which showed that temperature varied most in section 6, the 1% lignosulfonate section, and least in section 8, the 1% lignosulfonate plus 0.5% lime section. The other two sections, untreated and with lignosulfonate plus alum had less temperature variations than section 6 but more than section 8.

A small temperature variation when external temperature varies greatly is one of the properties of an insulating material. If the better insulating properties of section 8 were to be attributed to lower density a look at Fig. III-69 showed that section 8 did not always have a lower density than section 6. However section 9 had a



	HIGH		LOW	
	°F	Time p.m.	°F	Time a.m.
Base	91	4:00	75	7:00
Subgrade	92	6:30	82	8:00
Base	90	6:30	76	8:00
Air	105	3:00	64	5:00
Base	86	6:00	77	9:00
Subgrade	86	6:00	77	8:00
Base	89	5:30	77	7:30
Subgrade	87	6:30	83	9:00

Note: Subtract one-half hour from the chart hours for subgrade and air temperature to obtain real clock time

Fig. III-74. Temperature recording chart, June 1971, Marion test road.

much higher density than any of the others but did not have as great a temperature variation as sections 6 and 7. Density during the summer within the three sections with same aggregate content was highest in section 6 and lowest in section 8. The reverse was found with fall densities although the same temperature relations still existed.

Lower temperature variation and slower rate of temperature change in section 8 were attributed to the combined action of lignosulfonate and lime. Lignosulfonate with alum did not indicate as good insulating properties but appeared better than lignosulfonate alone. The untreated section with higher aggregate content also seemed to have lower thermal conductivity than section 6.

Similar indications of relative thermal conductivity of the treated layers of section 6 and section 8 were found by the following analysis. Basic theory could be found in most elementary textbooks dealing with heat flow.

Quantity of heat flow through a material is dependent upon (1) thermal conductivity of material, (2) surface area and thickness of material, (3) time interval, and (4) temperature differences. This analysis considered geometry and time to be constant between sections, leaving heat flow proportional to thermal conductivity and temperature.

During early morning hours (Fig. III-74) ambient temperature was nearly constant for four hours, then rose from 65 ^{o}F at 6:30 to 94 ^{o}F at noon. Temperature at the bottom of the treated layer in section 6 began to rise from 75 ^{o}F at 7:30 to 83 ^{o}F at noon. Temperature in the similar location in section 8 continued to decrease until 9:30, at which time it increased from 77 ^{o}F to 79 ^{o}F at noon. Subgrade temperature in section 6 was the same at 6:30 and noon, while subgrade temperature in section 8 was still decreasing at noon.

These observations showed that while ambient temperature increased 29 °F, base temperatures increased 8 °F in section 6 and 1 °F in section 8. Subgrade temperature in section 6 was the same at the beginning and end of the time interval while the subgrade temperature in section 8 continued to decrease another 2 °F. Although thermal conductivities of the treated soils were not directly measured these temperature measurements provided a good indication of relative conductivity of treatments. When subjected to the same ambient temperature change, heat flowed more rapidly through the 1% lignosulfonate treated soil and slowest through the 1% lignosulfonate plus 0.5% lime treated soil. Lowering rate of temperature variations reduced number of freeze-thaw cycles to which subgrade was subjected.

Extracted Lignosulfonate

Determinations of amount of lignosulfonate present in a section were made on samples dug out of each section of the Marion test road in the summer of 1971. Samples were taken as a representative portion from balloon density moisture content material after completion of dry density computations.

Weighed quantities of soil mixture were put into beakers and boiled with water to dissolve lignosulfonate. Gooch crucibles were used to filter contents of each beaker and several washings removed as much lignosulfonate as possible. Filtered solution was oven heated at 110 °C and water evaporated. Extracted lignosulfonate remaining was weighed and presented as a percent of original weight of oven dry soil mixture. Impurities which may have dissolved and appeared in extract were found by tests on samples from untreated sections to be insignificant.

Determinations resulted generally in lignosulfonate plus lime sections producing least extracted lignosulfonate (Fig. III-75). Either lignosulfonate had been leached out of these sections since construction or the reaction of lime with lignosulfonate produced an insoluble

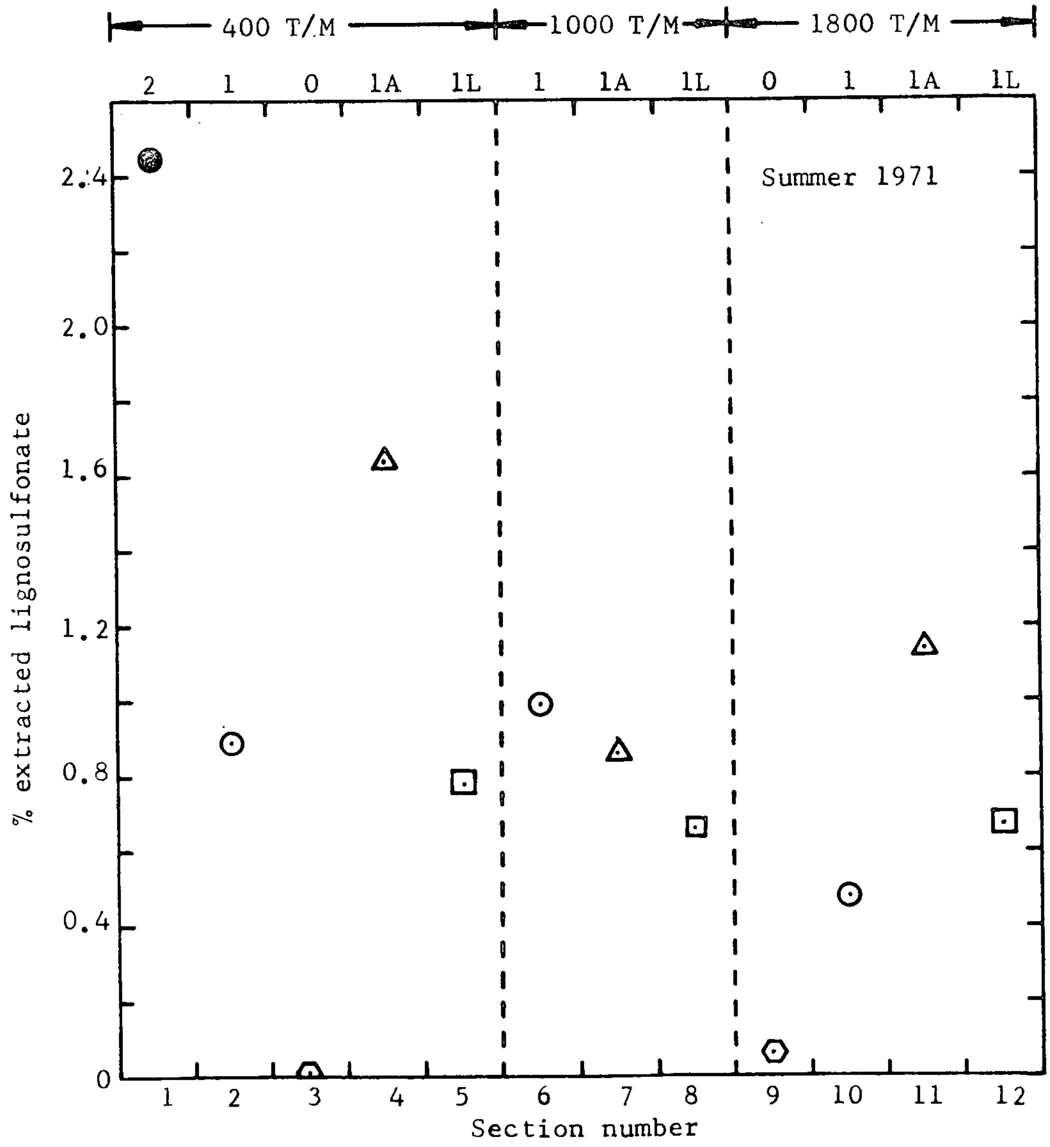


Fig. III-75. Extracted lignosulfonate from test section soil samples, summer 1971, Marion test road.

material which could not be removed from the soil mixture. Variations in amount of lignosulfonate originally added was a possible explanation for differences between theoretical and extracted lignosulfonate contents. However if less than recommended amount was used, the tighter aggregate discussed in previous pages was even more indicative of the superiority of lignosulfonate plus lime.

It was felt that a chemical reaction between lime and lignosulfonate was the most likely cause of those sections producing least extracted lignosulfonate of treated sections. Being water soluble, alum may have appeared in extracted lignosulfonate by soil samples from sections containing lignosulfonate and alum. This could have accounted for the concentration of extracted lignosulfonate being higher than the amount of lignosulfonate added during construction. Observation on extract of soil samples from untreated sections showed soluble components less than 0.1% of initial soil weight which indicated that impurities did not greatly influence final extracted lignosulfonate quantities.

LIGNOSULFONATE CONCENTRATION IN SURFACE RUNOFF

The question was asked whether or not lignosulfonate stabilization replaced one type of pollution with another, reducing air pollution while polluting runoff water into which lignosulfonates dissolved. Calculations showed that even under most severe conditions concentration of lignosulfonate solids did not approach the 4% maximum concentration for animal ingestion permitted by the Food and Drug Administration in CFR 121.234[2].

By in large, this investigation evaluated the effect of the addition of 1% of lignosulfonate solids on a dry soil weight basis. However for this analysis a 2% lignosulfonate stabilized roadway 24-ft-wide treated 4 in. deep was considered. Using an average dry density of 120 pcf and a cross section of roadway 1 ft in length and 4 in. deep there were 960 lb of soil. Lignosulfonate solids added at the 2% rate amounted to 19.2 lb in this volume of road material.

Field observations were that surface runoff leached to a depth of 1/4 in. during a rainstorm. In the volume under consideration the unrealistic assumption was made that 100% of the lignosulfonate solids in the top 1/4 in. went into solution during a single 1-in. rain. This amount of water, 1 in. deep over the 24-ft wide by 1-ft long section of road, was 2 ft^3 or 124.8 lb. In the top 1/4 in. of road material was located 1/16 of the 19.2 lb of lignosulfonate solids or 1.2 lb. Dissolving 1.2 lb of lignosulfonate solids in 124.8 lb of water produced a concentration of about 1%. This did not consider dilution during ponding by water from adjacent lands and was well below the allowable FDA 4% concentration.

ECONOMICS

Cost estimates were made for treatments similar to test sections studied. Price of lignosulfonate was $0.28 per gal. with an average freight cost of $0.05 per gal. for an average lignosulfonate cost of $0.33 per gal. with 5 lb of lignosulfonate solids per gallon. Secondary additives, alum or lime, cost about $1.00 per 100 lb.

Typical dry density was 120 pcf and width of roadway was 24 ft with a 4-in. depth of treatment. Using these figures material cost for a 1% lignosulfonate treatment was $3350 per mile and required 10,140 gal. of lignosulfonate per mile or 0.72 gal. per yard of roadway surface. Secondary additive cost at a concentration of 0.5% by dry soil weight or 1.8 lb per square yard was $250. Total material cost with secondary additive was $3600 per mile.

Attempts to use a waterproof surface over a lignosulfonate stabilized material were successful and except for a hot mix mat total costs for base stabilization and surface seal were not over $10,000. For example, Linn County reported total treatment costs for 1 gal. lignosulfonate and 0.1 lb alum per square yard for a 27 ft wide surface either 4 in. or 6 in. deep as $6500. However, this average included a considerable amount of experimentation of application methods which increased costs. Seal coat cost for 0.25 gal. MC cutback and 25-lb chips per square yard on a 27-ft width was $2000. Double application of seal coat was $3500. Contract cost for a 2-in. asphaltic concrete mat 24 ft wide was $18,000 to $24,000 per mile (private communication, Linn Co. Engr. Office, May 1971).

Lee County has used a local lignosulfonate since before 1960 to treat unpaved secondary roads. Lignosulfonate available was only 11% solids and created a disposal problem for the manufacturer, so the only coat was for hauling, which was done by the county. Very comprehensive cost figures were kept on projects and provided the basis for the following analysis.

Construction process used was a simple procedure and involved little equipment and manpower. Two blade graders and a spray truck were required and the operation was mostly maintenance, starting with the first application. Two blade graders passed over a roadway, and loose aggregate and shallow cut material were placed in a windrow along one edge. The spray truck distributed lignosulfonate on the roadway and graders spread windrow evenly over lignosulfonate. Compaction was provided by normal traffic.

This process was repeated two or three times during the summers, depending on traffic and weather. Heavy traffic and dry weather degraded the surface quicker and were causes for periodic replenishment. With such a simple efficient operation, over the years an in-depth treatment eventually developed. Aggregate replacement was required less frequently. Dust observations showed treatment reduced dusting by about 80%[25].

One such project was 7.3 miles long with a 1969 ADT of about 300 vehicles per day. Treatment began in 1961 and annual quantity of lignosulfonate applied, increased during the following several years. Highest annual cost per mile for this project was $807.04 in 1968. Cost included aggregate, application, and hauling costs.

Local lignosulfonate was used that year at a rate of 45,324 gal. per mile which was equivalent to 6125 gal. of commercial concentrate. Cost per mile at the commercial price for lignosulfonate would have been about $2000 resulting in total cost per mile of around $2500 or a threefold increase.

The previous analysis considered highest annual cost of about $800 per mile for a particular project. High cost resulted from over 2000 tons of crushed stone being applied to the road during that year. A e average annual cost per mile taken from several projects was out $400. Based on the average amount of lignosulfonate used, commercial lignosulfonate cost would have made commercial annual cost per mile about $2200 or an increase of over five times.

Treating roads in Lee County using commercial lignosulfonate would have increased annual costs three to five times with over 140 miles of treated roads being maintained. Lee County could not afford such widespread treatment if required to purchase lignosulfonate at the commercial rate.

CONCLUSIONS

This study investigated the effectiveness of ammonium lignosulfonate alone and in combination with calcitic lime or aluminum sulfate as secondary additives in dust palliation and surface improvement of unpaved secondary roads. Laboratory investigations were made on field materials sampled from roads in Clinton, Floyd, and Linn Counties prior to treatment. Field tests made on these roads before and after construction included in-place density determinations, dust and loose aggregate measurements, Benkelman beam and spherical bearing tests, and base and subgrade temperature recordings. The following conclusions are based on analysis of these tests and investigations.

Laboratory Investigations

1. Unconfined compressive strength increased with increasing additive contents.

2. Resistance to disintegration under water immersion increased with increasing additive contents.
3. Dry density at optimum moisture content increased with increasing lignosulfonate content.
4. Moisture content required to produce maximum density, decreased with increasing lignosulfonate content.
5. Probable stabilizing mechanisms were ionic interactions up to 0.5% and adhesive bonding above 1.5% lignosulfonate concentration.

Field Studies

1. All treatments reduced dusting up to 80% compared to untreated roadways.
2. 1% lignosulfonate treatment was as effective as 1% lignosulfonate plus 0.5% of either lime or alum for dust palliation only.
3. 2% lignosulfonate, compared to 1% lignosulfonate stabilization, was not warranted on the basis of cost or observations of dry density, loose roadway aggregate, and strength tests.
4. 1% lignosulfonate plus 0.5% lime treatment produced a tighter surface and less loose aggregate than any other treated or untreated roadway at comparable aggregate content.
5. More improvement at less cost was realized by inclusion of secondary additives than by addition of more lignosulfonate.
6. Best insulating properties were indicated by treated surface containing 1% lignosulfonate plus 0.5% lime.

7. Decreased solubility of lignosulfonate was indicated with lime as a secondary additive.
8. Surface runoff was slightly discolored by low concentration, less than 1%, of lignosulfonate.
9. Treated roadways with aggregate content less than 1800 tons per mile were slick when wet.
10. Although lignosulfonate is available in almost unlimited quantities, its use in roadway treatments in Iowa will probably be limited due to the present delivered cost of the lignosulfonate.

APPENDIX

Supplemental Lignosulfonate/Secondary Additives Study

Screening studies with Guar derivatives and sodium silicate, were completed both separately and in combination with lignin. 2 X 2 in. compacted specimens were tested with varying quantities of additives, curing periods, and saturation by capillarity.

The soil material in this study was removed from the untreated 1000 ton per mile aggregate section of the Marion test road. Properties of the soil are as follows:

Textural Composition

Gravel	48%
Sand	24%
Silt	20%
Clay	8%

Physical Properties

Liquid limit	25.4%
Plasticity index	11.2%
Specific gravity	2.71
Standard dry density	137.5 pcf
Standard optimum moisture	8.0%

Textural Classification	Gravelly loam
Engineering Classification	A-2-6(0)

The chemicals used in this study were a dry powder ammonium lignosulfonate, chemically pure sodium silicate ($Na_2SiO_2 \cdot 5\ H_2O$), and a Guar derivative termed Polymer JB by the distributor, Jaguar Products.

Shown in the attached table are results of the unconfined compressive strength tests at 0, 6, and 10 days air cure, 10 days air cure plus 24 hours of exposure to a 100% humid atmosphere, and 15 days air cure plus 24 hours of exposure to capillary saturation at the base of each

Additive content, % dry soil weight			Molded moisture content, %	Molded dry density, pcf	Unconfined compressive strength, lb				
Lignosulfonate	Sodium silicate	Polymer JB			0-day air cure	6-day air cure	10-day air cure	10-day air cure 24 hr 100% humidity	15-day air cure, 24 hr capillary saturation
0	0	0	7.6	137.5	82	1700	1700	950	0
1	0	0	8.0	139.5	37	1770	1840	450	66
1	1/2	0	8.5	136.8	75	1600	1650	410	35
1	1	0	8.3	136.6	116	1340	1450	390	30
1	0	1/2	8.3	137.1	60	2380	3040	700	65
1	0	1	7.8	136.7	78	2400	2500	770	61
1	0	1-1/2	8.4	133.4	123	2300	2540	970	25
2	0	0	7.1	137.3	66	1550	3500	1000	60
2	1/2	0	7.5	137.0	80	1950	2800	645	30
2	1	0	7.4	136.6	129	1900	2800	525	10
2	1-1/2	0	7.8	136.1	95	1950	1850	310	50
2	2	0	8.4	135.1	106	1800	2650	700	75
2	0	1/2	8.1	135.5	71	2300	3900	960	490
2	0	1	7.7	134.5	98	2800	3950	1300	235

Additive content, % dry soil weight			Molded moisture content, %	Molded dry density, pcf	Unconfined compressive strength, lb				
Lignosulfonate	Sodium silicate	Polymer JB			0-day air cure	6-day air cure	10-day air cure	10-day air cure 24 hr 100% humidity	15-day air cure, 24 hr capillary saturation
2	0	1-1/2	7.4	133.5	110	3000	4200	1325	90
0	1/2	0	7.2	137.4	138	1700	1850	310	50
0	1	0	8.2	136.8	128	2000	2650	700	75
0	1-1/2	0	7.6	138.0	99	1950	1180	900	20
0	0	1/2	7.9	139.2	82	1220	1535	650	12
0	0	1	7.7	137.5	168	1450	1550	750	11
0	0	1-1/2	8.1	134.9	130	1260	1320	725	18
0	0	2	7.8	131.4	135	1350	1350	500	17

specimen. Strength results are the average of three specimens each, while molded moisture contents are the average of four determinations made during molding, and dry densities are the average of the 15 strength specimens.

With the exception of the Polymer JB only specimens, the 1-1/2% silicate specimens, and the 1% lignosulfonate plus 1% silicate specimens, all additives increased in strength with increased air curing time. This was probably due only to decreased water content since exposure to a 100% humid atmosphere significantly reduced strength of all chemical combinations and capillary saturation created even greater reductions in strength, though curing time was increased an additional five days.

Lowest treated strengths were observed with the Polymer JB only specimens, which after 15 days air cure and 24-hour capillary saturation were even difficult to handle during placement in the testing machine. Highest saturated strengths were obtained with the combination of 2% lignosulfonate and Polymer JB. It can be noted however that the 2% lignosulfonate-1-1/2% Polymer JB produced consistently higher air cure and 100% humidity exposure strengths while the 2% lignosulfonate plus 1/2% Polymer JB had the highest 15-day air cure-24-hours capillary saturation strength of the respective combinations. While an exemplary reaction of some type may have thus occurred with the combined lignosulfonate and Polymer JB, as was pointed out previously in this report, economics of use of 2% lignosulfonate was considered beyond the guidelines of this study. The addition of either 1/2 or 1% Polymer JB would further increase the additive costs and in addition would involve either (a) combining two dry products prior to construction spreading, or (b) two separate construction spreading and mixing operations. Thus the combined economics and construction practice were considered impractical for the scope of this study and further testing was eliminated.

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PART IV

SOIL-CHEMICAL ADDITIVES AS SURFACE IMPROVEMENT AGENTS FOR UNPAVED ROADS

by

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CONTENTS

INTRODUCTION

The use of polyester and thermo plastic resins as soil stabilizers is relatively new and untried. The objective of this research was to investigate the capabilities of these resins and several other chemicals and commercial dust palliatives as potential low-cost dust palliatives and surface improvement agents. The investigation was conducted entirely in the laboratory but care was taken to approximate field conditions as accurately as possible.

Various soil-chemical additives were evaluated on the basis of the following tests:

1. Unconfined compressive strength
2. Durability or erosibility
3. Trafficability
4. Resistance to freezing and thawing
5. Moisture retension and density.

Some of the tests were standard while others were specifically modified, but in every case the treated specimens were compared with untreated control specimens.

Materials

Soil

Samples of in-situ material were obtained directly from an existing unimproved county road northwest of Ames to a depth of 6 in. This soil is considered representative of those found on many of Iowa's unpaved secondary roads, and laboratory test results were considered as being applicable throughout the bulk of state.

The engineering properties of the soil are given in Table IV-1.

Table IV-1. Soil properties.

Textural composition		
Gravel	(> 2.0 mm)	0 %
Coarse sand	(2.0 to 0.42 mm)	38.2
Fine sand	(0.42 to 0.074 mm)	23.7
Silt	(0.074 to 0.005 mm)	21.4
Clay	(< 0.005 mm)	16.7
Colloid	(< 0.001 mm)	12.2
Textural classification		Sandy loam
Engineering classification		A-2-4(0)
Physical properties		
Liquid limit	28.1%	
Plastic limit	19.5%	
Plasticity index	8.6%	

Additives

The additives evaluated were quite varied but generally fell into two basic categories: (a) polymers and polymeric compounds commonly referred to as plastics and (b) commercial dust palliatives and stabilizers. The additives are listed below and manufacturer names are included for the reader's benefit only, without implications of endorsement.

Requirements for an ideal chemical additive are:

1. Water-soluble or water dispersible when being mixed with the soil
2. Should bond to and link soil mineral particles
3. If not a chemical bonding type, it should waterproof compacted soil
4. Become insoluble after reaction is complete

Table IV-2. Additives.

Manufacturer/distributor	Trade name/product		Chemical
Petro Chemical Co.	Petro D Dust	(l)[a]	Proprietary[b]
Central Chemical Co.	SA-1	(l)	Proprietary
	Kelpak	(l)	Proprietary
	Clapak	(l)	Proprietary
	Claset	(l)	Proprietary
Dupont de Nemours & Co.	Elvanol 71-30	(S)[a]	Polyvinyl alcohol
	Elvanol 52-22	(S)	Polyvinyl alcohol
Dow Chemical Co.	PVC-133-4	(S)	Polyvinyl chloride
	Resin P-5-2	(S)	Polystyrene
	301-Natural	(S)	Polypropylene
Freeman Chemical Co.	Stypol 40-2417	(l)	Polyester resin
	Stypol 40-5020	(l)	Polyester resin
Interplastic Corp.	Corezyn 101	(l)	Polyester resin
	Corezyn 1323	(l)	Polyester resin
Rohm & Haas	Paraplex P-13	(l)	Polyester resin
	Paraplex P-444A	(l)	Polyester resin

[a] l = liquid form; s = solid form.

[b] Proprietary: chemical constituency not released by company.

5. Should be economical to use

6. Should resist biochemical decomposition.

Insofar as was possible, the additives were all evaluated relative to the above criteria. Solutions were prepared to determine the degree of solubility of the additives in two common solvents: water and benzene. In addition, qualitative measurements of the ease of mixing were also noted. The solubilities are shown in Table IV-3.

Benzene, due to its toxicity and volatility, was not considered practical as a solvent for field application. Those products requiring

Table IV-3. Solubilities of additives.

Product	Solubility		
	Water		Benzene
	Cold	Hot	
Petro D Dust	S[a]	—	—
SA-1	S	—	—
Clapak	S	—	—
Claset	S	—	—
Kelpak	S	—	—
Elvanol 71-30	i[a]	S	—
Elvanol 52-22	δ[a]	S	—
PVC 133-4	i	i	i
Resin P-5-2	i	i	S
301-Natural	i	i	δ
Stypol 40-2417	i	i	S
Stypol 40-5020	S	—	—
Corezyn 101	i	i	S
Corezyn 1323	i	i	S
Paraplex P-13	i	i	S
Paraplex P-444A	i	i	S

[a] S = soluble; i = insoluble; δ = partially soluble.

benzene as the transporting/mixing solvent were thus eliminated from further consideration. The following additives were retained for laboratory screening due to their solubility in water.

1. Petro D Dust
2. SA-1
3. Kelpak
4. Clapak
5. Claset
6. Elvanol 71-30
7. Elvanol 52-22
8. Stypol 40-5020

The next step in evaluation of additives was determination of economic additive percentages. Based upon a 6-in. depth of treatment and a unit weight of 110 pcf, percentage weights of the selected additives per square yard were computed, and when multiplied by the manufacturer's unit cost, additive cost per square yard of treatment was determined. Cost per square yard for various percentages is shown in Table IV-4.

Table IV-4. Cost of additive, dollars/square yard. Additive content, % of dry soil weight.

Additive	0.05	0.10	0.20	0.30	0.50	1.0
Petro D Dust	$0.06	$0.12	$0.25	$0.37	$0.62	$1.20
SA-1	0.34	0.67	1.35	2.02	3.37	6.70
Kelpak	0.21	0.42	0.84	1.26	2.10	4.20
Clapak	0.21	0.42	0.84	1.26	2.10	4.20
Claset	0.21	0.42	0.84	1.26	2.10	4.20
Elvanol 71-30	0.11	0.22	0.45	0.67	1.11	2.20
Elvanol 52-22	0.14	0.27	0.54	0.82	1.35	2.70
Stypol 40-5020	0.08	0.16	0.32	0.48	0.80	1.60

Initially, Table IV-4 provided a guide for selection of maximum range of additive contents for laboratory evaluation based on an arbitrary cost limit of $1.00 per square yard. As testing progressed, it became apparent that such a guideline was somewhat infeasible, in that it imposed a narrow range on several of the additives, which did not allow workable graphical representation of maximum additive content versus results of the conducted test. The following additive contents, expressed as a percentage of dry soil weight, were ultimately evaluated.

Petro D Dust	0.05, 0.1, 0.25, 0.50, 1.0, 2.0
SA-1	0.01, 0.10, 0.20
Kelpak	0.01, 0.10, 0.20
Clapak	0.01, 0.10, 0.20
Claset	0.01, 0.10, 0.20
Elvanol 71-30	0.10, 0.25, 0.50, 1.0
Elvanol 52-22	0.10, 0.25, 0.50, 1.0
Stypol 40-5020	0.25, 0.50, 1.0

Combination of Claset and Clapak < 0.1 each

UNCONFINED COMPRESSIVE STRENGTH

The unconfined compression test has been used by many investigators in the study of stabilized soils. It is fast, economical, simplistic, and requires no special equipment. The test itself is a measure of strength rather than stability but the correlation between the two is considered good.

Sample Preparation, Curing, and Testing

The required quantities of soil materials needed for this portion of the study were passed through a 3/8-in. crusher to reduce the size of the coarse aggregate to less than one fifth of the final specimen diameter. While sieving the material through a 5/8-in. sieve would have accomplished much the same affect, most of the aggregate would have been eliminated. Since it was felt that the aggregate surfacing material was an integral part of the soil mixture, it was thus retained by reducing its size.

A predetermined amount of additive, dissolved in distilled water, was added to a measured quantity of soil and thoroughly mixed, both by hand and a mechanical mixer, until a uniform mix was obtained. Additional quantities of distilled water were added, when necessary, to bring the mixture to the desired moisture content. Mixing water was heated to approximately 160 °F prior to the addition of Elvanols 71-30 and 52-22 due to insolubility in cold water.

Cylindrical specimens of the soil-additive mix were compacted to 2 in. diameter and approximately 2 in. in height, were extruded following compaction, marked for identification, measured for height and weight, and allowed to air cure at room temperature at approximately 72 °F. The following cure periods were initially selected for evaluation:

1. immediate-no cure
2. 1-day air cure
3. 1-day air cure followed by 1-day water immersion
4. 7-day air cure
5. 7-day air cure followed by 1-day water immersion.

The cure periods involving immersion were later eliminated because the specimens either failed immediately or were too soft to test following immersion, leaving only air cures of 0, 1, and 7 days for evaluation. For the objectives of this study, i.e., dust palliation and general surface improvement, benefits of immersion testing were considered questionable.

Identical specimens were molded from each batch mix, and the strength values reported are the average of four tests. Moisture contents and dry densities referred to herein are those computed at time of molding unless otherwise stated, and are shown in Tables IV-5 and IV-6.

Table IV-5. Density variations of unconfined compression specimens.

Product	Additive % dry soil weight	Approximate liquid content % dry soil weight	Wet density, pcf	Dry density, pcf
Untreated	9.0	9.0	140.0	128.4
Petro D Dust	0.05	9.0	140.0	128.4
	0.10	9.0	139.9	128.3
	0.25	9.0	139.9	128.3
	0.50	9.0	139.2	127.7
	1.00	9.0	139.1	127.2
SA-1	0.01	9.5	140.0	127.9
	0.10	9.5	140.0	127.9
	0.20	10.0	139.6	126.9
Kelpak	0.01	10.0	138.5	125.9
	0.10	10.0	139.6	126.9
	0.20	10.0	141.0	128.2
Clapak	0.01	10.0	139.6	126.9
	0.10	10.0	139.0	126.5
	0.20	10.0	139.1	126.5
Claset	0.01	10.0	138.9	126.3
	0.10	10.0	138.9	127.6
	0.20	10.0	139.3	126.6
Polyvinyl alcohol				
Elvanol 71-30	0.10	10	136.4	123.7
	0.25	10	132.3	120.7
	0.50	10	129.9	118.8
	1.00	10	128.0	116.5

Table IV-5. Continued.

Product	Additive % dry soil weight	Approximate liquid content % dry soil weight	Wet density, pcf	Dry density, pcf
Elvanol 52-22	0.10	10	137.1	124.6
	0.25	10	133.8	121.6
	0.50	10	135.0	122.7
	1.00	10	133.0	120.9
Stypol 40-5020	0.25	9.0	139.7	128.2
	0.50	9.0	140.0	128.4
	1.00	9.0	138.9	127.4
Combined Clapak (.01) and Claset (0.05)		10.0	139.7	127.0

Table IV-6. Percent moisture loss of unconfined compression specimens.

Product	Additive % dry soil weight	Molding moisture content	% moisture lost on curing: 1-day cure	% moisture lost on curing: 7-day cure
Untreated	9.0	9.0	74.9	98.0
Petro D Dust	0.05	9.0	98.0	94.4
	0.10	9.0	94.8	98.7
	0.25	9.0	95.1	99.6
	0.50	9.0	98.9	64.3
	1.00	9.0	84.4	86.6
	2.00	9.0	98.8	85.0
SA-1	0.01	9.5	59.6	94.0
	0.10	9.5	57.1	91.9
	0.20	10	60.3	92.6
Kelpak	0.01	10	64.9	95.6
	0.10	10	63.6	94.9
	0.20	10	68.5	96.6
Clapak	0.01	10	66.3	91.6
	0.10	10	67.1	92.8
	0.20	10	63.4	94.9
Claset	0.01	10	61.0	95.1
	0.10	10	63.1	99.4
	0.20	10	58.2	94.6
Elvanol 71-30	0.10	10	—	—
	0.25	10	—	—
	0.50	10	58.1	89.7
	1.00	10	—	86.2

Table IV-6. Continued.

Product	Additive % dry soil weight	% moisture lost on curing		
		Molding moisture content	1-day cure	7-day cure
Elvanol 52-22	0.10	10	66.0	90.9
	0.25	10	62.9	93.5
	0.50	10	–	92.0
	1.00	10	92.6	–
Stypol 40-5020	0.25	9.0	86.8	95.3
	0.50	9.0	93.9	98.0
	1.00	9.0	92.9	97.2
Clapak (0.10) and Claset (0.05) combined		10	99.6	100+

Test Procedure

The unconfined compressive test was performed at a constant deformation rate of 0.1 in. per min. The load is transmitted through a calibrated proving ring with the maximum load causing failure taken as the specimen's compressive strength. Values reported in Figs. IV-1-9 are in units of load rather than stress for convenience only.

Test Results

Results of the unconfined compression tests are shown graphically on the following pages. Several general trends will be discussed which apply to all of the additives prior to discussing the effects of each in detail.

The graphs indicate initial reduction in strength from untreated to small additive content treatment. Soil strength of both the treated and untreated specimens increases with cure time, although rate of strength increase declines from 1-7 days as compared with 0- to 1-day curing.

Generally, the strength tests of specimens having 0-day cure are not valid reflections of strength for either 1- or 7-day cure specimens, though the 1-day tests tend to reflect 7-day results in most cases. Table IV-6 indicates that over one half of the molding moisture was lost during the first 24 hours and that after 7 days almost all moisture was lost. Any discrepancies are attributed to inaccuracy of measurements and devices.

Stypol 40-5020 Figure IV-1 shows the average unconfined compression results of specimens containing Stypol 40-5020. Strength generally

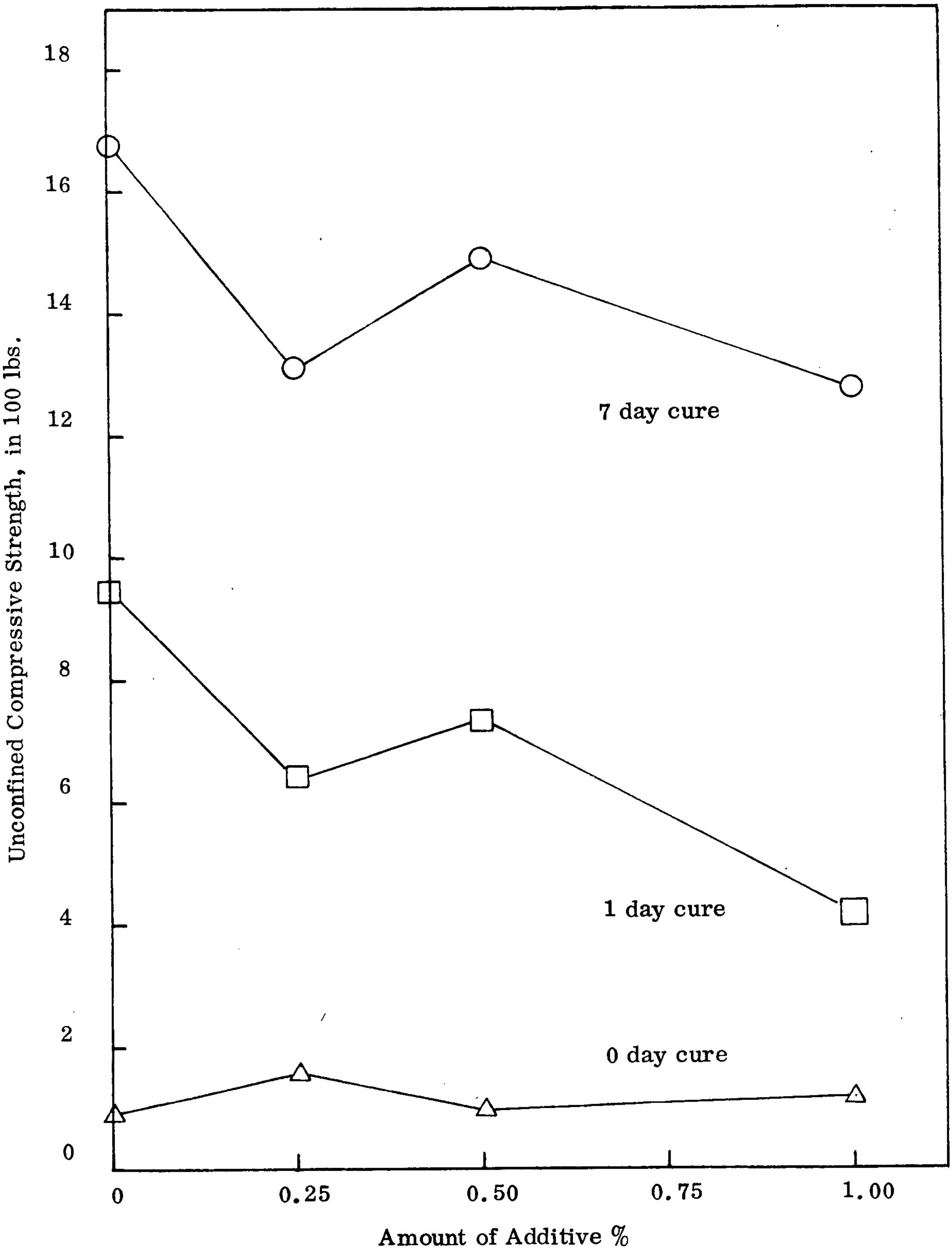


Fig. IV-1. Unconfined strength of Stypol 40-5020.

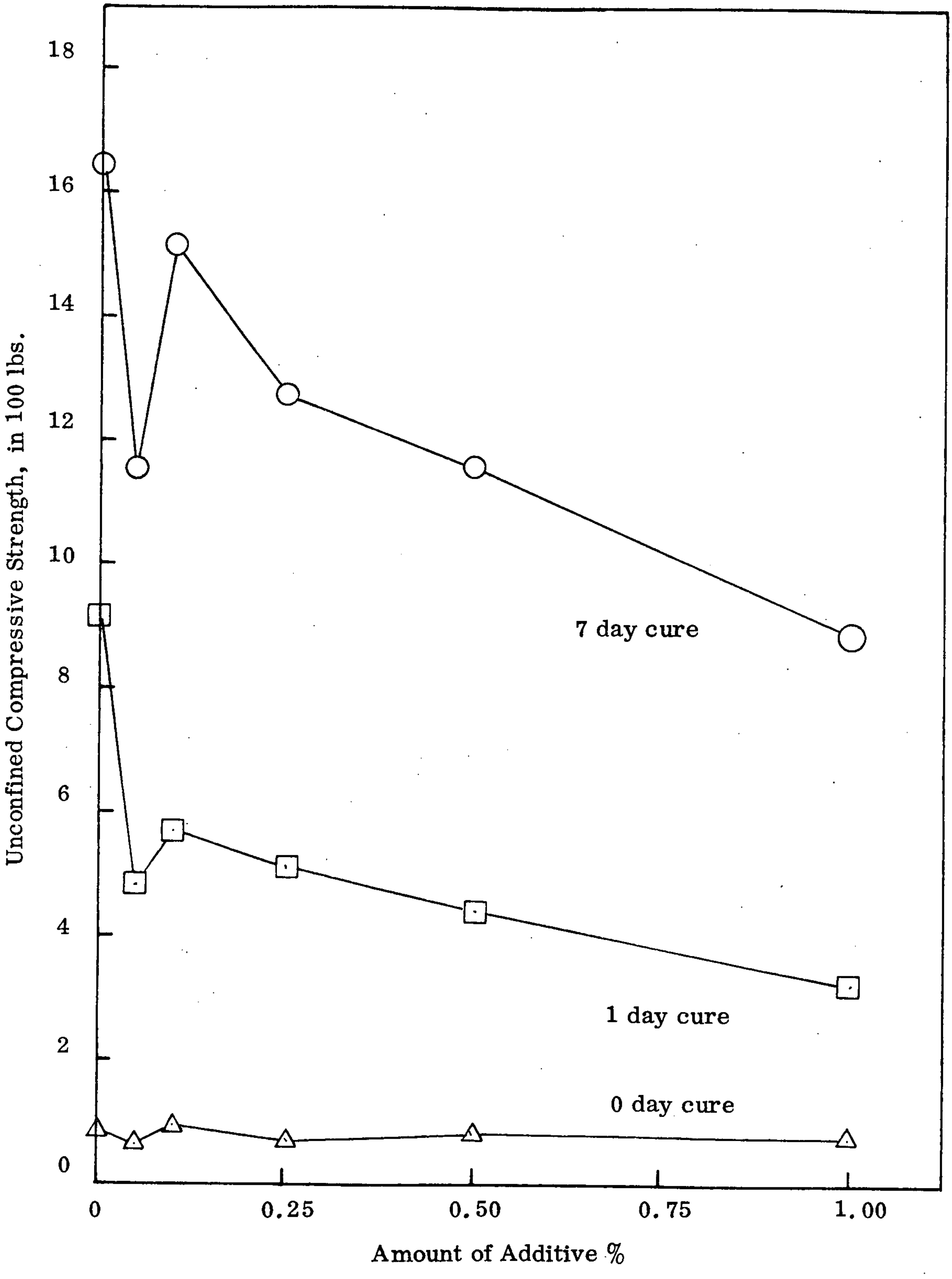


Fig. IV-2. Unconfined strength of Petro D Dust.

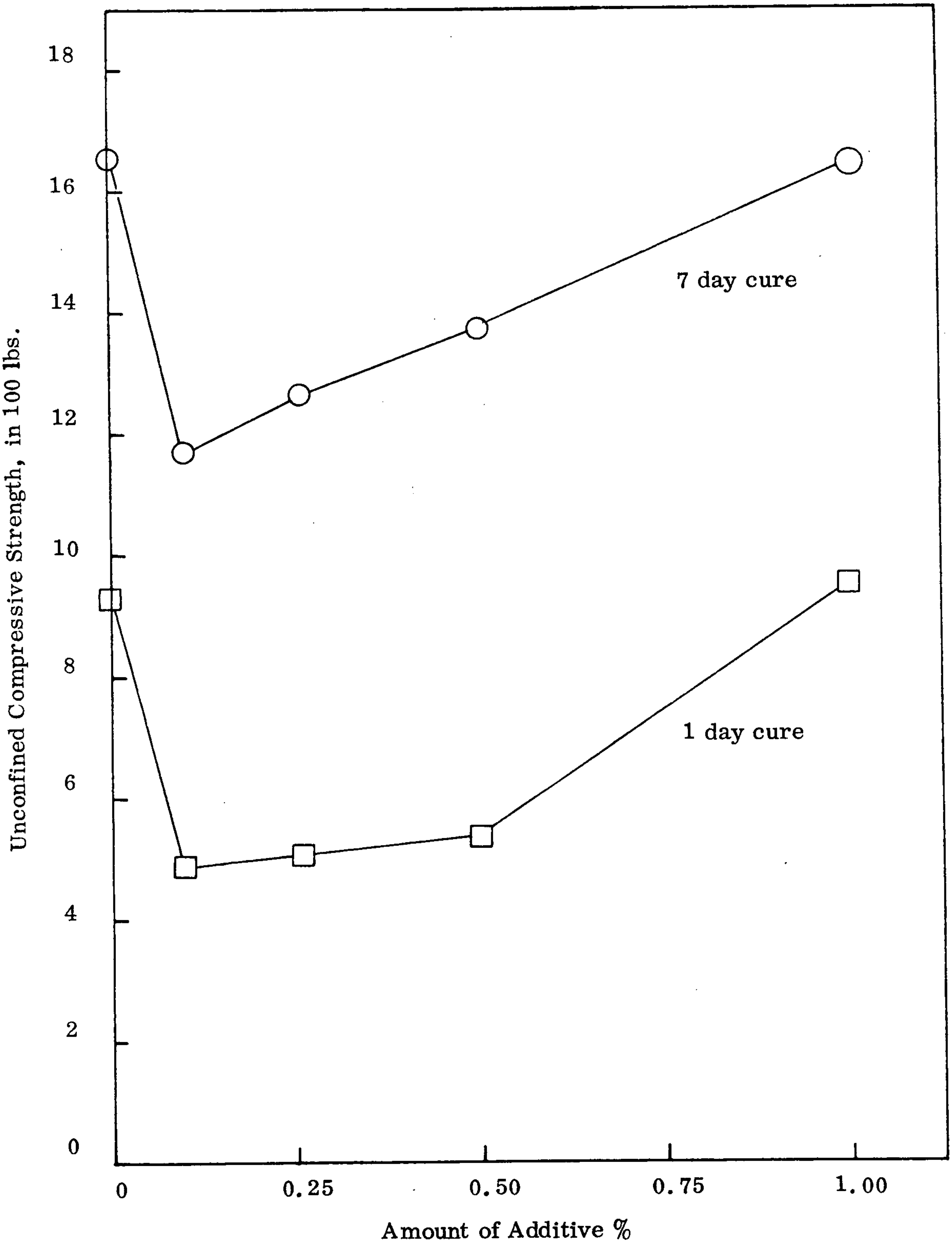


Fig. IV-3. Unconfined strength of Elvanol 71-30.

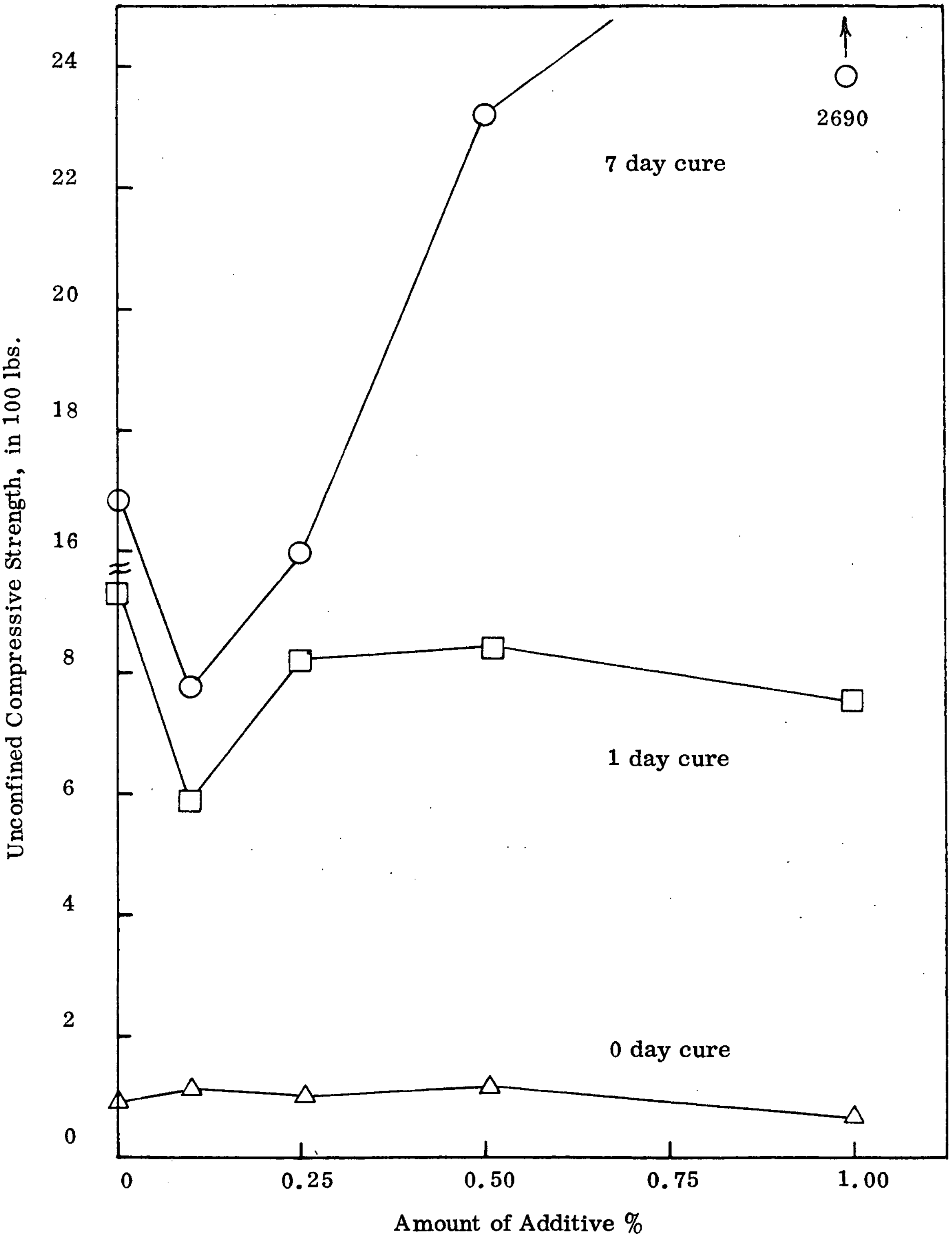


Fig. IV-4. Unconfined strength of Elvanol 52-22.

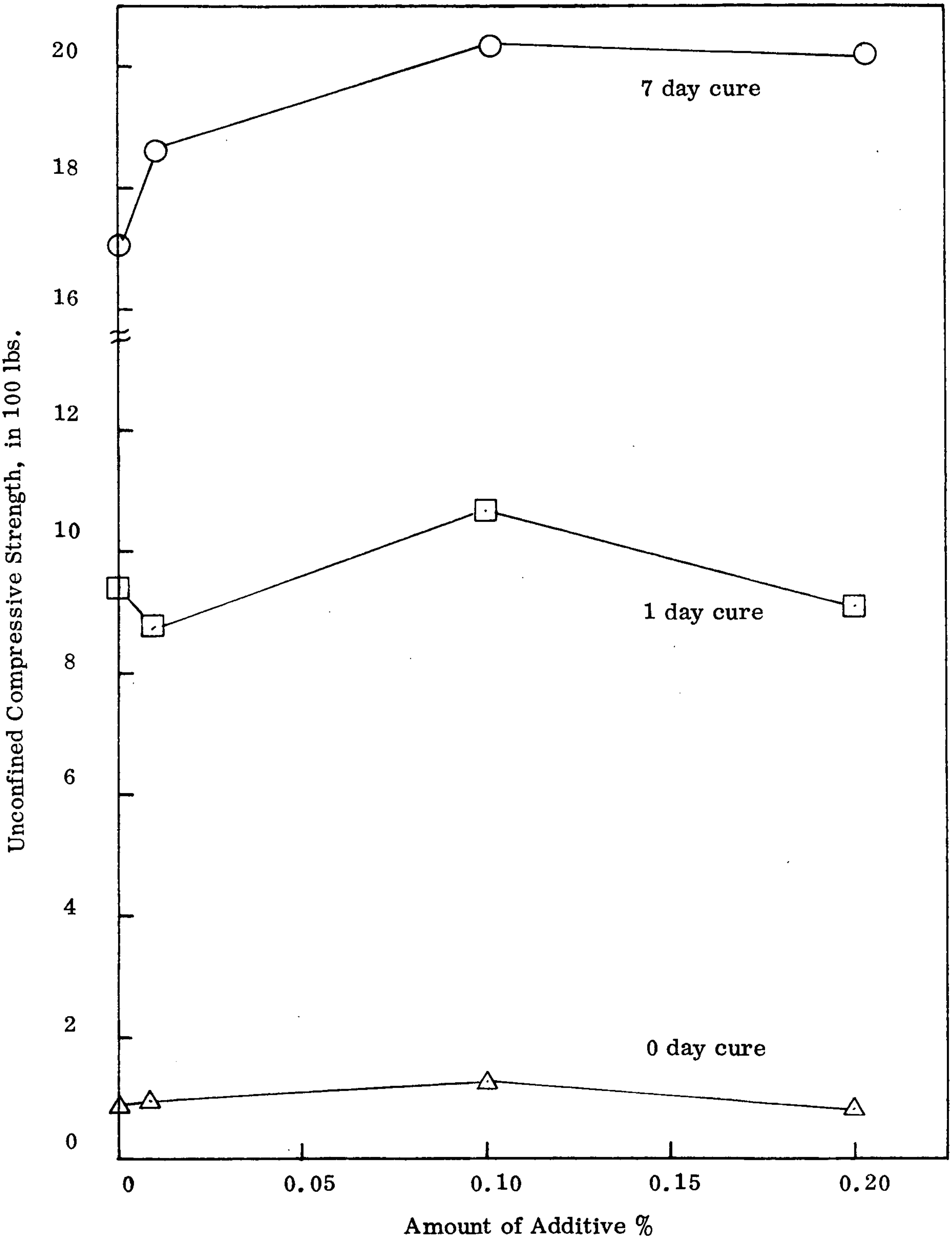


Fig. IV-5. Unconfined strength of Kelpak.

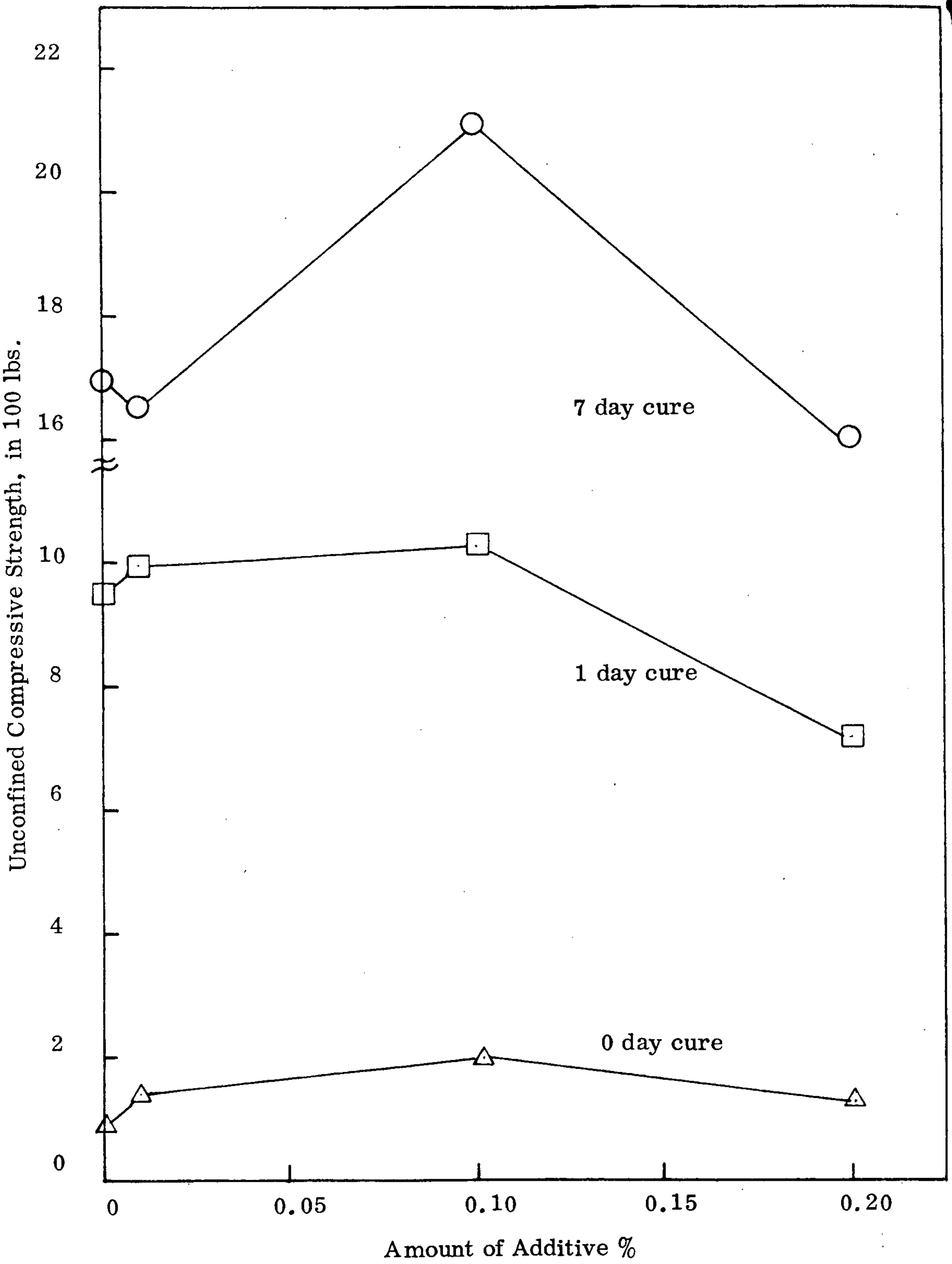


Fig. IV-6. Unconfined strength of SA-1.

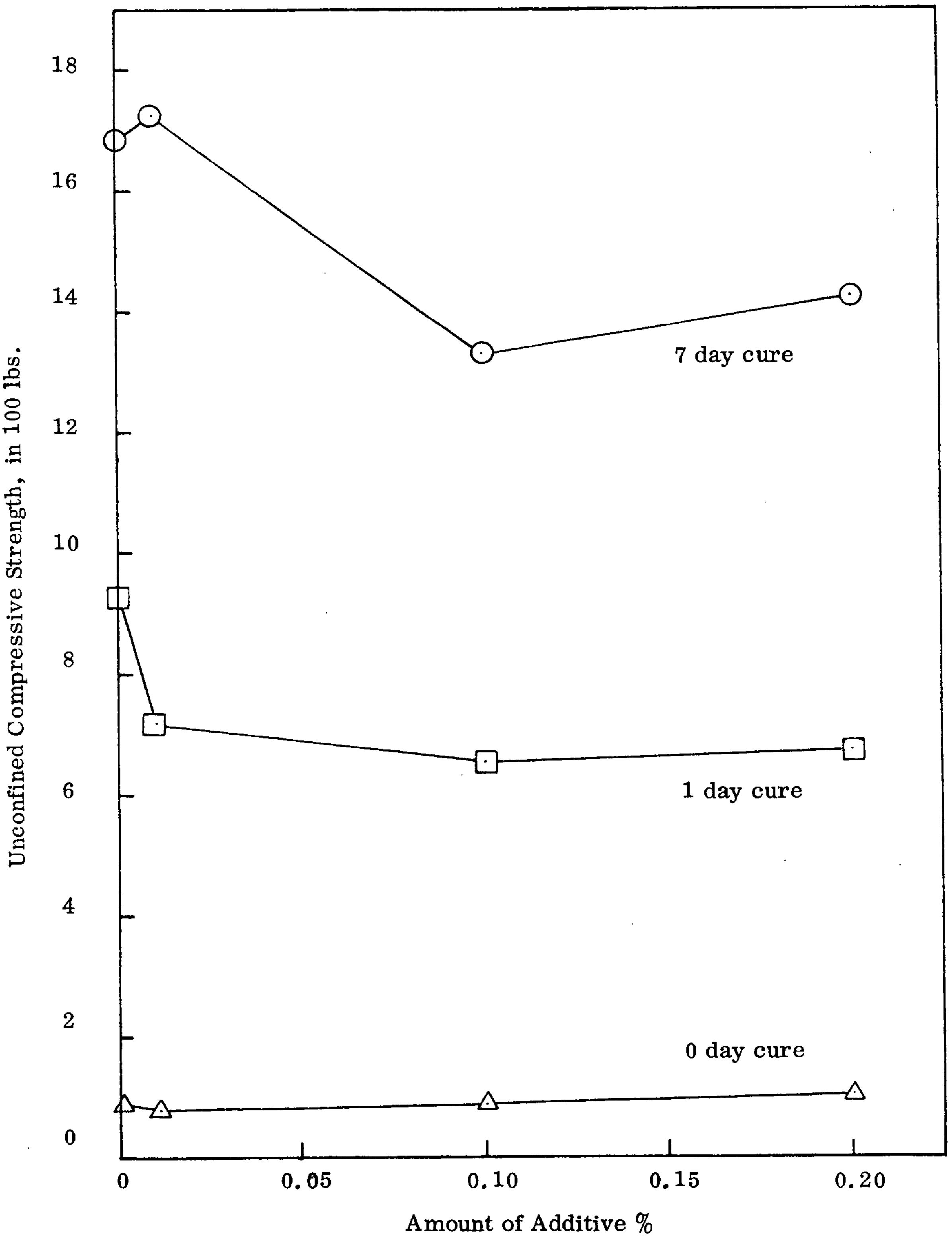


Fig. IV-7. Unconfined strength of Claset.

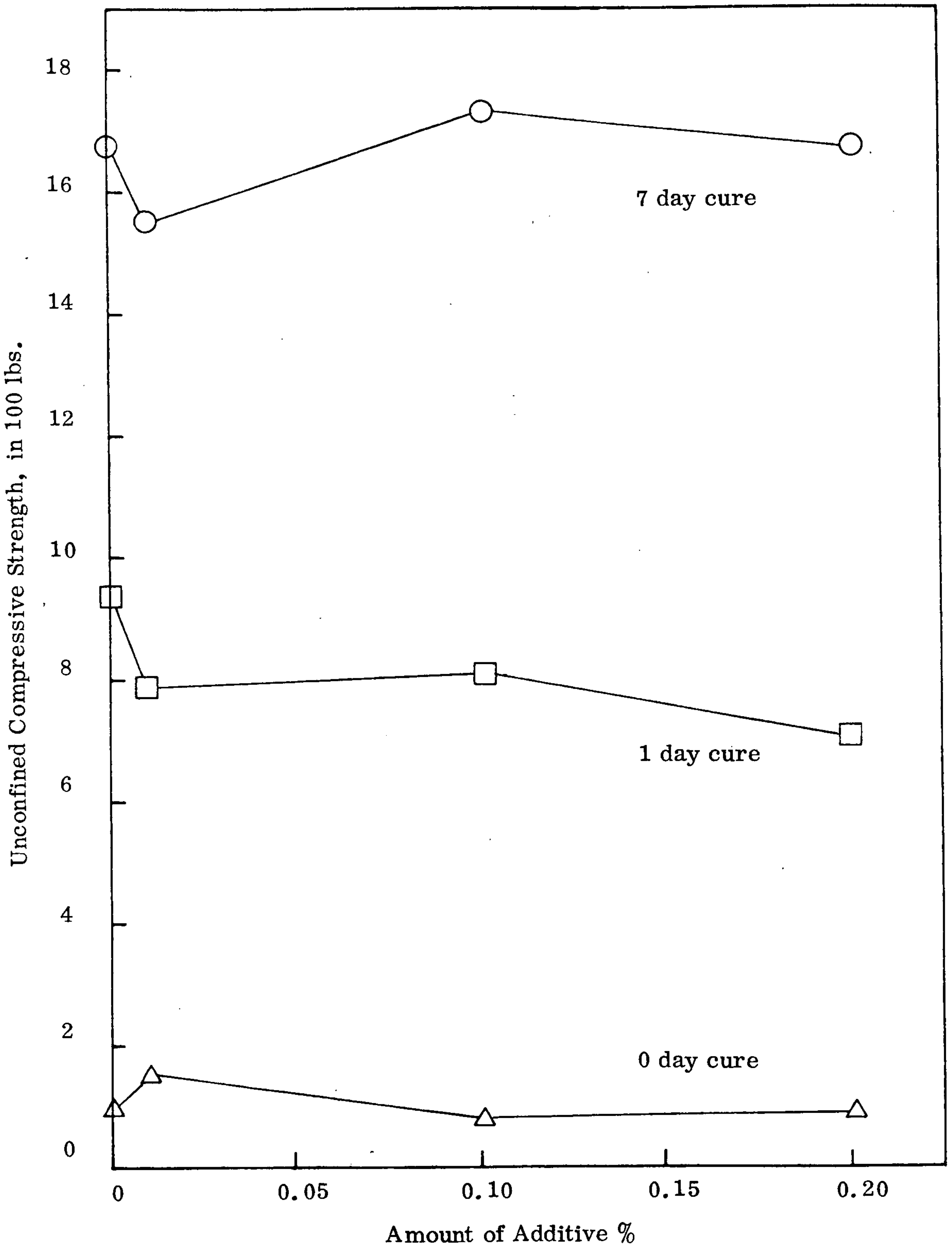


Fig. IV-8. Unconfined strength of Clapak.

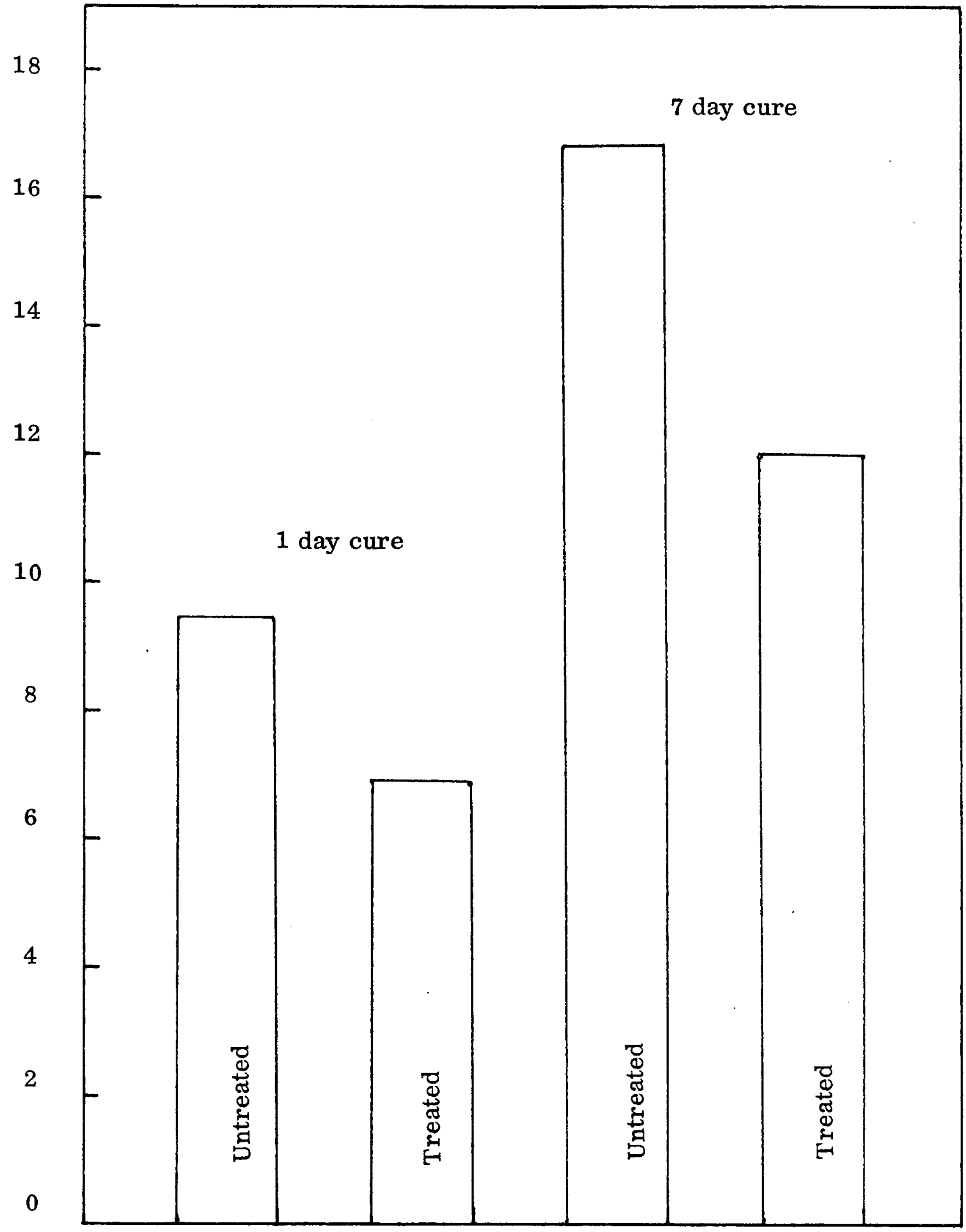


Fig. IV-9. Unconfined strength of Clapak and Claset combined.

decreases with increasing additive content and a corresponding decrease in dry density was also noted. However, an optimum strength is slightly apparent at 0.25%, 0-day cure, and well defined at 0.5%, 1- and 7-day air cures.

Generalized reduction of strength with increased amounts of Stypol may be due to (a) a reduction of cohesion due to soil grain-to-grain lubrication, (b) reduced density, or (c) lack of any chemical reaction. The optimum content of 0.5% Stypol may thus be due to optimization of Stypol-soil bonds. Addition of Stypol above 0.5% probably increased coating thickness on the grains, reducing cohesion, soil bonds, and stability in a manner similar to optimization of asphalt binders.

It should be noted that 0.5% Stypol reduces the strength by only about 200 lb (1700-1500) when compared with untreated 7-day cure specimens.

It should also be noted that the very slight optimum at 0.25% Stypol, 0-day cure, may only be due to water-soil bonds which later disappear during curing.

<u>Petro D Dust</u> Results of Petro D Dust are shown in Fig. IV-2. Tests on specimens containing 2% additive are not shown but lie on extensions of the lines drawn through 1/2, 1/4 and 1% for each curing period. In general, unconfined compressive strength decreased with increasing additive content, coupled with a corresponding slight decrease in dry density. It should be noted that strength for both 1- and 7-day cure periods converges toward 0-day strength at higher additive contents. Strength decrease may be due to decreased cohesion caused by lubrication of soil particles or interference of formation of high strength water bonds which form as the water layer becomes thinner due to evaporation.

There is an optimum at 0.10% which becomes more readily apparent as cure time increases. This may be due to optimization of Petro D Dust-soil-water bonding action, although the reason is not known and is merely conjective.

The potential for Petro D Dust as a seal coat was realized but no unconfined compression tests were performed. It was felt that any strength test would only be indicative of untreated material rather than a seal coat. Petro D Dust was observed to have a shallow penetration rate into the compacted soil, using an application rate of 0.2 gal/sq ft.

Elvanol Results of Elvanol grades 71-30 and 52-22, are shown in Figs. IV-3 and IV-4 respectively. Strength decreased with introduction of slight amounts of Elvanol but increased as the additive content increased beyond 0.1%. Density decreased as additive concentration increased (Table IV-5).

For concentrations above 0.25%, strength of specimens containing Elvanol 52-22 was greater than that of specimens containing Elvanol 71-30. The reason for this is not fully known, but may lie in the fact that Elvanol 71-30 is fully hydrolyzed (99.8%) whereas Elvanol 52-22 is partially hydrolyzed (87.5%). Elvanol 71-30 thus has greater potential for reaction with water to satisfy unsaturated bonding, thereby increasing strength. This hypothesis is further exemplified by definite decrease in density with increasing PVA content and is due to flocculation of soil particles. For concentrations below 0.25%, strength decrease may be due to lack of enough chemical to effectively bond within the soil matrix and merely decrease both density and strength.

The mechanism of Elvanol stabilization appears to be chemical rather than physical, and strength is more dependent on cure time than additive content. This is illustrated by the difference between 1- and 7-day cure curves of Elvanol 52-22 and other additives. In most cases the 7-day curve was reflected by the 1-day curve, indicating little dependence on cure time, although the rate of bonding, and bond strength, was related to cure time for Elvanol 52-22.

The relatively large decrease in dry density without loss in strength is due to the relative strength of the Elvanol-soil bonds and is another indication of chemical activity.

Central Chemicals Solutions containing Kelpak, SA-1, Claset, and Clapak were prepared at concentrations of 1, 10, and 20 parts per 1000 by volume with distilled water. When added to the soil at a moisture content of 10% these concentrations provided 0.01, 0.1, and 0.2% additive respectively, by dry soil weight. In keeping with manufacturer's data specifying that the chemicals be mixed "slightly above optimum moisture," 10% was chosen as the molding moisture content. Although considered "compaction aids," none of the compacted densities exceeded that of untreated specimens and may be due to the slightly increased molding moisture content. Results of unconfined compression tests are shown in Figs. IV-5-9.

Kelpak

In general, strength increased as additive content increased, though an optimum was apparent at 0.1% (Fig. IV-6). Density decreased to a minimum with initial addition of Kelpak; 128.4 for untreated specimens to 125.9 for 0.01%. Further additions of Kelpak increased density, reaching a maximum of 128.2 pcf at 0.2% additive.

Increased strength may be attributed to a chemical reaction rather than increased cohesion due to densification, since density was not increased above that of untreated specimens. Exactly what reaction occurred is unknown, but may be either (a) the formation of soil-Kalpak bonds or (b) enhancement of high strength water bonds formed by water evaporation. Further discussion is noted under the section on x-ray diffraction.

SA-1

Average unconfined compressive strengths are shown in Fig. IV-6. Strength reaches a maximum at 0.1%, with the optimum becoming more pronounced with increased curing. Density decreased slightly with increased SA-1 contents of 0 to 0.2%.

Strength again appears to have resulted from an unknown chemical reaction between the soil and SA-1. The decrease in strength at 0.2% concentration may be due to an overabundance of chemical activity, or possible deflocculation of the finer aggregations.

Claset

Average strength of specimens containing Claset are shown in Fig. IV-7. Generally, the strength decreased with increasing amounts of additive though a very slight optimum was apparent at 0.01%, 7-day cure. Density was somewhat erratic, decreasing about 2 pcf with the initial concentration of Claset, then bouncing slightly up and down about 1 pcf at the two additional contents. Reduced strength may be due to reduction in densification, absence of an adequate chemical reaction, or interference with formation of soil-water bonds during evaporation.

Clapak

Strength of specimens containing Clapak are shown in Fig. IV-8. Generally, strength characteristics improved as cure time increased, being less than untreated strength for 1-day cure, but slightly greater for 7 days. Optimum strength appeared at 0.1% Clapak, however it was of much less magnitude than strengths at optimums for Kelpak or SA-1 contents. Density decreased with initial addition of Clapak, but remained relatively constant with further additions of the product. Slight strength gains may be due to a chemical reaction which takes place slowly as water evaporates, or may result from enhancement of high strength water bonds formed upon evaporation of free water.

Claset and Clapak Combination

Strength of specimens containing a combination of Claset and Clapak is shown in Fig. IV-9. Combination of additives was recommended by the manufacturer at the following proportion, based on plasticity index of the soil. The plasticity index of the treated soil was 8.6.

Plasticity index	Claset[a]	Clapak[a]
10	5	10
20	7.5	15
30	10	20
40	15	30
50	20	40

[a] Parts per 1000, by volume.

Therefore the concentrations specified for PI = 10 were used, 0.1% Clapak and 0.05% Claset.

As noted in Fig. IV-9, no increase in strength was realized by the combination. Strength and density were both less than that achieved with untreated specimens, and judging from the strength results of Clapak and Claset alone, any benefit of Clapak was lost, when Claset was added.

Summary of Results

1. Except for Kelpak and Claset, each additive decreased soil strength when small amounts were mixed into the soil.

2. Dry density decreased as additive content increased. In no case, did dry density exceed that of untreated specimens.

3. Stypol 40-5020 and Petro D Dust behaved as asphaltic products insofar as there appears to be an optimizing effect of concentration, with the resulting stabilization as physical rather than chemical in activity.

4. Elvanol 52-22 provides greater strength at higher concentrations than Elvanol 71-30. There is most probably a flocculation between soil and Elvanol to form high strength, low density bonds.

5. Elvanol polyvinyl alcohols and the central chemicals appear to be chemical in nature.

6. Kelpak, Clapak, and SA-1 were optimized at 0.1% additive content whereas Claset may have optimized at 0.01%. These optimums increased strength to varying degrees, but decreased density compared to untreated specimens. The resulting strength appeared to be chemical in nature.

EROSIBILITY

Previous studies have used the unconfined compressive strength of an immersed specimen as a means of evaluating the durability of stabilized soils. Quantitative results are obtained only so long as the specimen does not fail upon immersion or become too soft to test. Otherwise the test results become merely qualitative.

If utilized as a surface course, the stabilized soil would be more susceptible to erosion due to rain, followed by traffic abrasion, than to complete immersion and saturation as in the unconfined immersed strength test. Since (a) the previous section evaluated the strength of specimens from near saturation (0-day air cure) up to 7 days of air drying, and (b) a form of abrasion was evaluated during the traffic simulator studies later in this report, a quicker screening test was needed.

The "Rainmaker" (Fig. IV-10) is composed of distilled water and compressed air supplies, a spray nozzle, and a receiver containing a specimen rack and drain line. A specimen is placed on the rack and subjected to a spray of distilled water. The principle is analogous to a moderate to heavy rainfall impacting on soil. Sediment which is eroded from the specimen is trapped in the bottom of the receiver, dried, weighed, and, knowing the original weight of specimen, a fraction-eroded is calculated. The eroded fraction, hereafter known as the Erosibility Index, or E.I., is recorded in grams per 1000 grams. Thus a quantitative measure of the surface durability, i.e. erosibility, can be determined for different soil stabilizers.

Sample Preparation, Curing, and Testing

Specimens were prepared in the same manner as previously noted for the unconfined compression test. Following molding, the samples were allowed to air cure at room temperature for 24 hours, height and weight

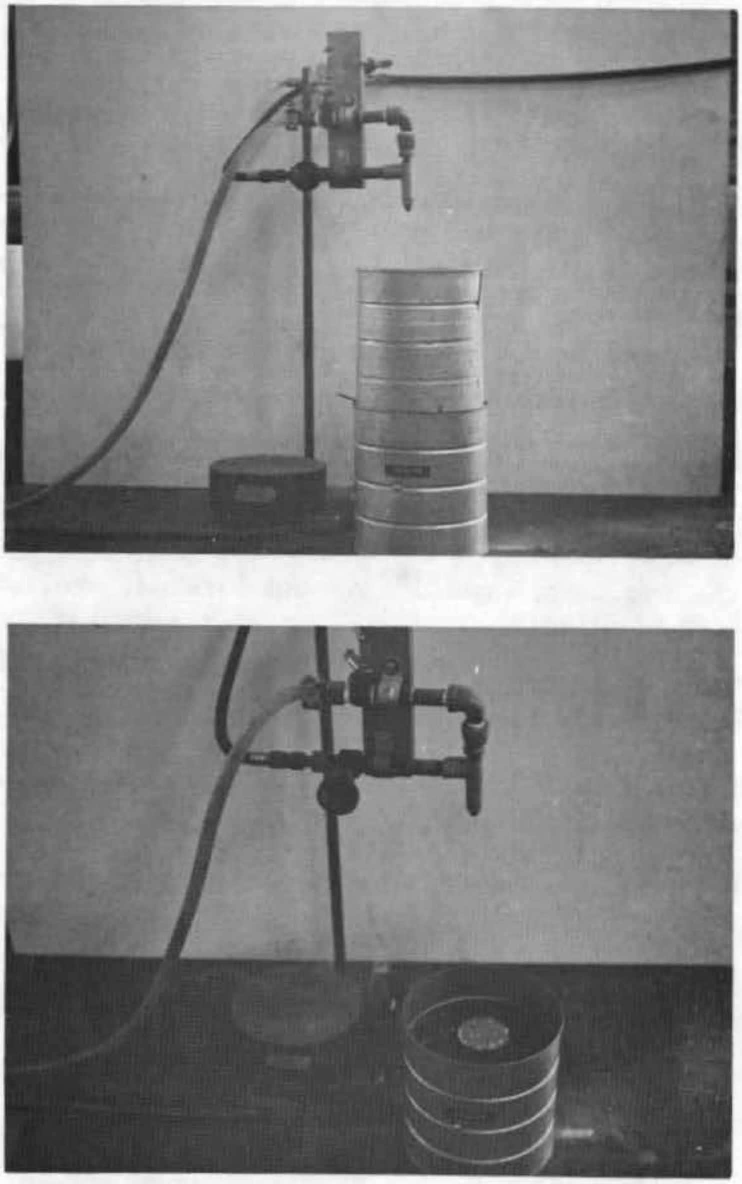


Fig. IV-10. "Rainmaker" used to determine erosibility index.

having been recorded. Twenty-four hours was selected as optimum practical cure period for evaluating erosibility after compaction, as based on curing relationships noted with the compressive strength tests.

Method of Test

Specimens were placed on the rack of the receiver, weight and height measurements again having been recorded, and subjected to a spray of distilled water for 15 minutes. Intensity of the simulated rainfall spray was approximately 0.82 in. per hour, or equivalent to a moderate to heavy rainfall. Qualitative visual observations were made throughout the duration of the test and also recorded. At the end of the test, the rack and specimen were removed, and the sediment was rinsed from the receiver, oven dried, and weighed. Following drying, the sediment was allowed to equilibrate with room conditions before weighing thereby giving credibility to the proportion of grams (air dry) per 1000 grams (air dry). The results are reported in grams per 1000 grams rather than a percentage.

Test Results

Results of erosibility tests are shown in Figs. IV-11-18. The Erosibility Index generally decreased when very small amounts of additive were introduced into the soil. The average Erosibility Index of untreated specimens was 268. With small additive contents, Erosibility Indices ranged from 128 to 0.75.

Stypol 40-5020 Results of erosibility tests on specimens molded with Stypol are shown in Fig. IV-11. With the addition of 0.25%, the E.I. decreased from 268 to 128, a substantial reduction. A concentration of 0.50% reduced the E.I. to 24 where it tended to remain even with further increases in concentration.

Because of its resinous nature, Stypol behaved much like asphalt, and in a sense waterproofed the soil rather than chemically bonding soil

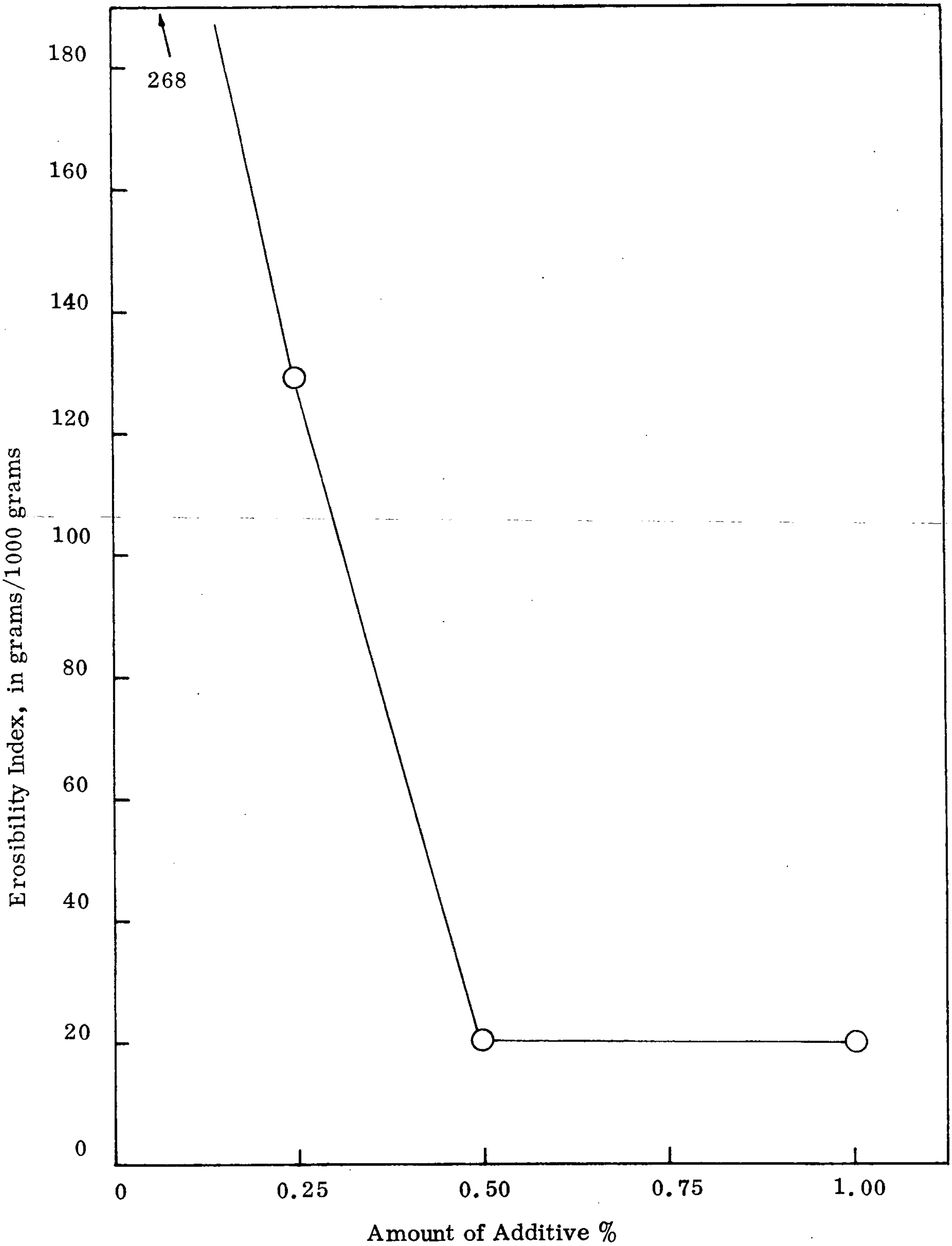


Fig. IV-11. Erosibility of Stypol 40-5020.

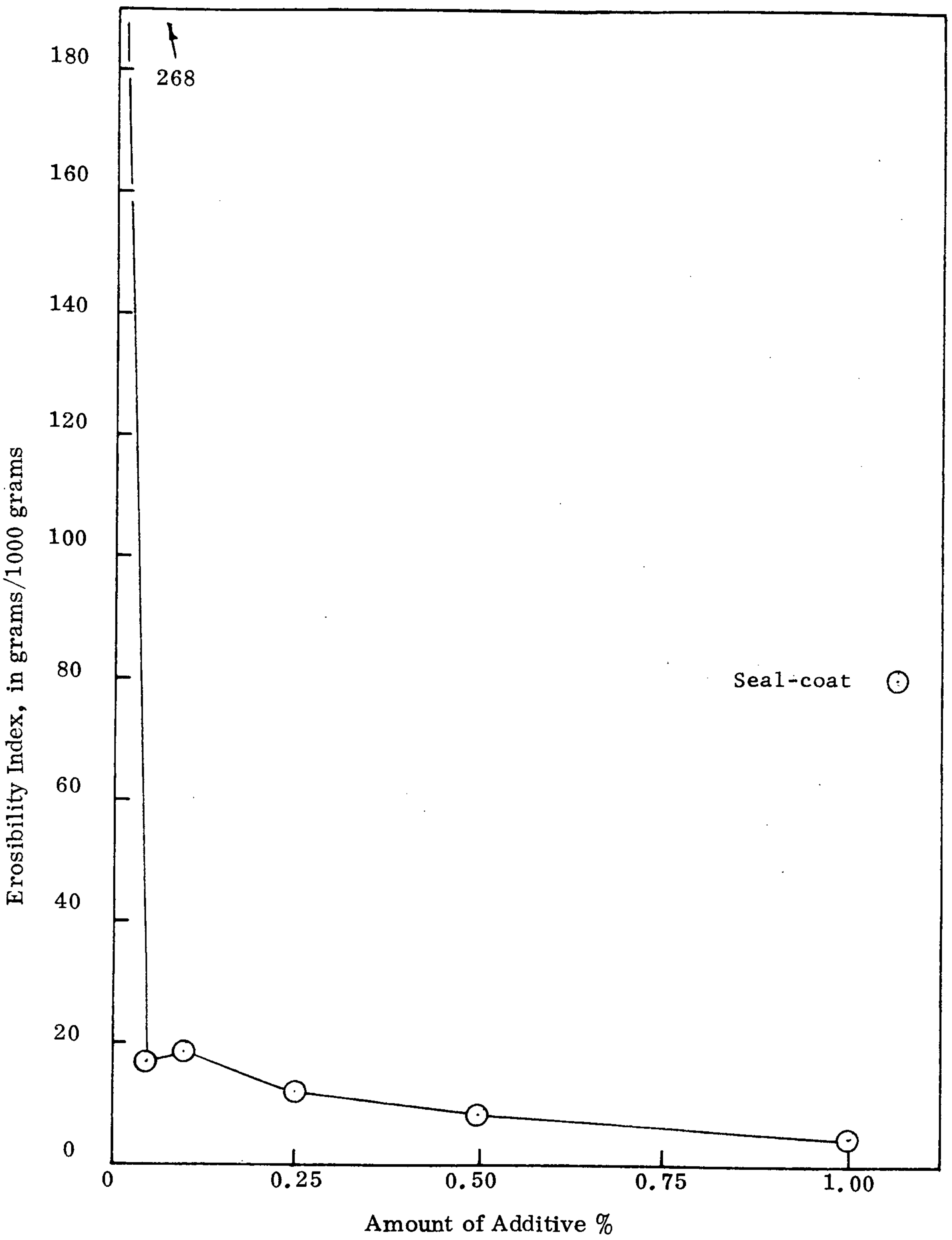


Fig. IV-12. Erosibility of Petro D. Dust.

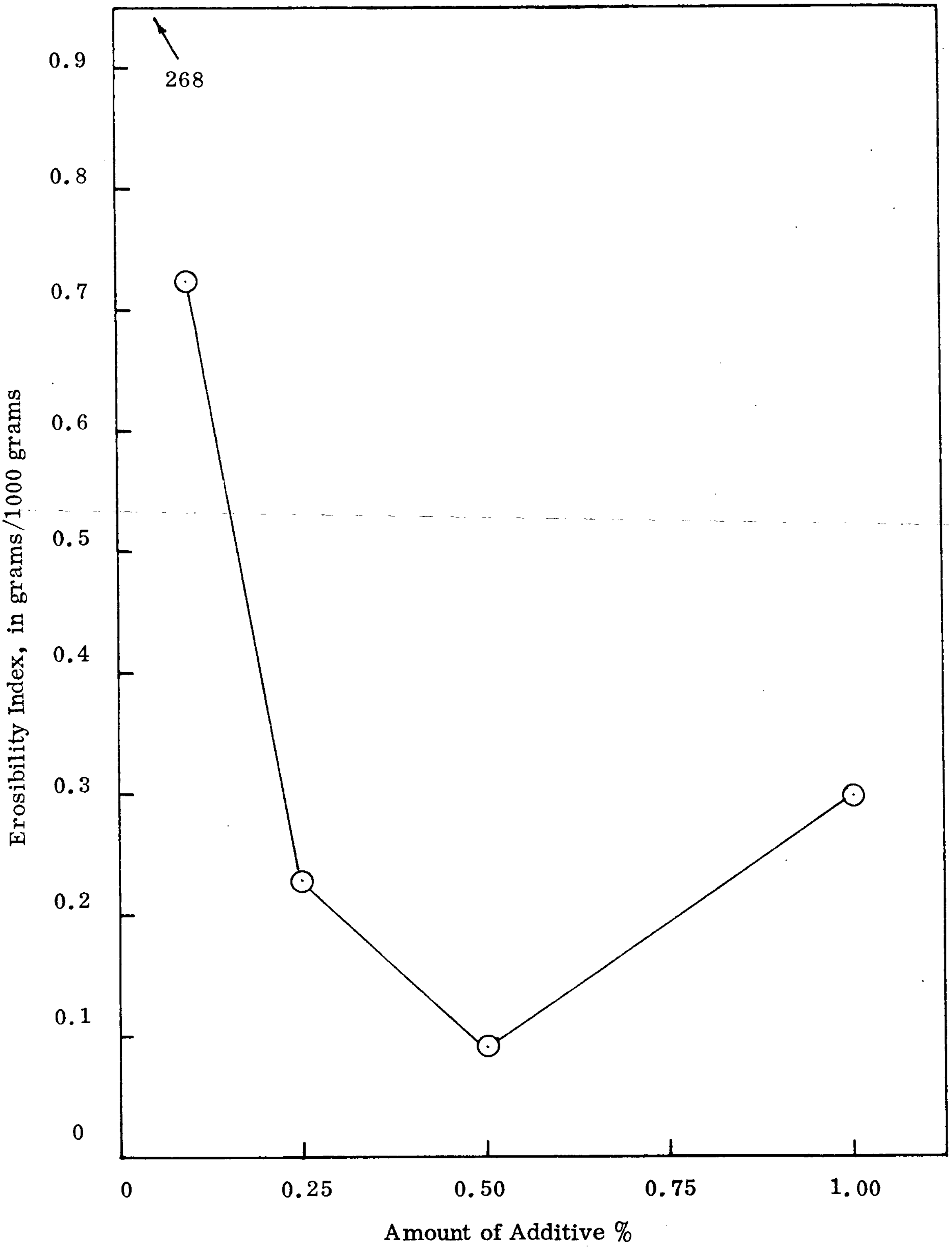


Fig. IV-13. Erosibility of Elvanol 71-30.

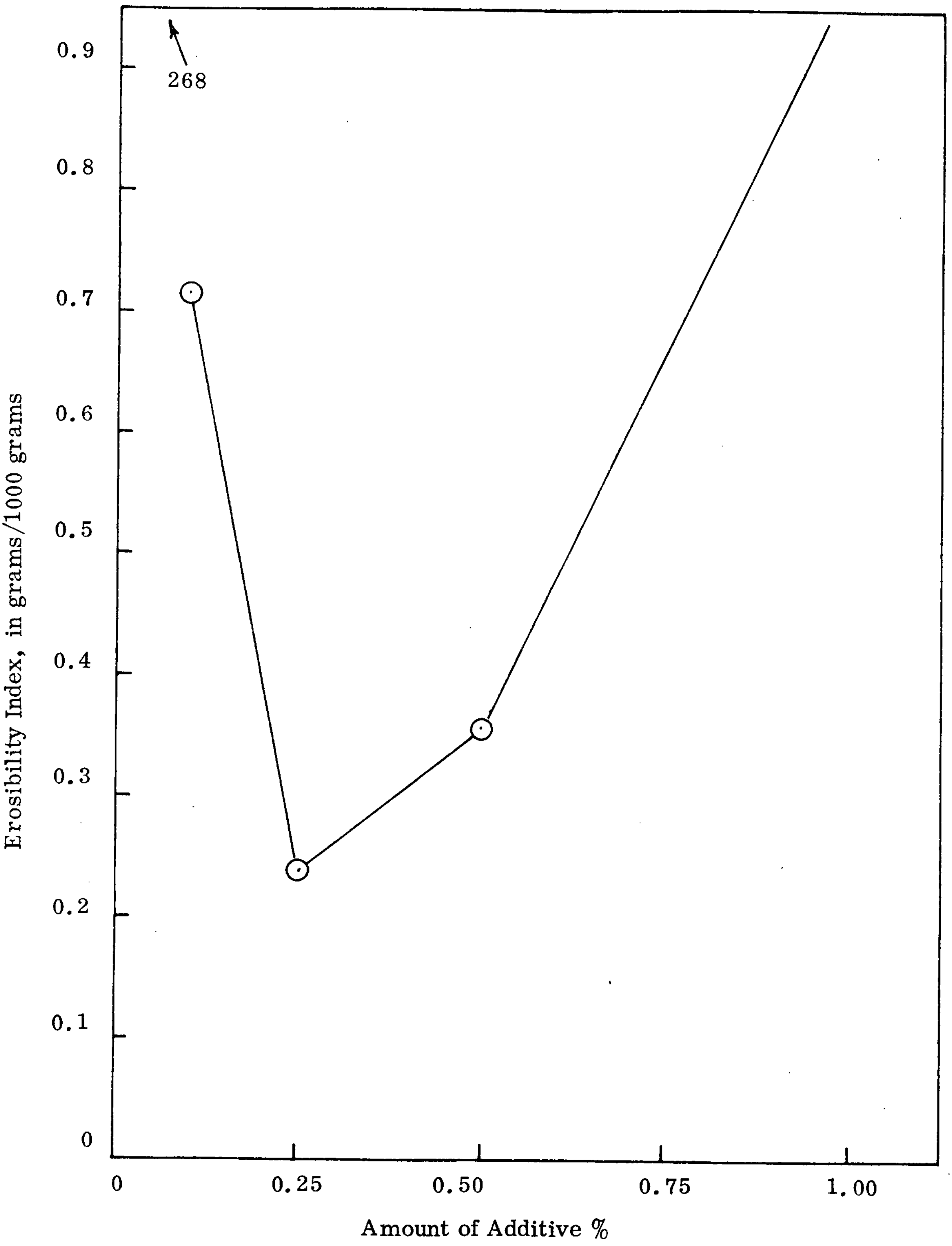


Fig. IV-14. Erosibility of Elvanol 52-22.

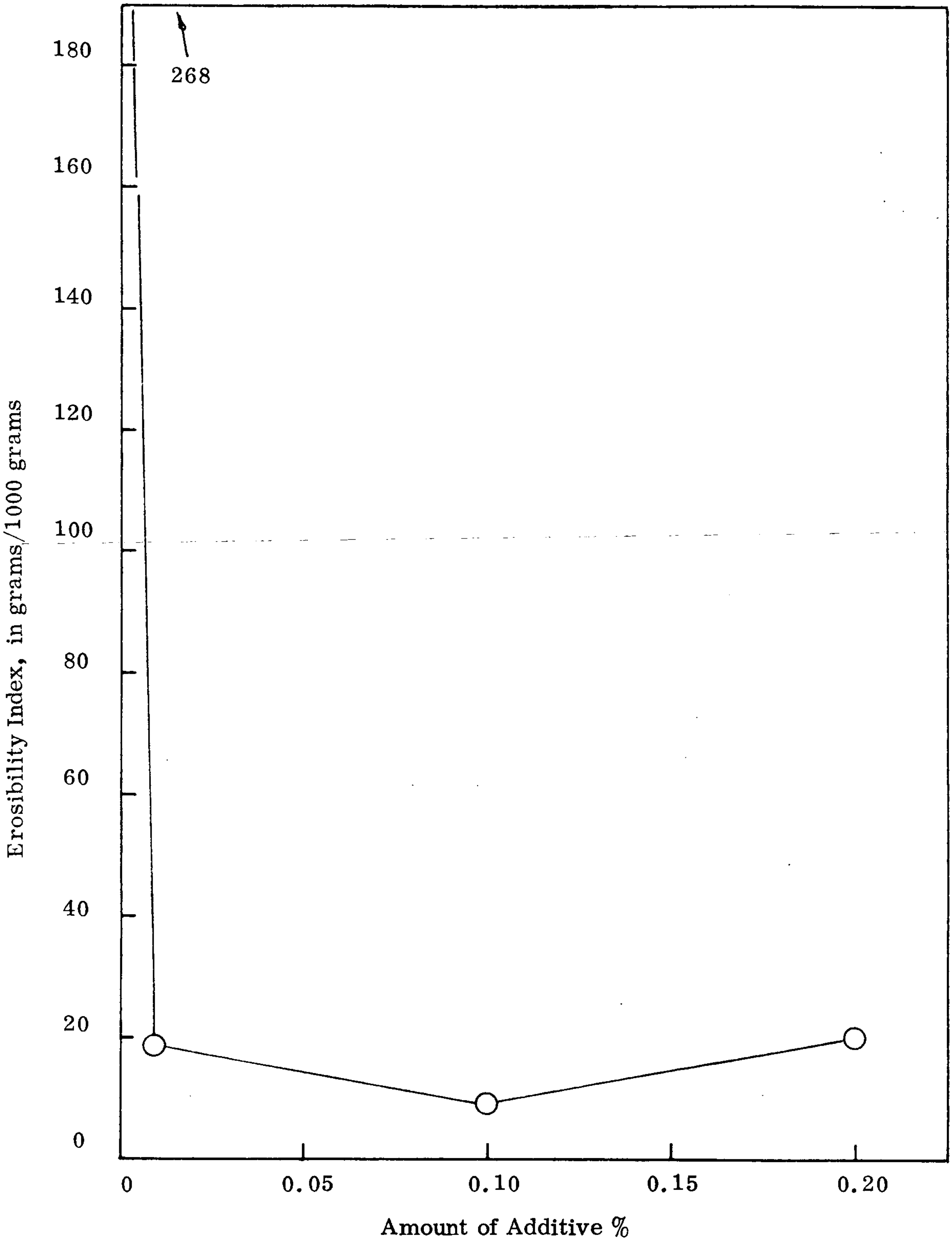


Fig. IV-15. Erosibility of Kelpak.

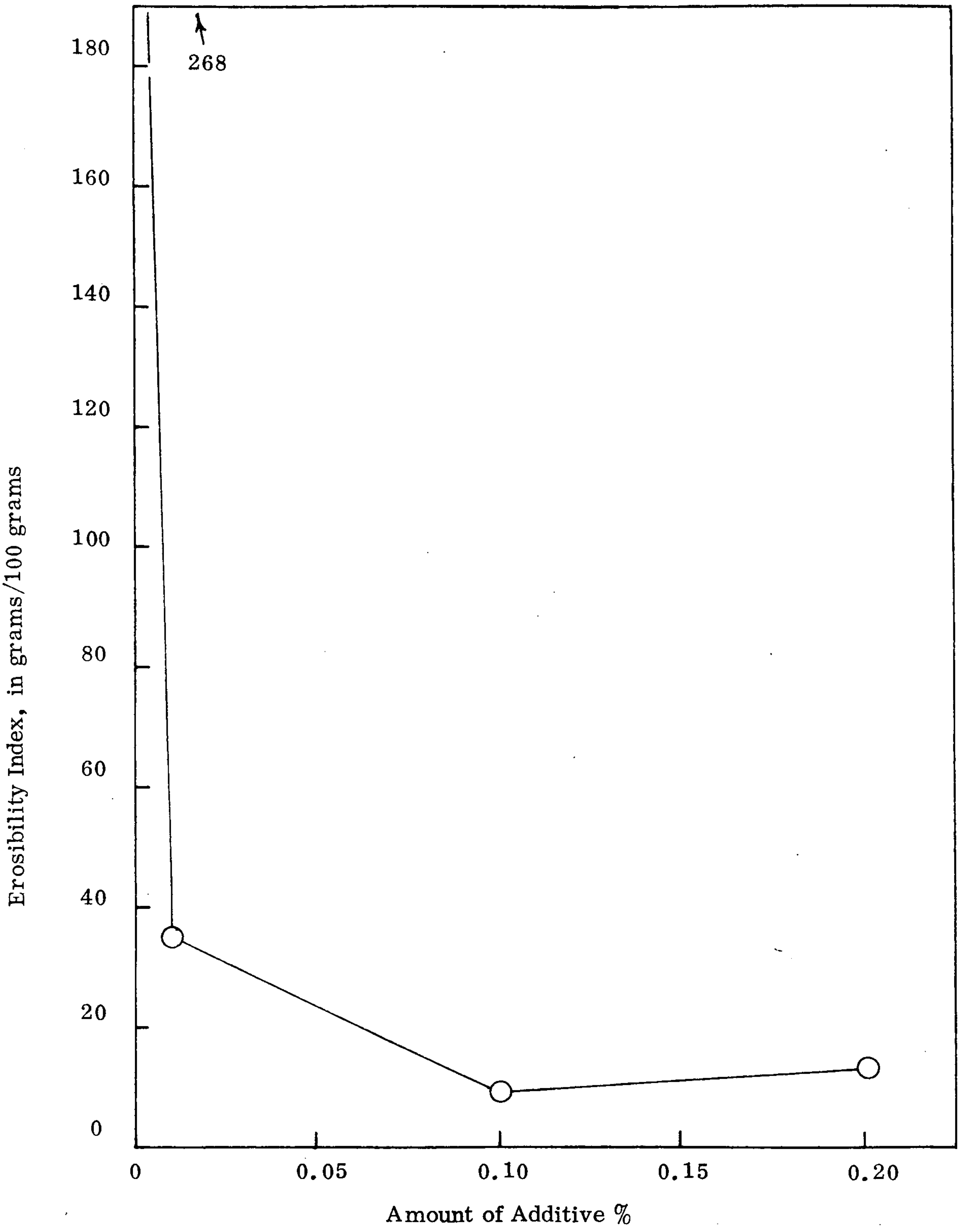


Fig. IV-16. Erosibility of SA-1.

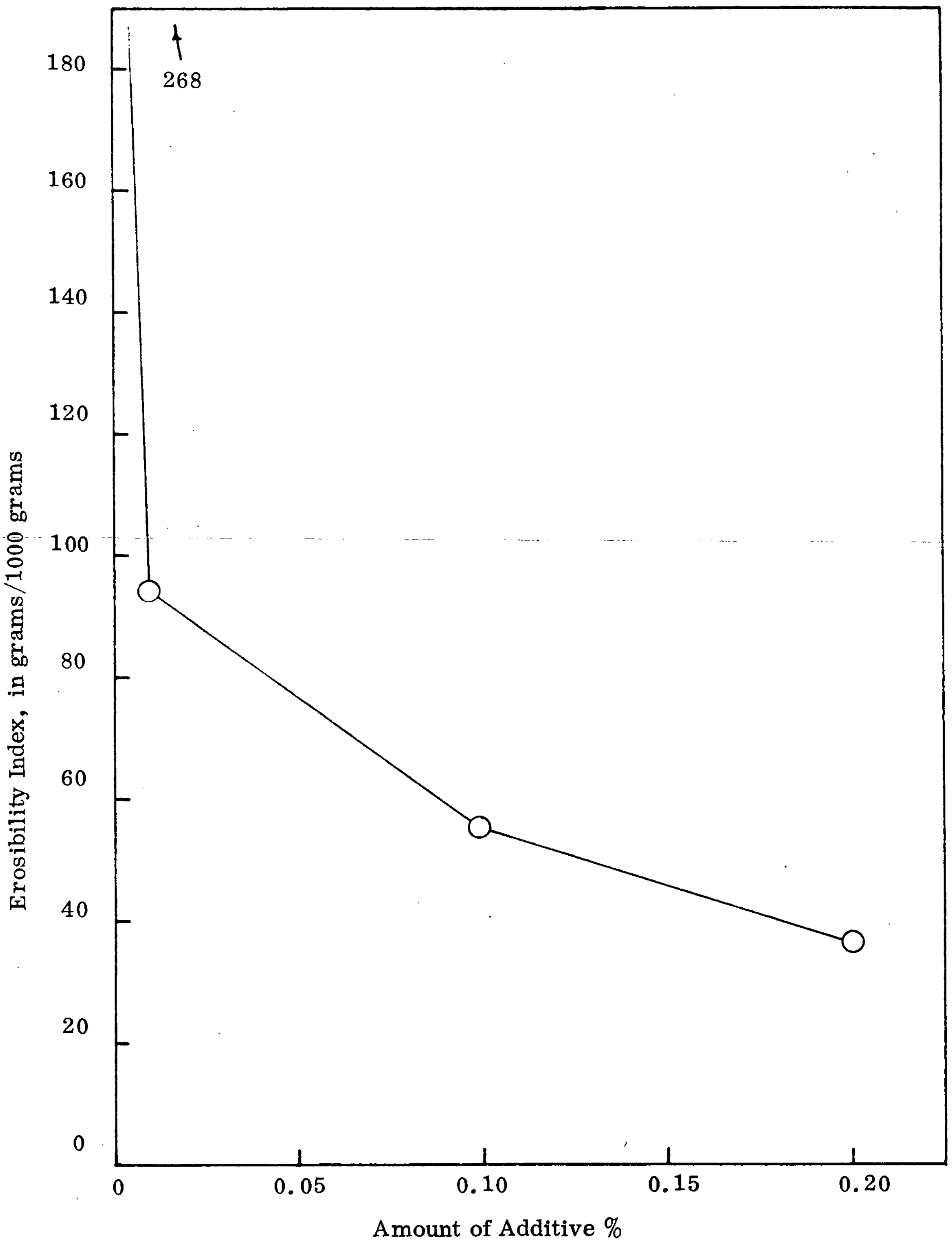


Fig. IV-17. Erosibility of Clapak.

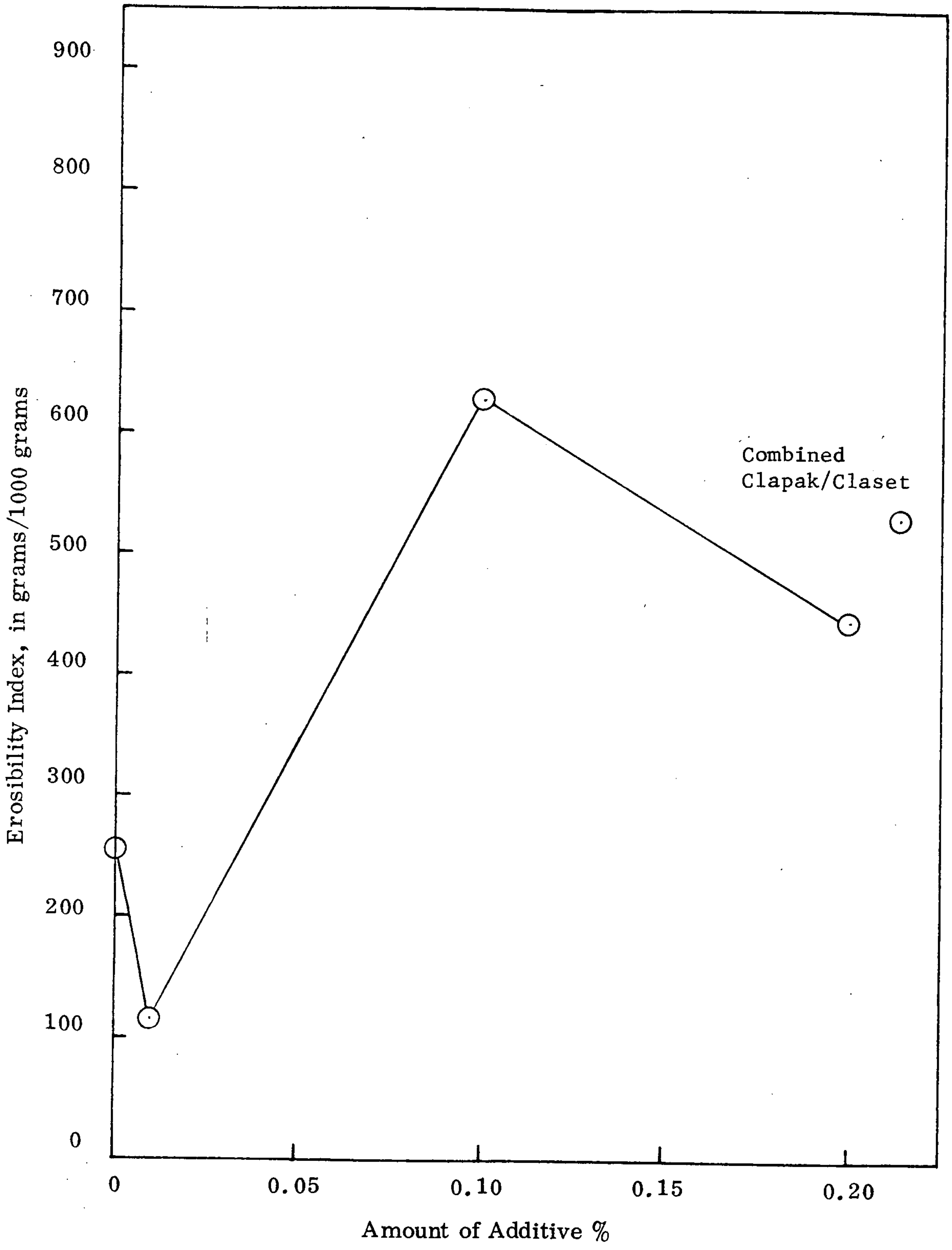


Fig. IV-18. Erosibility of Claset and combined Clapak/Claset.

particles together. An optimum waterproofing content was achieved at 0.50% and the E.I. properties were much improved over the untreated specimens.

Most of the material eroded from treated specimens seemed to spall from the surface indicating that perhaps the Stypol had filled the void spaces between the grains and acted as a waterproofer much the same as asphaltic products.

Petro D Dust   Results of erosibility tests performed on specimens containing Petro D Dust are shown in Fig. IV-12. The E.I. reduced from 268 to 18 with the addition of 0.05% additive, from which the index further decreased as the quantity of Petro D Dust was increased, but at a decreasing rate, much lower than for the initial decrease. There was a slight increase in E.I. at 0.10% for which there is no explanation but is not considered critical, still being much less than the untreated.

Petro D Dust, although not derived from petroleum distillates (according to manufacturers' data), behaves in much the same manner as many asphaltic products. This product also acts much like an oil, possibly a vegetable oil, and is thought to be an unrefined cotton seed oil. An infrared spectrophotometer analysis was performed to determine the product's makeup, but was inconclusive. Petro D Dust appears to impart a waterproofing characteristic to the soil in the form of filling void spaces rather than chemically reacting with the soil.

Petro D Dust was also analyzed as a spray-on seal coat. When applied at the rate of 0.20-0.25 gal./sq yd, it provided very good surface waterproofing characteristics but imparted no internal waterproofing or stability due to only an insignificant amount of surface penetration.

As long as the seal remained intact, water was impermeable to the specimen. Results of the surface seal are also shown in Fig. IV-12.

Elvanol 71-30 and 52-22 Erosibility test results on samples containing Elvanol 71-30 and Elvanol 52-22 are shown in Figs. IV-13 and IV-14, respectively.

Small amounts of both grades reduced the E.I. to almost zero, from 268 grams/1000 grams to 0.75 grams/1000 grams. Essentially Elvanol prevented erosion from occurring. An optimum E.I. fell between 0.25% and 0.50% for both grades. Both Elvanol 71-30 and Elvanol 52-22 substantially reduced erosion due to polymer bonding between the soil grains. Void spaces were probably not plugged because of the presence of water in the soil matrix, which was confirmed following testing. Elvanol forms a strong, water insoluble bond with only a very small quantity of additive introduced into the soil.

Kelpak Results of tests performed with Kelpak treated specimens are shown in Fig. IV-15. With very slight amounts of Kelpak, the E.I. was reduced to 19 grams per 1000 grams. A further but very slight reduction was achieved with increasing amounts of additive content and an optimum was apparent at 0.1%. Kelpak appears to form soil-chemical bonds within the soil matrix, rather than waterproof the soil by the filling of void spaces.

SA-1 Erosibility results on specimens containing SA-1 are shown in Fig. IV-16. SA-1 behaves similarly to Kelpak, reducing the E.I. from 268 for untreated specimens to 35 with the addition of 0.01% SA-1. An optimum was observed at 0.1% additive content which further reduced the E.I. to 9. Observations indicated that stability was due to a chemical reaction rather than merely a physical waterproofing of the specimens.

Clapak Figure IV-17 presents the results of specimens prepared using Clapak. Like Kalpak and SA-1, low concentrations of Clapak reduced the E.I. from 268 to 95. The Erosibility Index decreased to 56 as additive content increased to 0.2%. Reduction of E.I. is probably due to chemical rather than physical reactions.

Claset Figure IV-18 shows the results of the erosibility test on specimens containing Claset. E.I. was reduced by about 1/2 with the addition of 0.01% Claset. However with increased concentrations, the E.I. increased abruptly, peaking at 0.1% (632) and then falling slightly at 0.2% (445).

The reason for the increase may be due to (a) no chemical reaction or (b) interference in formation of high strength water bonds during curing. Although not readily apparent, Claset may have an attraction for water and may hold more free water in the soil matrix during curing, thus hampering development of the water bonds, and thus reducing strength and stability.

Clapak and Claset in Combination Specimens were prepared containing 0.1% Clapak and 0.05% Claset according to manufacturers' recommendations. The erosibility test results are shown in Fig. IV-18. Erosibility Index was increased from 268 to 562 indicating a loss of stability with the combination.

Claset was judged to be the main factor contributing to the instability of the specimens due to the erosion results of Clapak and Claset tested individually. The reason for the increase in E.I. may be due to (a) absence of chemical bonds, (b) interference of Claset with formation of soil-Clapak bonds (which were evident with Clapak alone) or, (c) interference with high strength water bonds formed during curing.

Summary of Results

1. Erosion was generally inhibited with the introduction of a very slight amount of chemical into the soil.

2. Only a slight further reduction was realized with increasing amounts of additive.

3. Specimens containing Stypol and Petro D Dust retained stability due predominantly to waterproofing of the void spaces rather than any chemical reaction.

4. A surface treatment of Petro D Dust amounting to 0.2 to 0.25 gal./sq yd sealed the surface but penetrated the surface only an insignificant amount.

5. The erosibility of specimens containing Elvanol 71-30 and 52-22 was reduced to almost zero. These specimens derive their stability solely from soil-chemical bonds and not from physical waterproofing of void spaces. Water was observed in the interior voids after testing.

6. Specimens containing Kelpak derived their stability from a soil-chemical reaction and subsequent bonding.

7. Specimens containing SA-1 were stabilized by soil-chemical bonding rather than physical waterproofing.

8. Stability of specimens containing Clapak is due to chemical reaction with the soil. The Erosibility Index decreased as the concentration increased.

9. Specimens containing Claset slightly decreased the Erosibility Index at very low contents, but increased E.I. to over 2 times the untreated specimens with increasing concentrations.

10. Instability of specimens containing Claset may be due to (a) lack of any soil-chemical bonding or (b) interference of formation of high strength water bonds.

11. Manufacturers recommended combination of Claset and Clapak was totally ineffective.

Traffic Simulation Tests

The purpose of this test was to evaluate the performance of treated and untreated specimens under a simulated wheel load equivalent to that of a moderate to heavy truck tire contact pressure. In addition, variable climatic conditions were introduced to test treated specimen resistance to rutting wet, as well as dry.

Traffic simulator results are thought to provide valid indications of both fine material retention and waterproofing. In addition, the test is indicative of a materials stability under moving load and imposed environmental conditions.

A traffic simulator apparatus, originally developed by the Bituminous Research Laboratory and slightly modified by the Soil Research Laboratory and slightly modified by the Soil Research Laboratory, both at Iowa State University, is fully described and shown in Figs. II-13 and II-14, Part II Final Report.

Specimen Preparation and Curing

All specimens were molded in holding rings capable of being installed directly in the retainer box without extrusion. Four-in. diameter by 2.4-in.-high specimens were compacted to the same densities and moisture contents as in the unconfined compression and erosibility tests. The holding rings were securied to a modified Proctor molding stand during compaction and a 1/4-in. plate placed in the bottom of the mold (for adjustment after installation). Compaction was accomplished in two layers, each being rodded 25 times with a 3/4-in. round tipped rod and subjected to 20 blows of a standard 5.5-lb Proctor hammer. Finally a 1/4-in. plate was inserted in the top of the mold and an additional five blows were applied to the center to provide a smooth surface. The upper plate was removed and the specimen allowed to air cure for 24 hours following initial height and weight measurements. Two identical specimens containing the various additive concentrations were molded and tested; results are reported as the average thereof.

Method of Testing

The method of testing used in this study is fully described in Part II Final Report. Using the applied contact pressure of 85 psi the carriage and loading wheel make:

1. One thousand passes with no imposed environmental conditions.
2. One thousand passes with simulated rain of 0.15-0.20 in./hr.
3. A two-hour fogging period with no traffic, fogging adjusted so that water was visibly evident on all specimen surfaces at all times.
4. One thousand passes with simulated rain of 0.15-0.20 in./hr.

Rut depth measurements were made following each interval of 250 passes, though in most tests, an initial measurement was made after only 50 passes. Failure of specimens was judged to occur when rut depth reached 0.5 in.

Test Results

Traffic simulator tests for Stypol 40-5020, Petro D Dust, and Elvanol 71-30 and 52-22 are shown in Figs. IV-19-25. Trafficability tests have not been conducted using Central Chemicals at time of this report

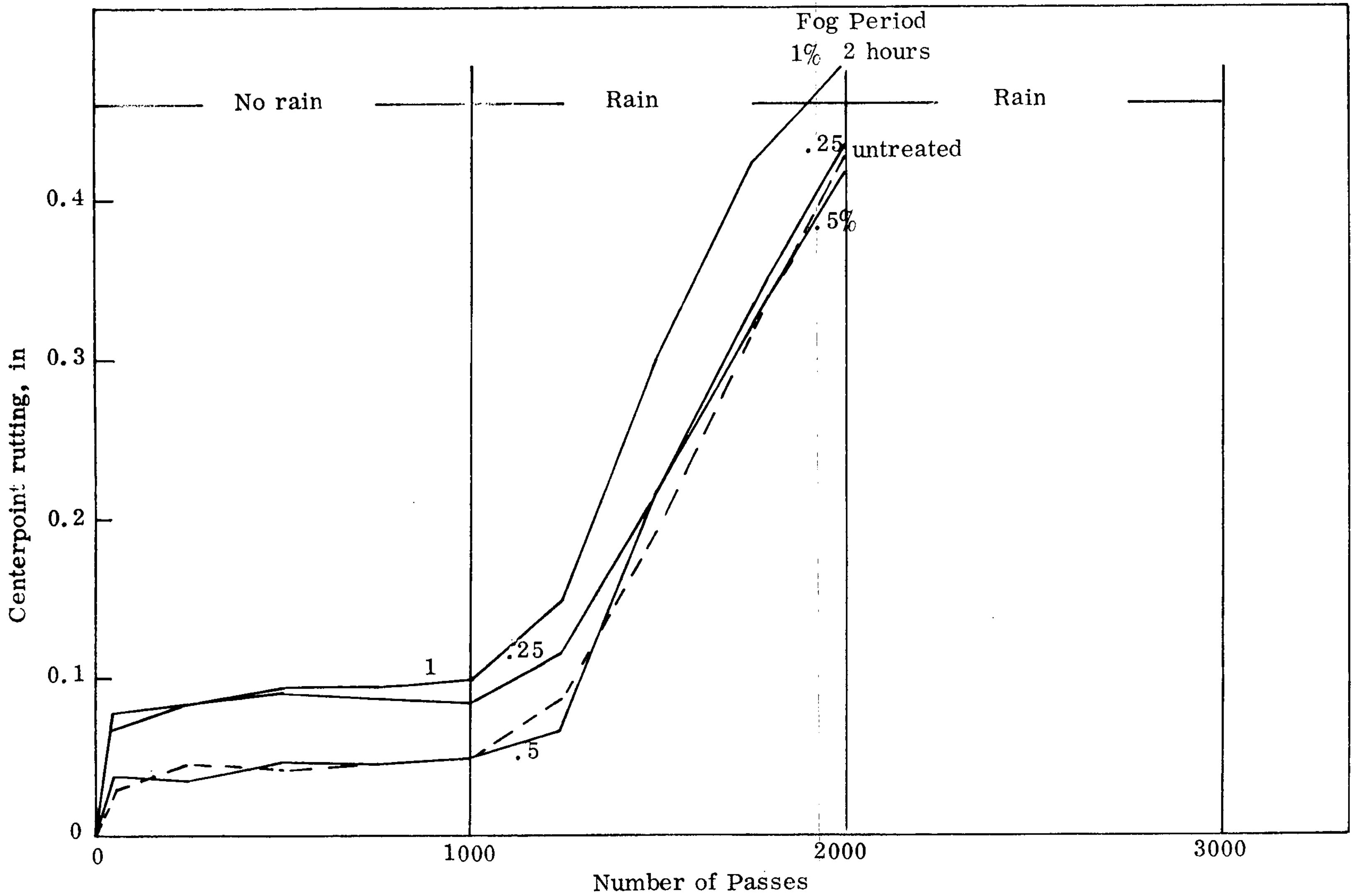


Fig. IV-19. Traffic simulator results — Stypol.

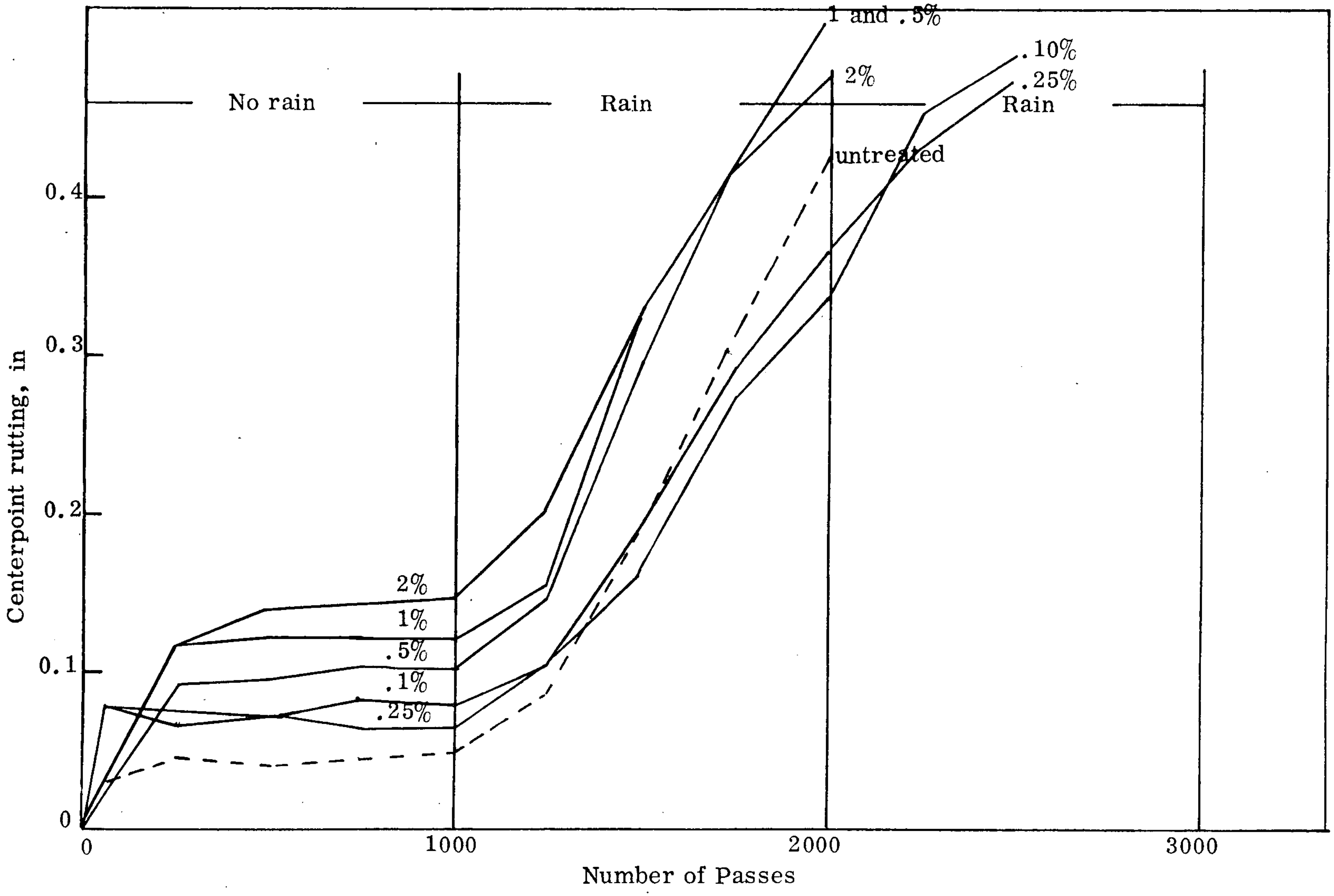


Fig. IV-20. Traffic simulator results — Petro D Dust.

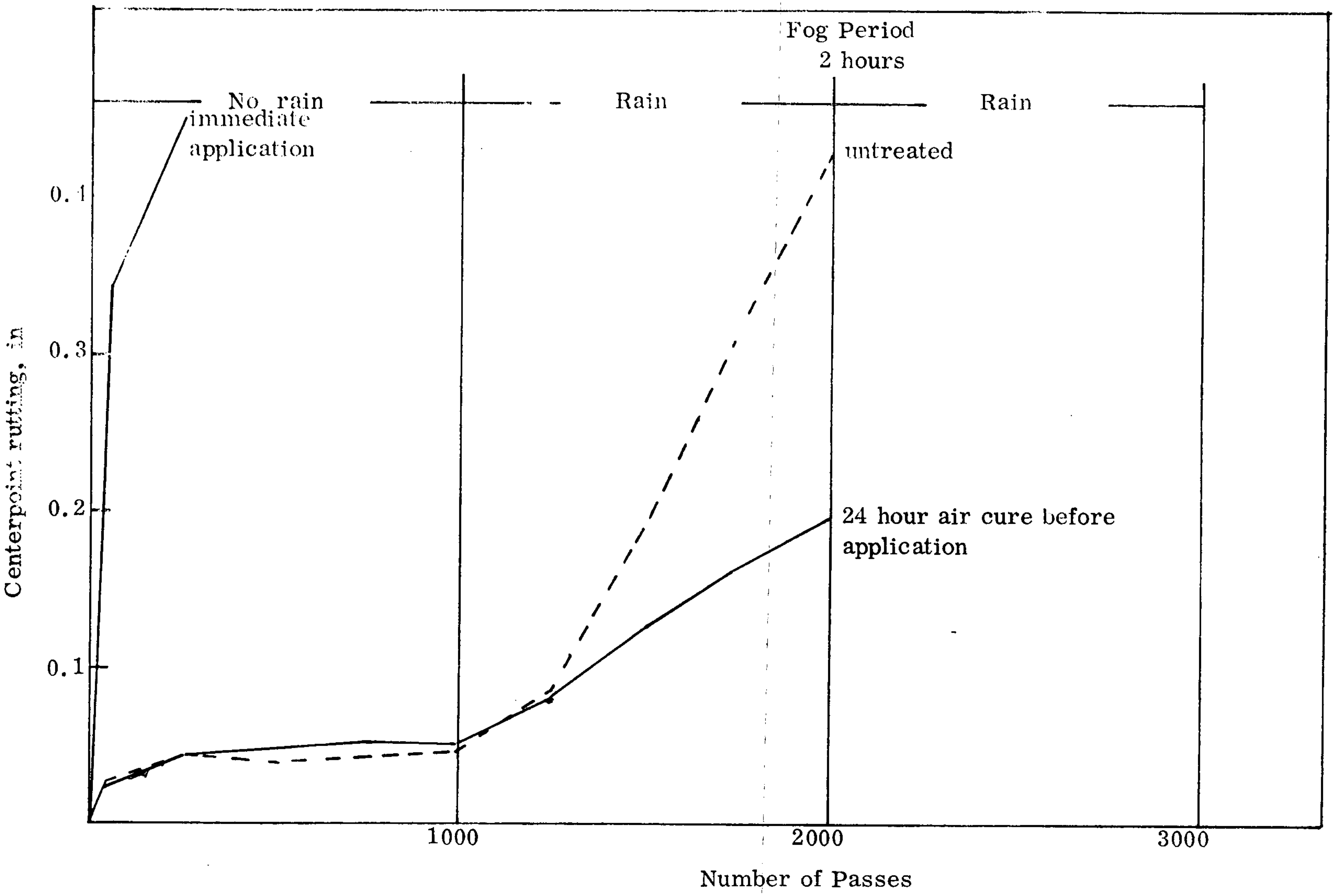


Fig. IV-21. Traffic simulator results — Petro D Dust seal coat.

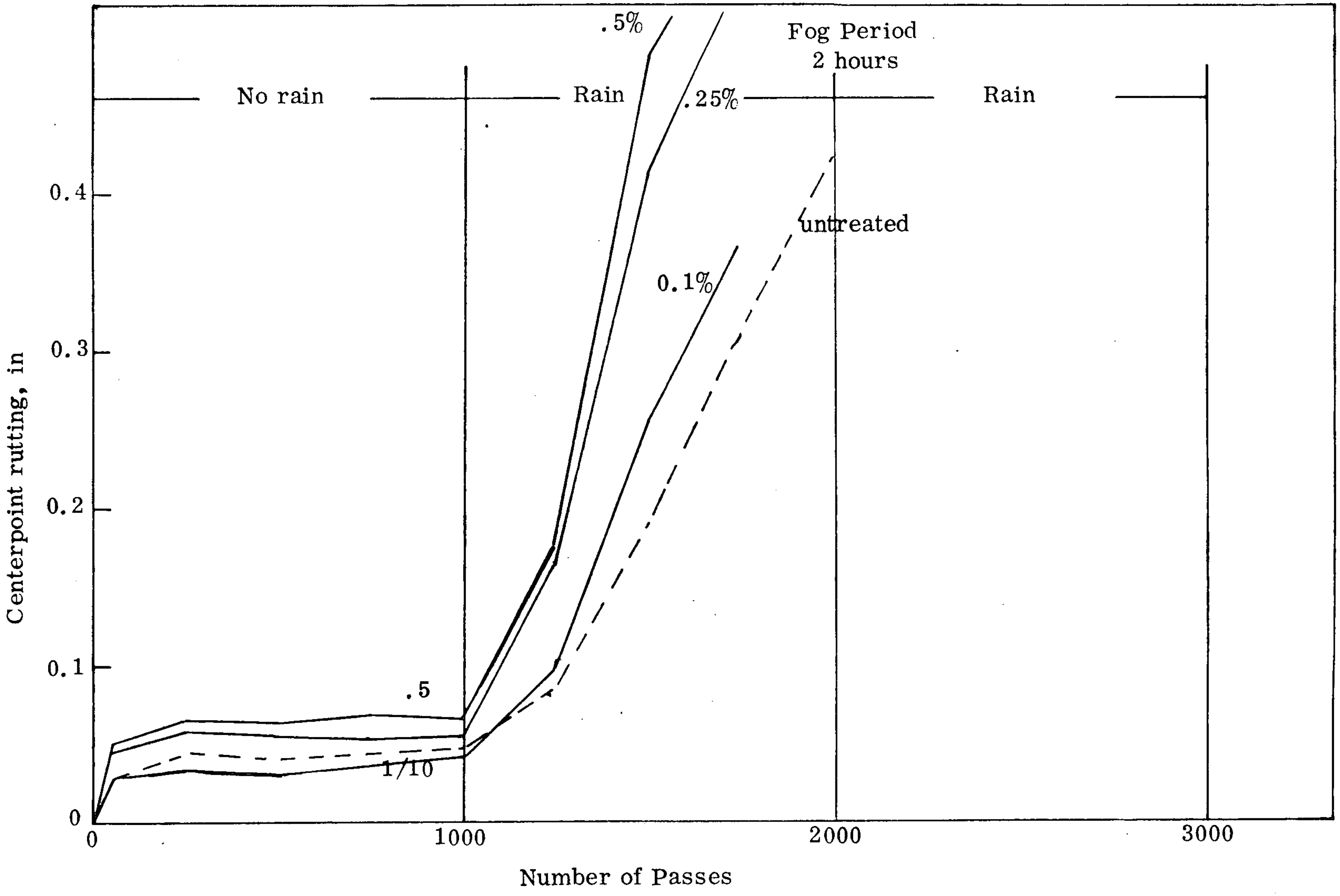


Fig. IV-22. Traffic simulator results — PVA, Elvanol 71-30.

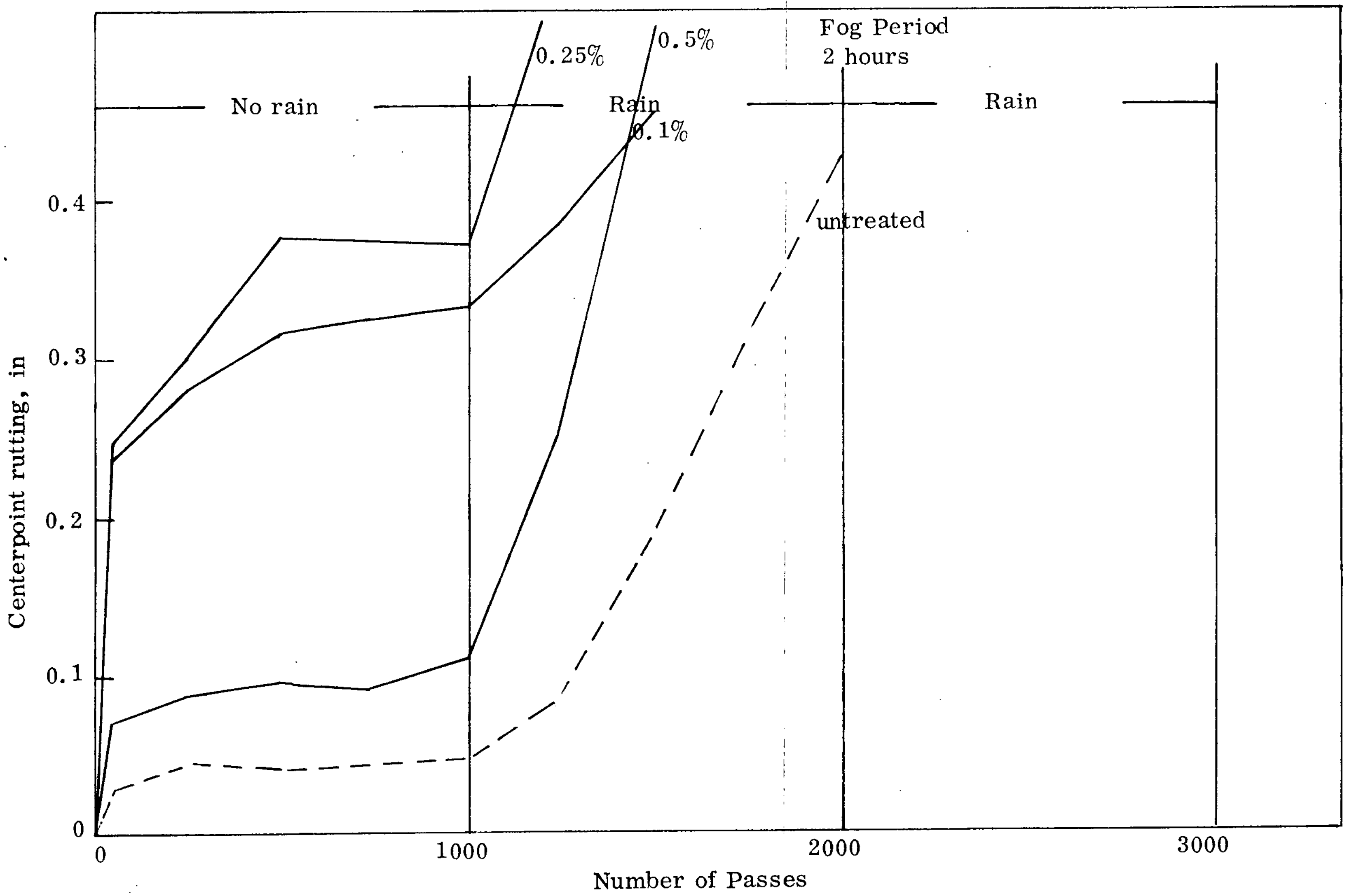
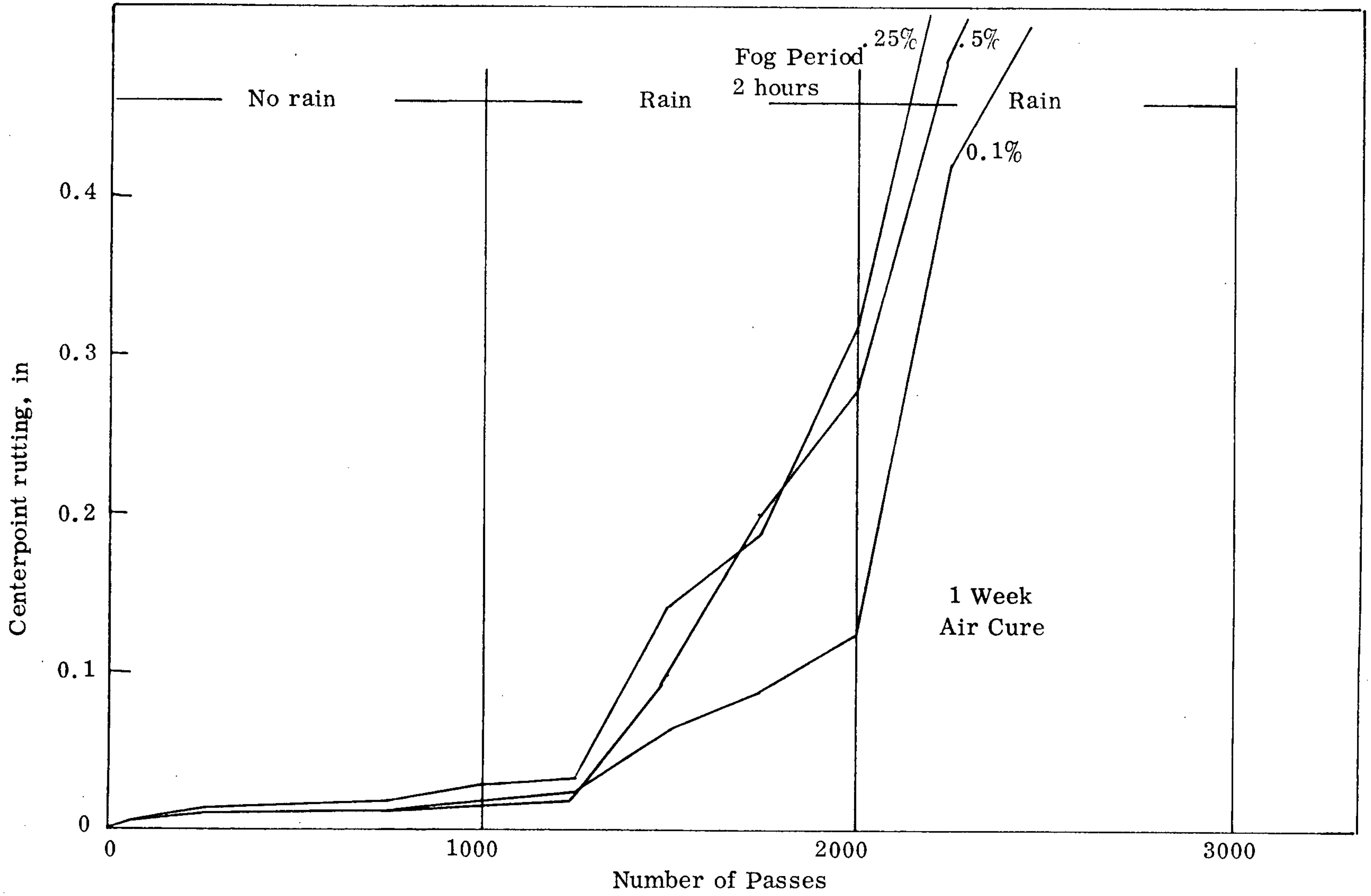


Fig. IV-23. Traffic simulator results — PVA, Elvanol 52-22.



. IV-24. Traffic simulator results – PVA, Elvan ,1-30.

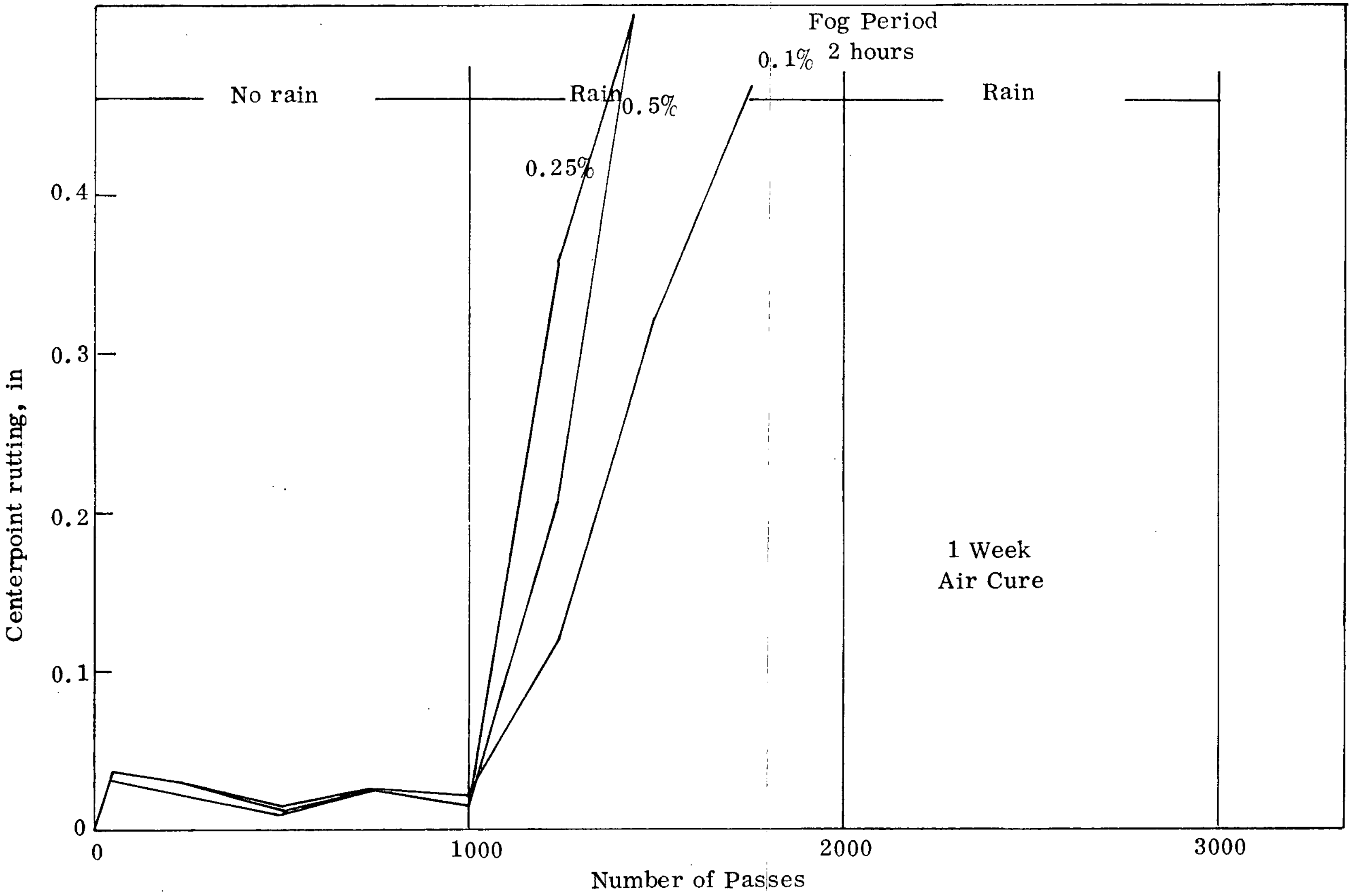


Fig. IV-25. Traffic simulator results — PVA, Elvanol 52-22.

preparation. Generally, during the first 1000 passes, treated specimens exhibited somewhat higher rut depth than untreated specimens. Rutting occurred early in the tests, usually within the first 50 passes and increased very little throughout the remainder of the first 1000 passes. As noted by Bergeson and Hoover, Part II Final Report, this effect is probably due to seating of the specimens in the holding rings and densification of the specimen in the wheelpath.

Table IV-7 shows the initial density variations of traffic simulator specimens prior to testing. In general these specimens exhibited the same trends as the unconfined and rainmaker samples, having decreased density with increasing additive content.

Table IV-7. Density variations of traffic simulator specimens.

Product	Additive content % dry soil weight	Approx. liquid content % dry soil stability	γ_w pcf	γ_d pcf
Untreated	9.0	9.0	138.8	126.8
Petro D Dust	2.00	9.0	136.8	125.5
	1.00	9.0	137.8	126.4
	0.50	9.0	137.6	126.2
	0.25	9.0	138.3	126.6
	0.10	9.0	138.8	126.8
Elvanol 71-30	0.50	9.0	131.36	120.5
	0.25	9.0	135.8	124.6
	0.10	9.0	136.9	125.6
Elvanol 52-22	0.50	9.0	134.7	123.6
	0.25	9.0	134.2	123.1
	0.10	9.0	137.1	125.8
Stypol 40-5020	1.00	9.0	137.9	126.7
	0.50	9.0	138.8	127.2
	0.25	9.0	138.4	126.9

Stypol 40-5020 Figure IV-19 shows results of trafficability tests on specimens containing Stypol. During the first 1000 passes, rut depth remained fairly constant after an initial 50-pass deflection.

Rut depth of treated specimens was somewhat greater than the untreated for the first 1000 passes with the exception of 0.5% Stypol which was about the same as the untreated. This trend continued through the second 1000 passes with simulated rainfall, although rut depth increased sharply with the addition of surface water. Optimum beneficiation of Stypol concentrations thus appeared at 0.5%. Rutting depth thus correlated very closely with the unconfined compressive strength and erosibility test results, as well as densities.

Initial rutting after 50 passes may be due to increased densification of the specimens in the wheel track. Since the rutting occurs in the first 50 passes and then remainded relatively constant for the remainder of the first 1000 passes, rutting may also be due to seating of the specimen.

The presence of water during the second 1000 passes contributed very heavily to increased rutting depth. As in most road surfaces following compaction by traffic, water ponded in the wheel track and as the wheel hit the specimen, fine particles were splashed out. In addition, the wet wheel "tracked" .the smaller particles out as it passed over the specimen. Together, these factors accounted for increased rut depth.

At the end of the 2-hour fogging period, the specimens were completely saturated and failed almost immediately when the load wheel was applied. Specimens, either treated or untreated, thus showed no resistance to the ponding of water over the two-hour fogging period.

Additive percentages of 0.25 and 0.50 showed lower values of rutting depth during the simulated rain-load period, especially the last 500 passes of that cycle.

Petro D Dust

Internal Mixture

Results of specimens containing Petro D Dust are shown in Fig. IV-20. In general, rutting depth increased as the additive content percentages were increased. Rut depth increased during the first cycle of loading, 250 passes, and then remained constant during the "no rain" phase. Sharp increases were again noted with simulated rain and rut depth increased until failure.

During the first 1000 passes, rut depths of treated specimens were greater than that of untreated specimens; however, during the last 500 passes of the "rain" phase, untreated rut depths exceeded that of both 0.1% and 0.25% Petro D Dust. These two concentrations also did not fail immediately on loading following two hours of fogging but reached failure after a total of 2500 passes while all other failed after 2000 passes. Failure was caused by the amount of water in the system, creating instability and loss of fine material. There was a very good correlation between the results of the unconfined compressive strength, erosibility, and trafficability tests, especially for 0.1% concentrations.

Seal Coat

Figure IV-21 shows the results of untreated specimens treated with a surface seal application of Petro D Dust immediately following compaction and cured for 24 hours before testing. The seal coat appeared dry at time of testing.

The immediate application failed after 250 passes, probably due to sealing of the surface without sufficient curing of the sample. These specimens were thus essentially tested at zero cure rather than 24 hours cure.

The later surface application survived up to 2000 passes, including 1000 passes in simulated rain. Rut depth was essentially the same as untreated through the first 1250 passes after which the rate of rut depth development of the untreated increased more sharply than that of the treated specimens. Surface sealing with Petro D Dust probably created a more tightly bound surface and prevented "splashing" and "tracking" of the finer particles during each pass of the loading wheel.

Surface seal specimens failed after two hours of fogging due to ponding of water in the untreated base of the specimens.

A combination of internal treatment followed by a later application of Petro D Dust to the surface could prove much more effective than either treatment used individually.

Elvanol 71-30 Figure IV-22 shows the results of traffic simulator tests on Elvanol 71-30. Again, rut depth increased with increasing additive percentage. In addition density decreased with increasing additive content.

Rut depth increased sharply during the first 50 passes and then leveled off throughout the rest of the first 1000 passes, indicating seating of the specimens and slight compaction in the wheel track. During the second 1000 passes with simulated rain, all specimens failed; 0.5% and 0.25% after 1500 passes and 0.1% after 1750 passes. Introduction of water into the system was followed by an increase in development of the wheel tracks due to splashing and tracking of fines out of the soil matrix. Rut depth appeared tied directly to compacted dry density in that those specimens having higher dry densities tended to have lower values of rut depth during testing.

Elvanol 52-22   Figure IV-23 shows the results of specimens containing Elvanol grade 52-22. The reason for the strange departure of these specimens from those which have been discussed already is unexplained. Perhaps lack of a completed chemical reaction after 24 hours would account for the high values of rut depth during the first 1000 passes. It is thought that Elvanol 52-22 combines with the soil to form a strong, dispersed structure and that strength increases with cure time. If molding and curing conditions did not allow adequate cure and yet still dispersed the structure, then increased rut depth would be the result of recompacting a dispersed structure. If the additive percentage were increased sufficiently beyond 0.5%, then strength of the dispersed system might also have been increased and rut depth decreased.

Introduction of water into the system induced failure at or before 1500 passes. Lack of an adequate chemical reaction followed by saturation, thus created failure.

Figures IV-24 and IV-25 represent Elvanol grades 71-30 and 52-22, respectively, following a 7-day air cure. Obviously, cure time has an effect on the overall trafficability of specimens, as shown by comparison with Figs. IV-22 and IV-23 respectively. Improvement, in both cases, is obviously attributed to the formation of more bonds due to increased cure time. Centerpoint rutting for both cases was significantly lessened during the first 1000 passes, being nearly identical for all three concentrations.

During the first 250 passes under simulated rain, each concentration of grade 71-30 remained reasonably consistent with the preceeding 1000 "no rain" passes, but then increased sharply, survived the two-hour fog period, and ultimately failed after a total of 2250 to 2500 passes of wheel loading. Specimens containing grade 52-22 while affected by increased cure time, failed after 1500-1750 passes under "rain." From Figs. IV-24 and IV-25 it is obvious that increased cure time, decreased rutting depths and significantly improved the trafficability of both Elvanol 71-30 and 52-22. It may at least be postulated that similar effects might occur with the other products analyzed in this study.

Summary of Results

1. Generally, rut depth increased as the dry density decreased; density being a function of the additive content.

2. Initial rut depth occurred early, usually within the first 50 to 250 passes of the first 1000 pass cycle, then leveled off. This was due primarily to increased compaction of the wheel track as well as seating of the specimens in their holding rings.

3. Increased rut depth during the second 1000 pass cycle with simulated rain of 0.15 to 0.20 in. per hour was due to either (a) ponding of water in the wheel track and subsequent "splashing" out of fines, due to the dynamic impact of the loading wheel, (b) "tracking" of the fines by the loading wheel itself, or (c) a combination of both. This resulted in removal of fines from the soil matrix and increased settlement in the wheel track.

4. Failure of specimens was due principally to presence of water in the soil matrix rather than the loading wheel. Water in the

base of the specimens caused saturation and a subsequent reduction in bearing capacity, denoted by bulging on either side of the wheel track.

5. Petro D Dust, used as a seal coat, provided excellent trafficability when applied after the base had cured, but failed almost immediately, when applied immediately after molding.

6. Generally, low additive concentrations showed the best results on the traffic simulator.

7. Although increased cure time influenced performance of the Elvanols, low dry densities were still considered the main factors in increased rut depth.

8. When compared with the untreated specimens, 0.5% Stypol, 0.10 to 0.25% Petro D Dust, appeared to provide the most significant increase in fine material retention and waterproofing as well as stability under moving load and imposed environmental conditions.

Freeze-Thaw Tests

The purpose of this test is to determine the freeze-thaw durability characteristics of treated soils in the laboratory utilizing the freeze-thaw apparatus developed by K. P. George and D. T. Davidson at Iowa State University. The test procedure however was modified slightly so that actual field conditions could be more closely approximated.

The freeze-thaw apparatus is shown in Fig. IV-26. The Iowa Freeze-Thaw Test requires unconfined compressive strengths of the test specimen and an auxiliary specimen of the same dimensions, having been immersed in water for the duration of the freezing and thawing cycles. In this manner, a resistance to freezing could be calculated. Resistance to freezing was defined as the strength following freeze-thaw divided by the immersed strength, expressed as a percentage.

Because none of the specimens evaluated were capable of withstanding immersion for any lengthy periods of time, a modified test was developed.

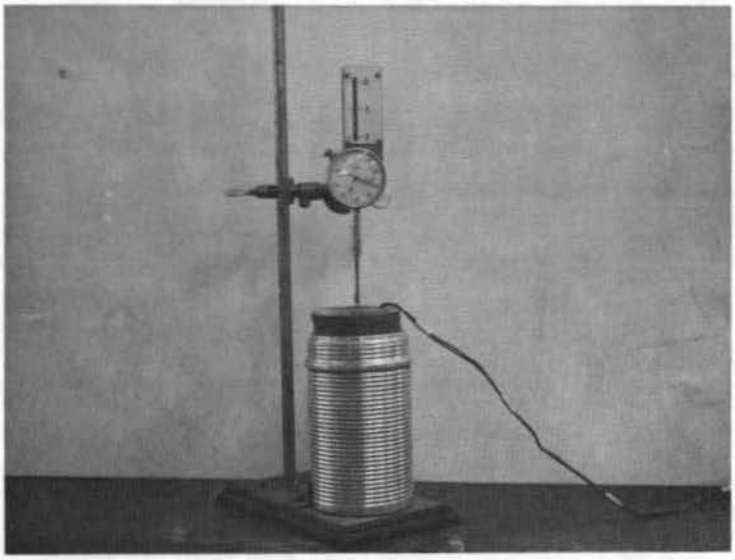


Fig. IV-26. Freeze-thaw test apparatus.

Sample Preparation, Curing, and Testing

The freeze-thaw test utilizes the same 2-in. diameter by 2-in.-high specimens that have been referred to previously in unconfined strength and erosibility. Specimen preparation and curing conditions remained unchanged, with specimens in this test air cured for 24 hours. Air curing was judged to be more realistic than a moist cure for actual field application. In addition, 24 hours cure time was selected over 7-day cure for convenience, speed, and simplicity of testing. Comparative results of the 24-hour cure were also more readily applicable to both trafficability and erosibility tests.

Following curing, specimens were placed in their containers, which in turn were placed in water filled thermos flasks where the water was regulated to retain intimate contact with the specimen base, and initial height measurements were made. Specimens remained unsealed in order that differential movement would be indicative of unsealed field application.

Flasks and specimens were placed in a freezer maintained at 20 °F ± 2 for 16 hours, removed and allowed to thaw for eight hours. This constituted one cycle of freezing and thawing. A light bulb at the base of the specimen holder maintained the water in the thermos at about 35 °F so that capillary moisture was available at the base of the specimen during both freezing and thawing; simulating field conditions. Height measurements were recorded after each cycle of freezing and thawing for the duration of test.

Additive treatment levels were selected on the performance results obtained from unconfined strength, erosibility, and trafficability tests. The additive treatment levels evaluated in the freeze-thaw test were:

Petro D Dust	0.05, 0.25, 0.10, 0.05
Petro D Dust	Surface seal coat
Elvanol 71-30	0.50, 0.10
Elvanol 52-22	0.50, 0.10
Stypol 40-5020	0.25, 0.50.

A minimum of two treatment levels were tested for each product. Central Chemicals, Kelpak, Clapak, Claset, and SA-1 had not been tested at time of this writing.

Results

Figures IV-27-38 present results of freeze-thaw tests. Height percentage change is shown on the ordinate and number of cycles of freezing and thawing are shown on the abscissa. Only six cycles are presented due to equipment malfunction. The area under the curves was measured to determine composite elongation during the six cycles. In this manner, a quantitative evaluation was achieved without relying on unconfined compressive strength results.

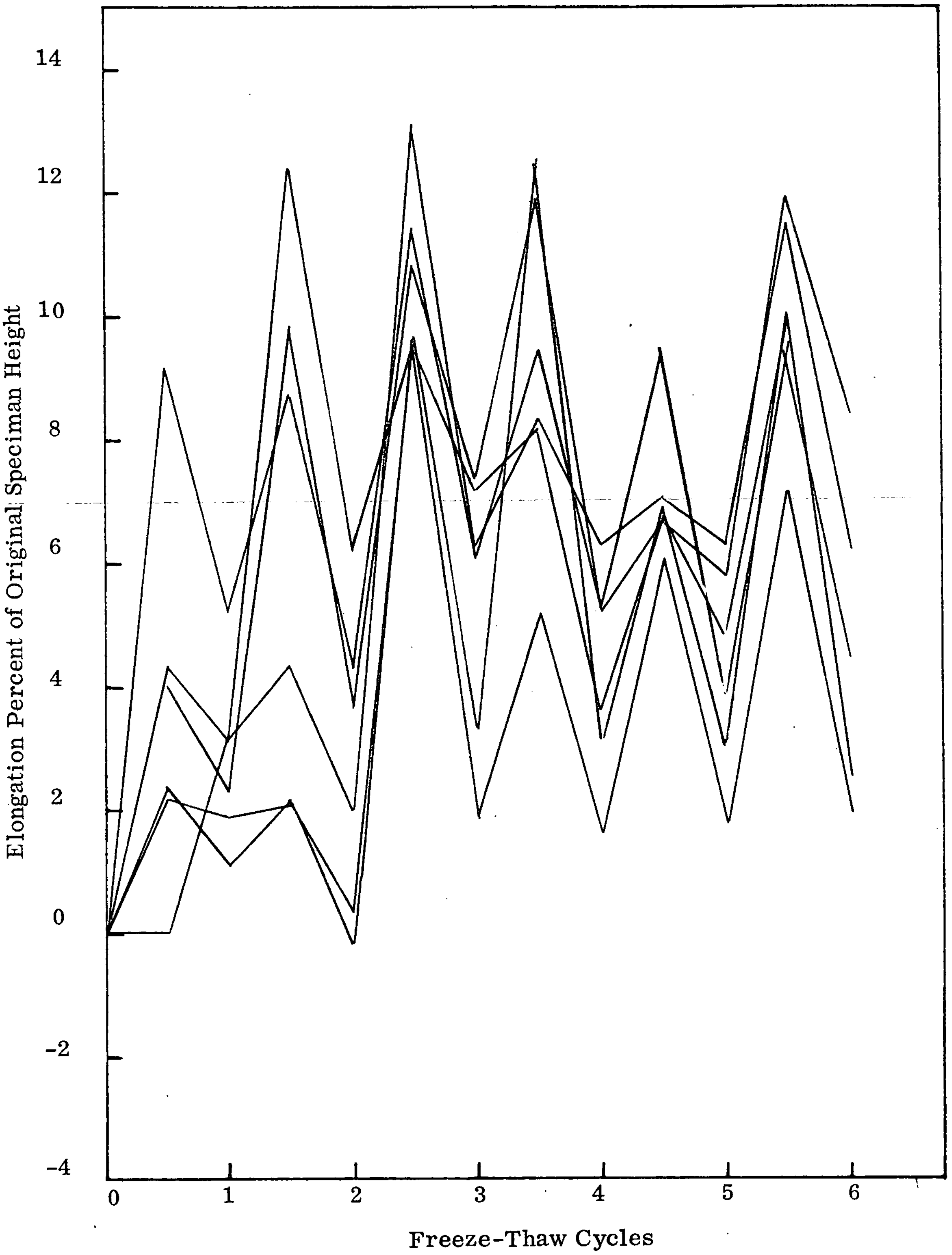


Fig. IV-27. Elongation-contraction traces for untreated specimens.

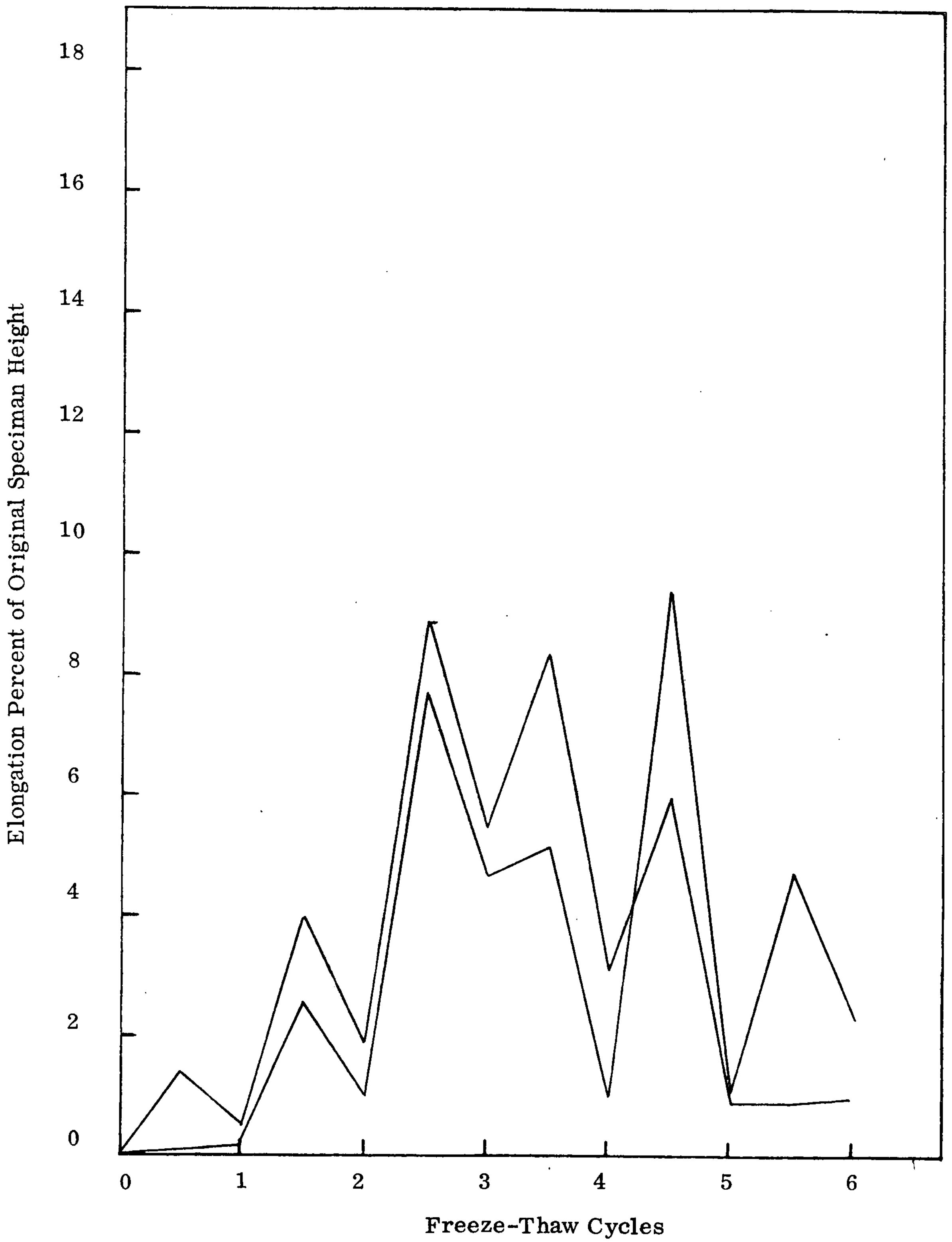


Fig. IV-28. Elongation-contraction traces for Stypol 40-5020, 0.5%.

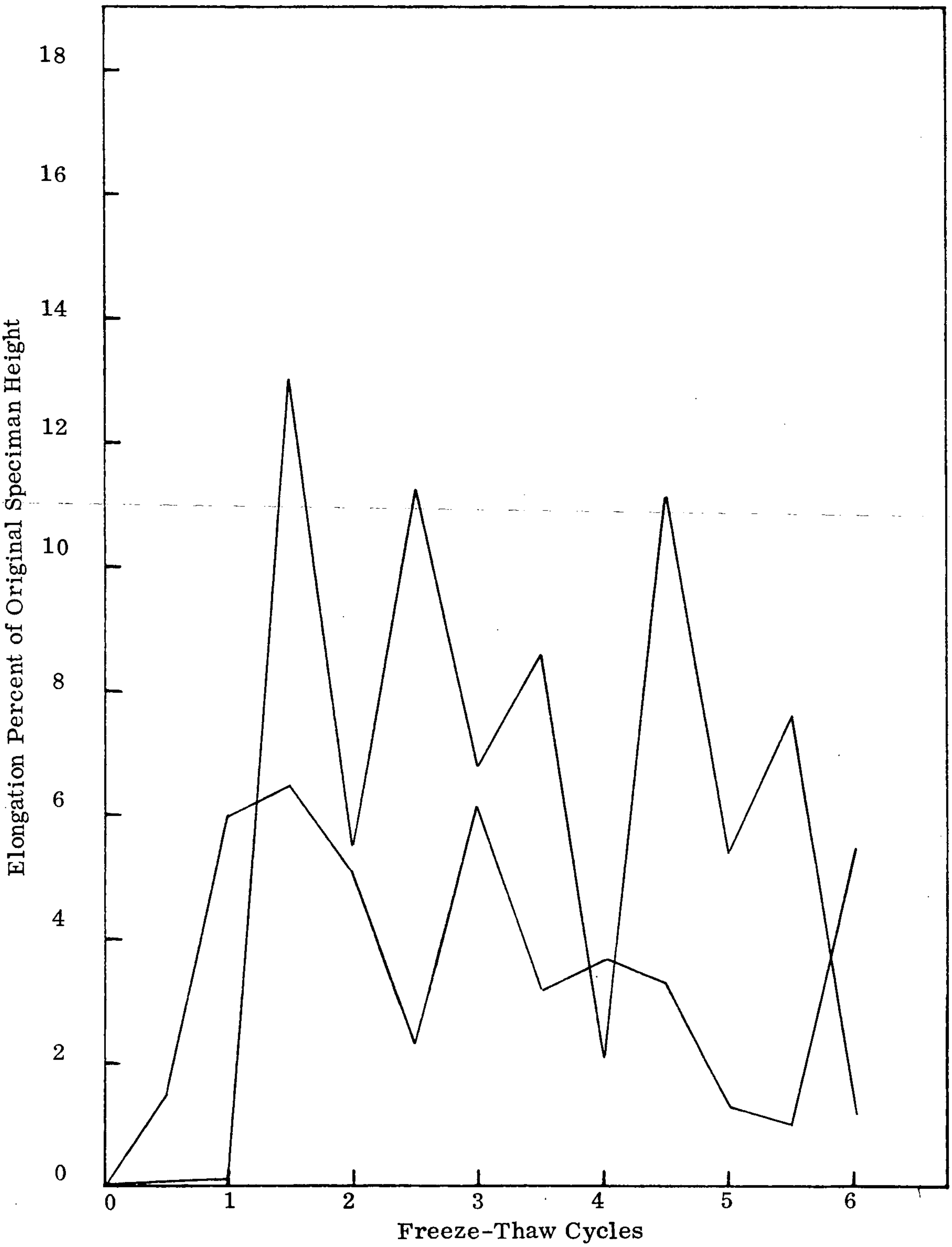


Fig. IV-29. Elongation-contraction traces for Stypol 40-5020, 0.25%.

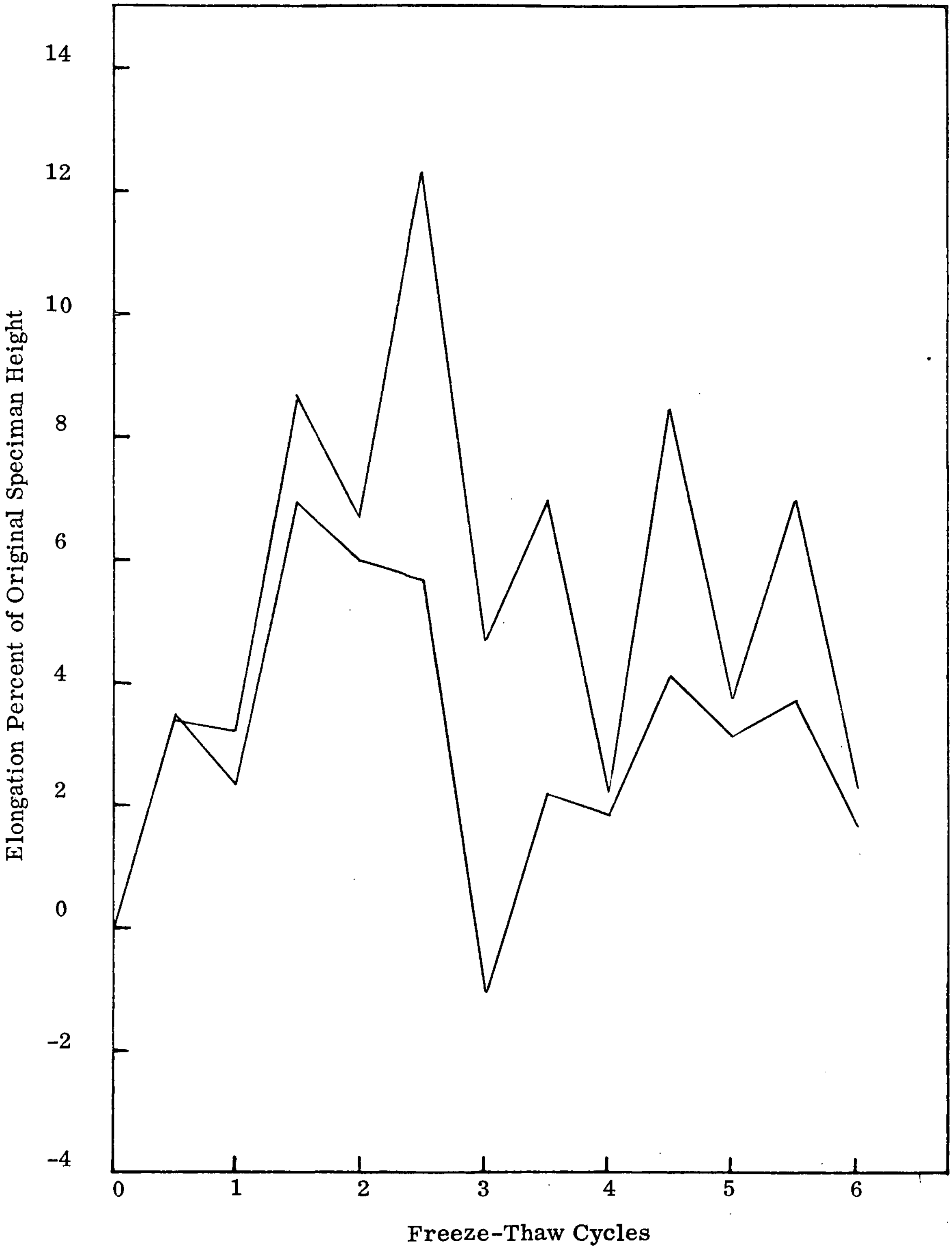


Fig. IV-30. Elongation-contraction traces for Petro D Dust surface application only.

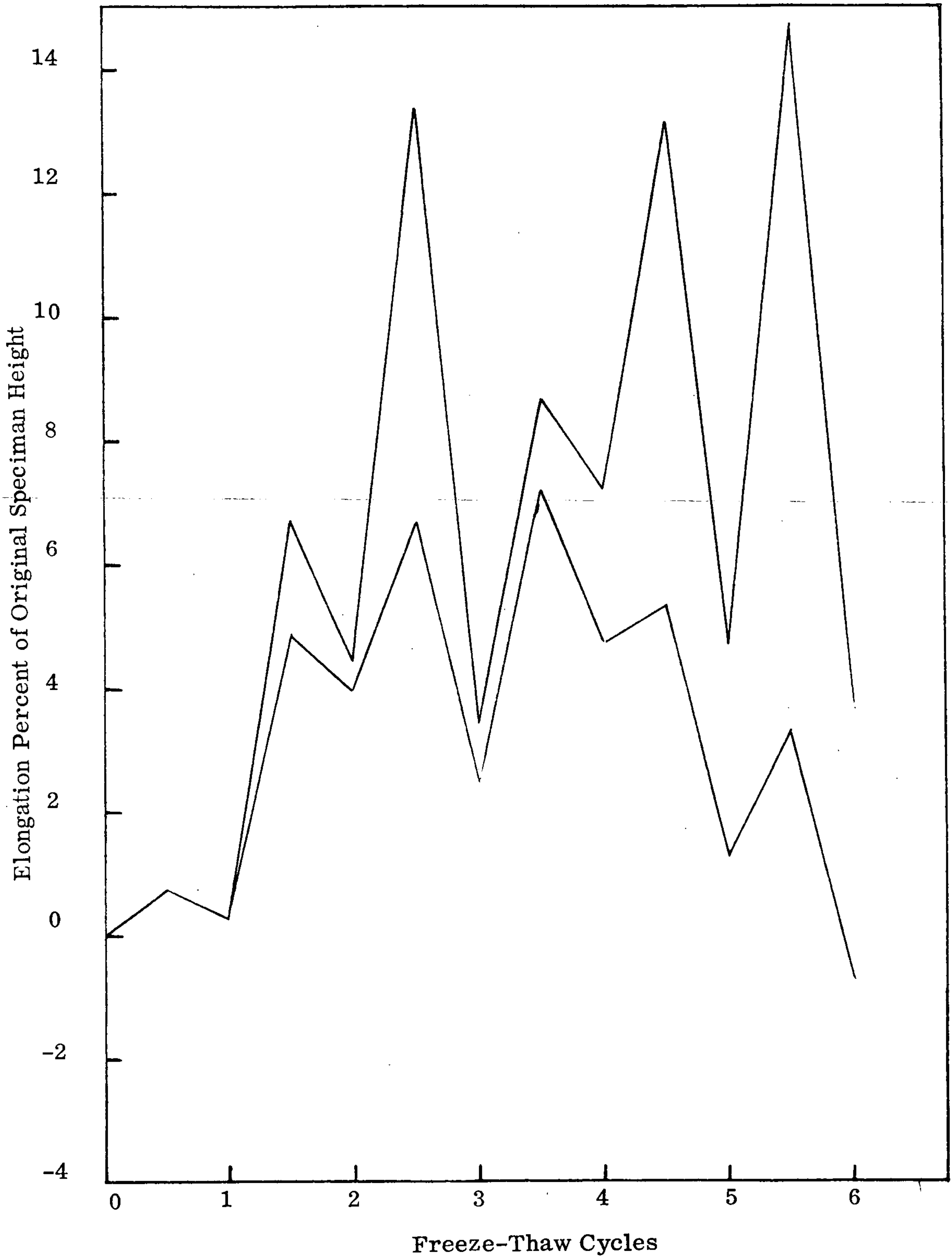


Fig. IV-31. Elongation-contraction traces for Petro D Dust, 0.50%.

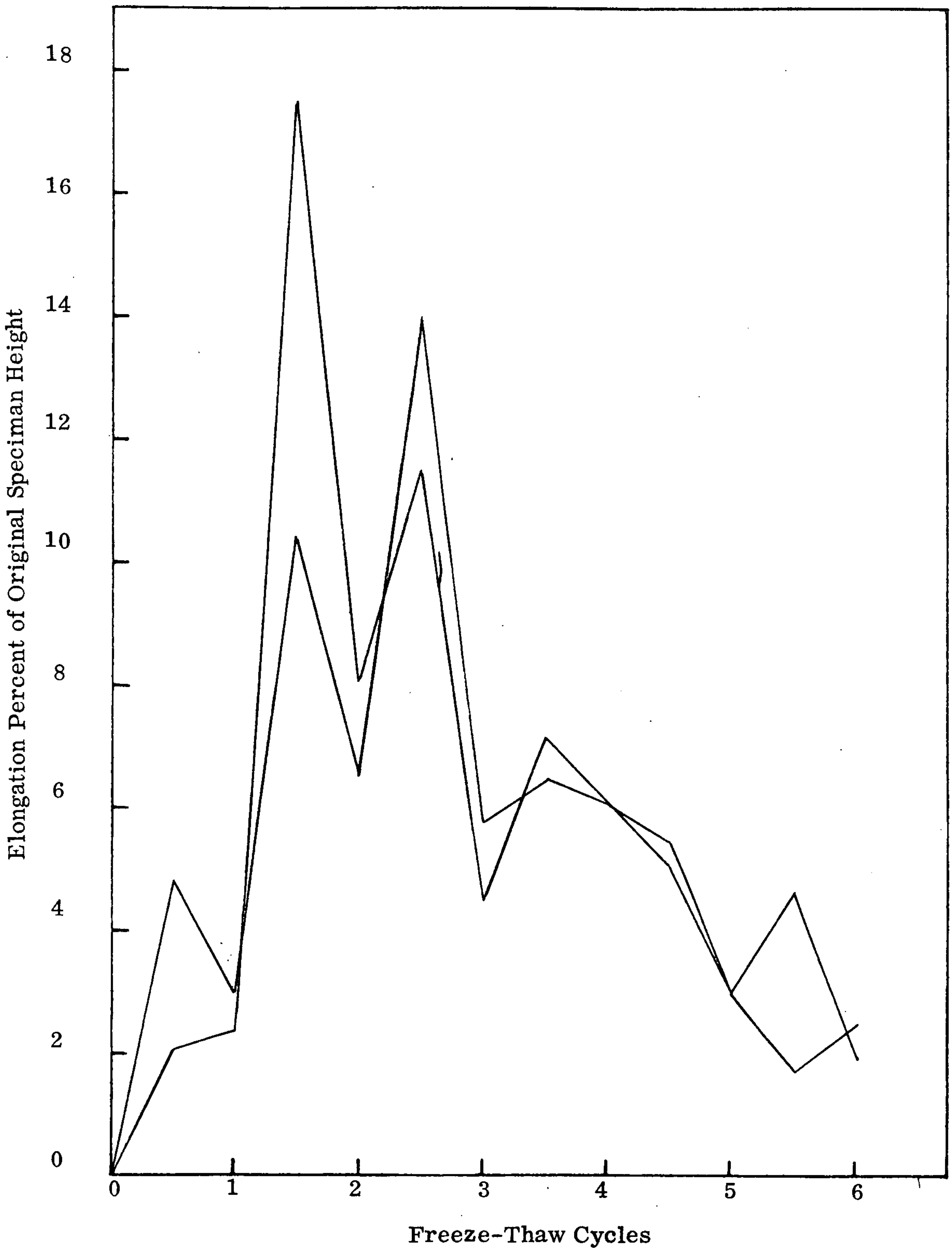


Fig. IV-32. Elongation-contraction traces for Petro D Dust, 0.25%.

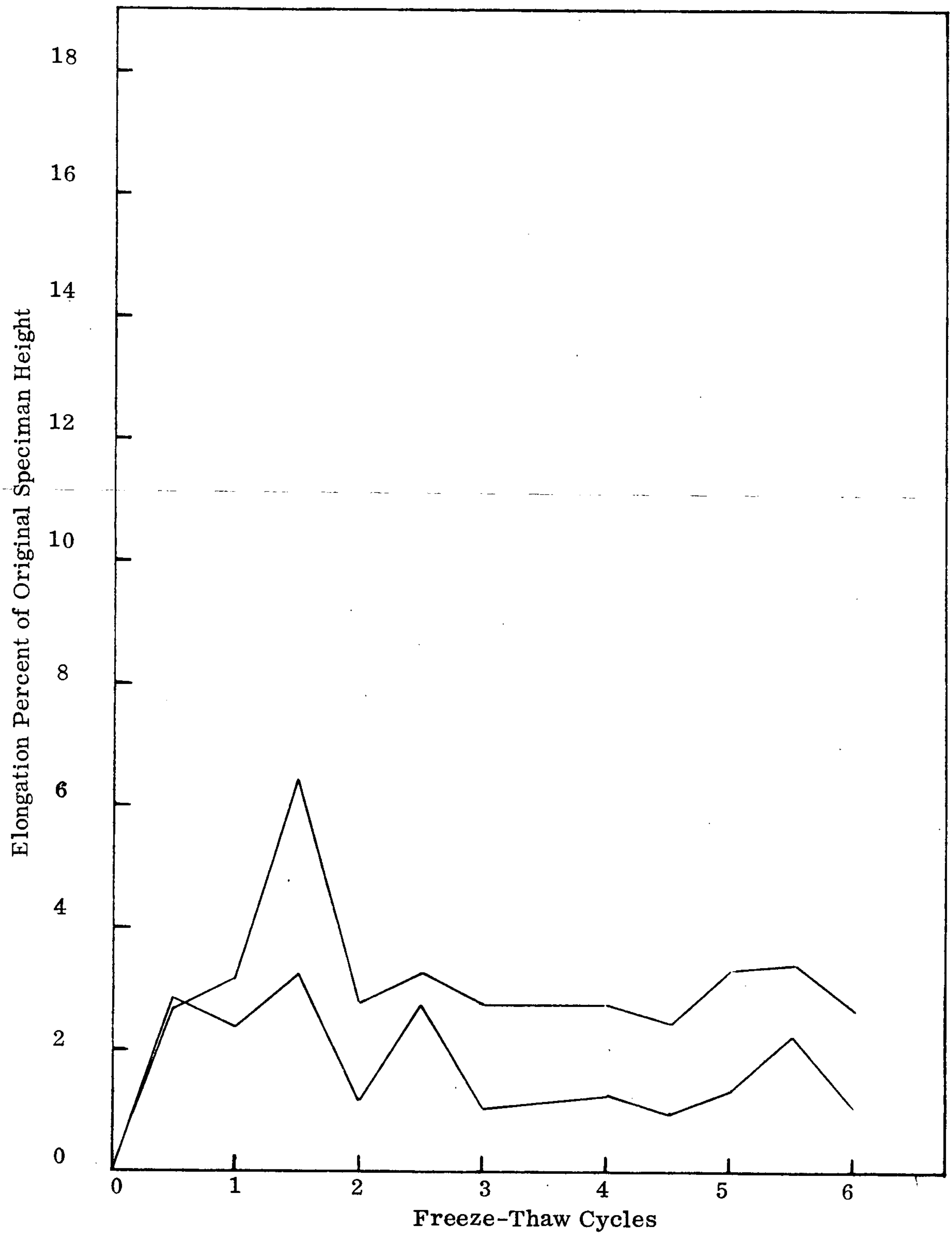


Fig. IV-33. Elongation-contraction traces for Petro D Dust, 0.10%.

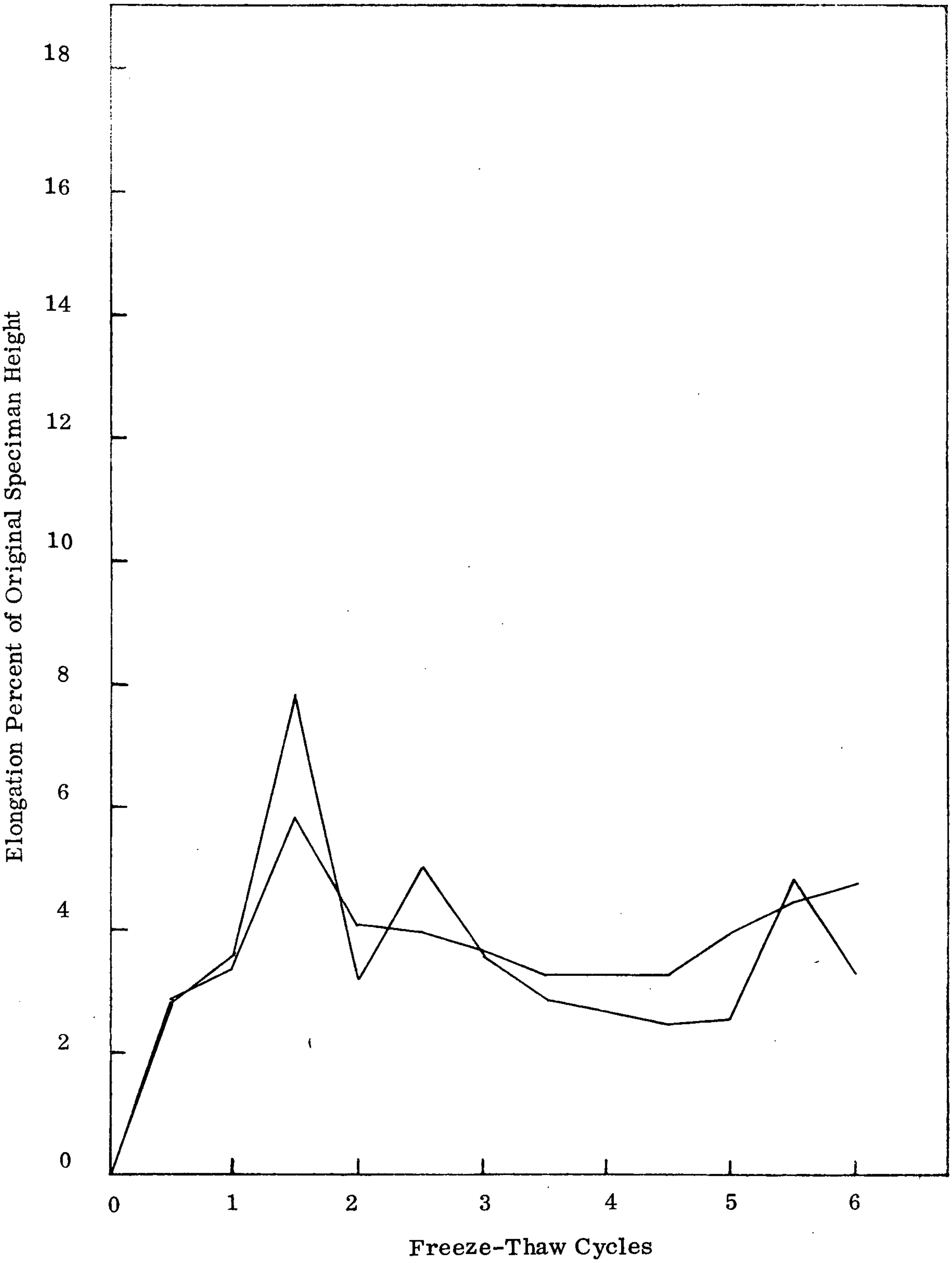


Fig. IV-34. Elongation-contraction traces for Petro D Dust, 0.05%.

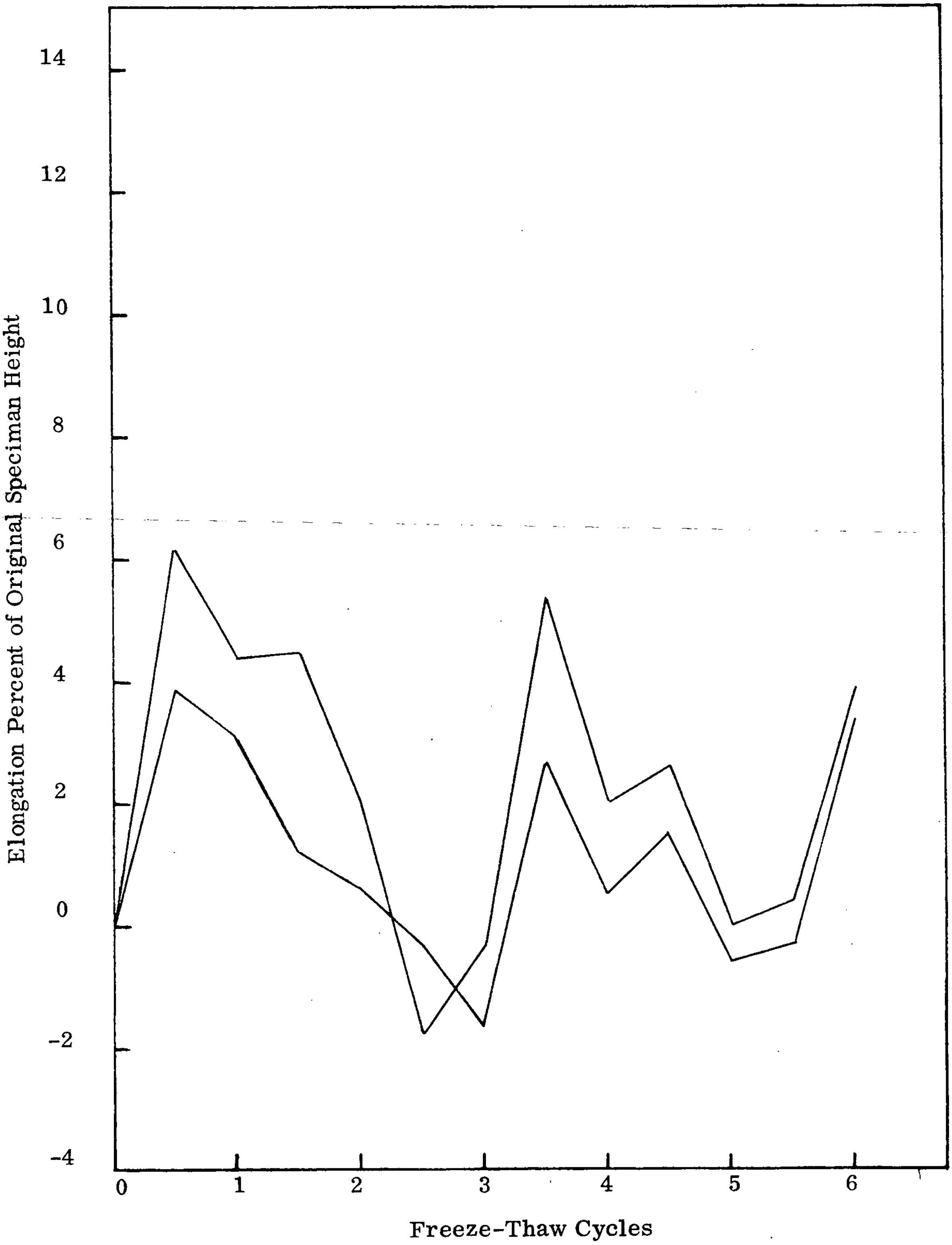


Fig. IV-35. Elongation-contraction traces for Elvanol 71-30, 0.50%.

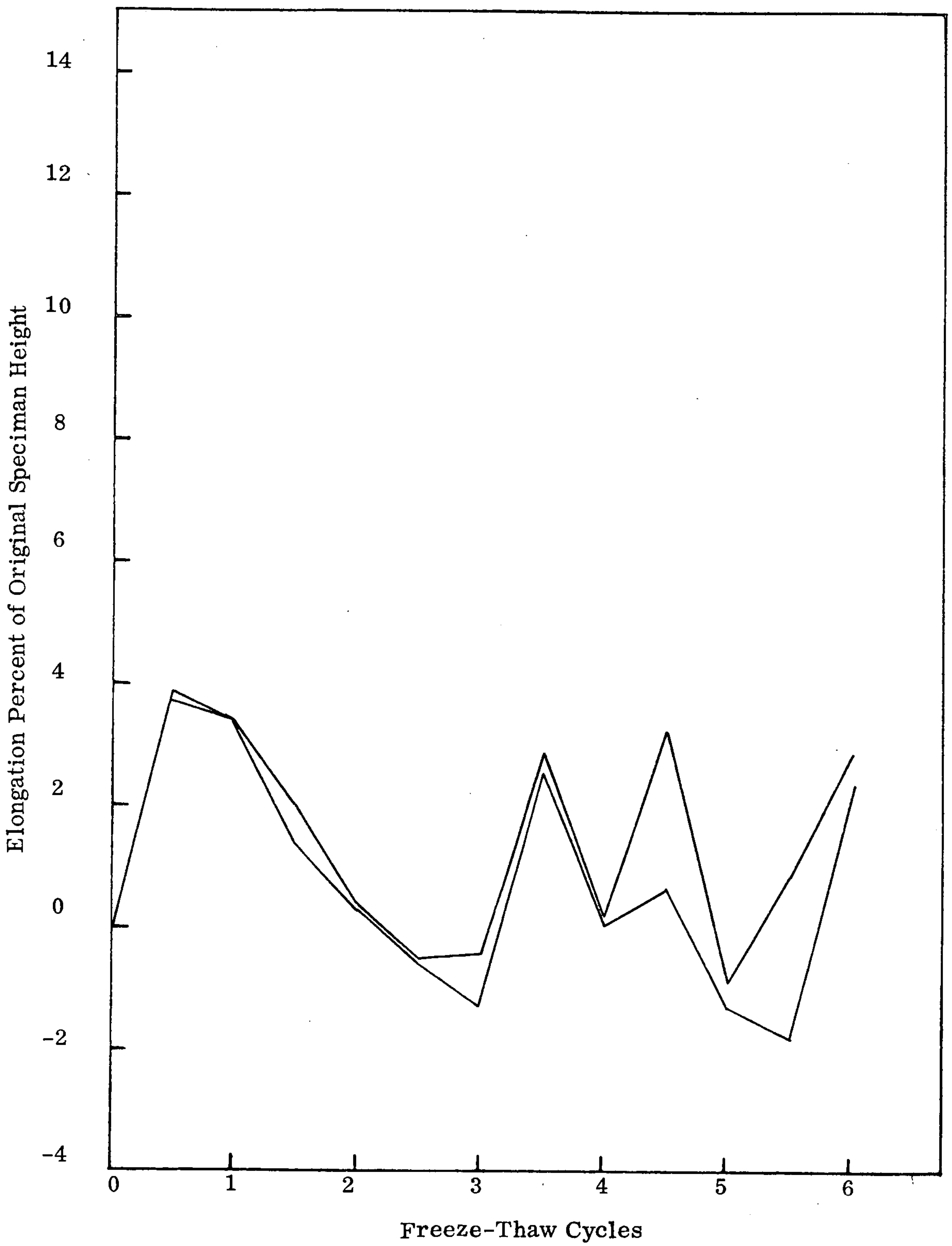


Fig. IV-36. Elongation-contraction traces for Elvanol 71-30, 0.10%.

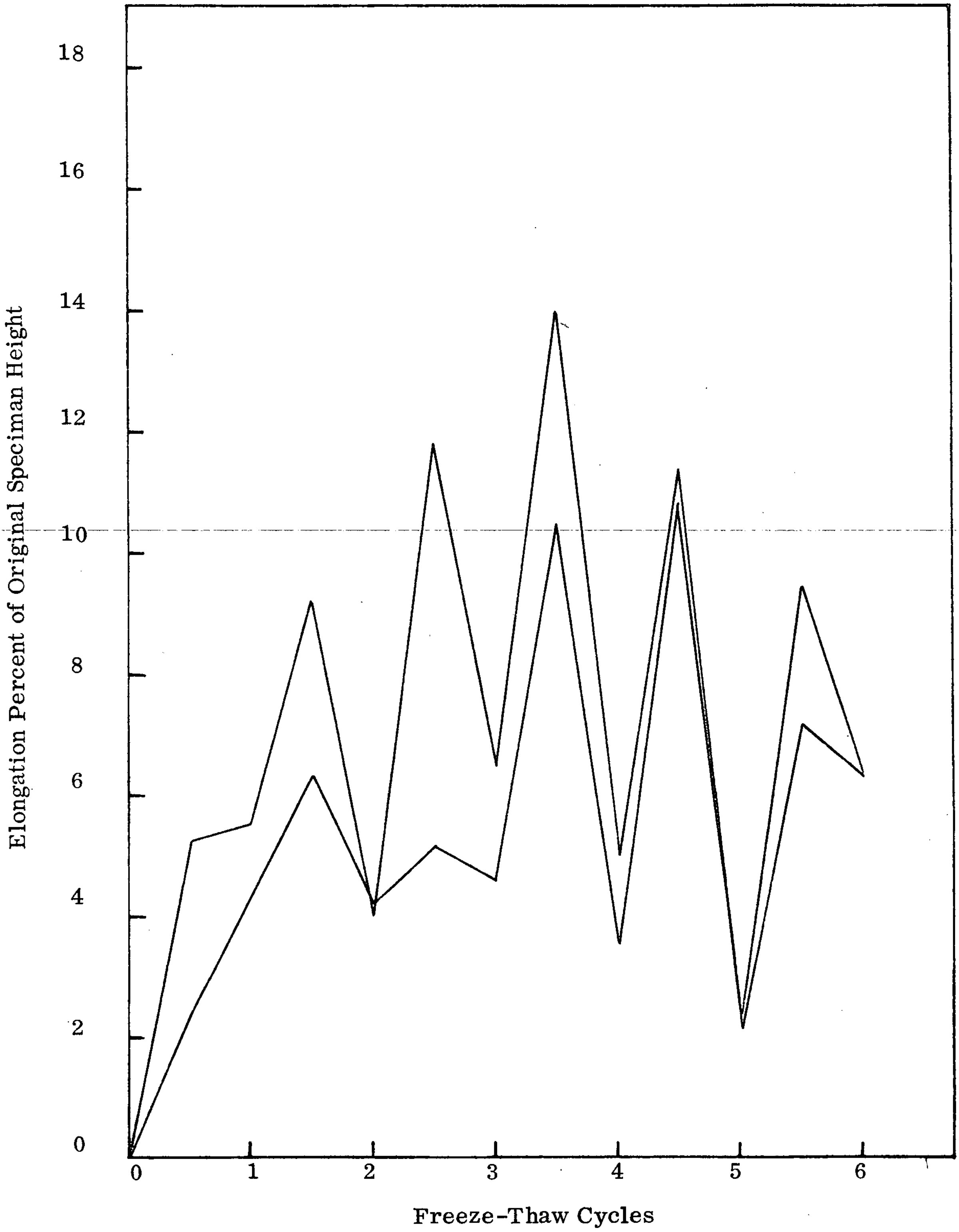


Fig. IV-37. Elongation-contraction traces for Elvanol 52-22, 0.50%.

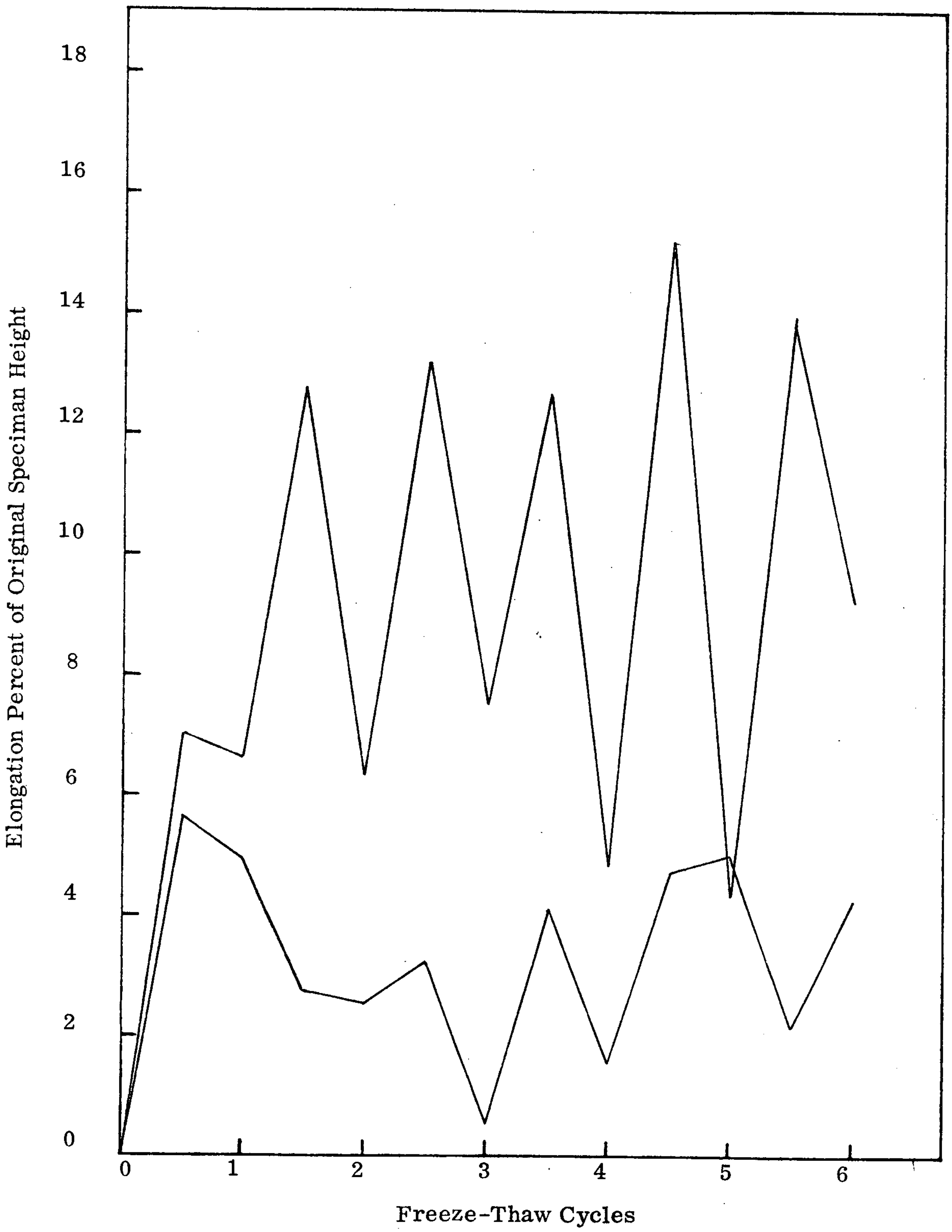


Fig. IV-38. Elongation-contraction traces for Elvanol 52-22, 0.10%.

Figure IV-27 presents height elongation and contraction traces of points on the surface of untreated specimens during the six cycles of freezing and thawing. Average elongation in six cycles was determined to be 5.6%, a value which was used as a control for the treated specimens.

Figures IV-28-38 show the traces of points on the surfaces of treated specimens. Values of average elongations in six cycles for each treatment are shown in Table IV-8.

Table IV-8. Average freeze-thaw elongation.

Product	Treatment level, % dry soil weight	Elongation, %
Untreated	0	5.6
Petro D Dust	0.50	5.0
	0.25	5.9
	0.10	2.4
	0.05	3.6
	Seal coat only	4.5
Elvanol 71-30	0.50	1.4
	0.10	1.0
Elvanol 52-22	0.50	6.3
	0.10	6.2
Stypol 40-5020	0.50	3.3
	0.25	4.7

Introduction of small amounts of Petro D Dust, especially 0.10%, increased specimens resistance to freeze-thaw elongation and contraction as noted by a composite elongation of 2.4% compared to 5.6% for untreated specimens. As in unconfined strength results, an optimum appears at 0.10% Petro D Dust. A concentration of 0.25% Petro D Dust, while indicating good abrasion resistance and waterproofing in trafficability and erosibility tests, did not improve the freeze-thaw durability characteristics, having a composite elongation of 5.9% compared to a control value of 5.6%.

The surface seal application of Petro D Dust improved freeze-thaw resistance somewhat; not as much as 0.10% treatment level but more than the control, having an elongation of 4.5%. These results correlated very well with erosibility results.

Elvanol 71-30 exhibited the best resistance to freezing and thawing, being slightly better at 0.10% (1.0% elongation) than at 0.50% (1.4% elongation). These results correlated very well with trafficability and erosibility tests.

Elvanol 52-22, however, exhibited the worst resistance to freezing and thawing of any specimens tested, having composite elongations of 6.2 and 6.3% for treatment levels of 0.10 and 0.50%, respectively. Results of trafficability tests indicated that Elvanol 71-30 performed better than Elvanol 52-22 and obviously the same trend was evident in the freeze-thaw test. There was no apparent correlation between strength, erosibility, and freeze-thaw test results.

0.5% Stypol 40-5020 exhibited a beneficial effect on freezing and thawing resistance as was expected from unconfined strength, erosibility, and trafficability test results. 0.25% also provided some beneficiation.

Summary of Results

1. Low concentrations of Petro D Dust tend to inhibit frost heave by reducing the average elongation compared to untreated specimens.

2. Elvanol 71-30 reduces elongation most significantly especially at the lower concentration of 0.10%.

3. Stypol 40-5020 tends to decrease elongation although not as much as Elvanol 71-30 or low concentrations of Petro D Dust. An optimum remains apparent at 0.50%.

4. Elvanol 52-22 does not effectively reduce elongation at either of the concentrations tested.

5. There is a slight beneficiation using a Petro D Dust seal alone.

6. Overall results of the freeze-thaw tests correlate well with previous tests.

Moisture Tension Test

Water is a very important factor in highway construction and embankment stability. There should be enough water in the soil to allow proper compaction and care should be taken so as not to overcompact. Theoretically, bases and subgrades should be kept at or below optimum water for compaction following construction, to provide increased stability, reduce effects of frost heave in the winter, rutting and potholing in the spring. However, this is seldom practical even in dry regions because condensation can keep these components moist. In addition, once an embankment is covered with an impervious pavement or wearing surface, moisture content will begin to increase due to prevention of evaporation and transpiration. It should be noted, however, that any treatment which tends to reduce moisture content in the base and subgrade should be considered beneficial to the overall stability of the pavement structure.

Basically, there are two categories of water held above the water table, hygroscopic and capillary. The former is water absorbed by soil from an atmosphere of water vapor, occurring as thin films surrounding individual particles, while the latter is that water held in the pores between grains with aid of surface tension. It is this latter capillary water which is of greatest interest to the engineer.

Capillary water is best studied through the use of sorption curves which show the relationship between moisture content and the reduced pressure of the capillary water held above the water table; the latter is referred to as soil water suction or simply soil suction.

Saturated soil has no suction, but from the water table upward, suction develops in the soil according to the principles of hydrostatics and capillarity.

For example, at a point 10 ft above the water table, capillary water has a pressure head of - 10 ft, a gage pressure of - 4.3 psi, or a value of soil suction of 4.3 psi. It has been found experimentally that the decrease in moisture content with increasing soil suction is continuous from saturation to oven dryness.

The moisture tension test was selected to obtain sorption curves of soil treated with various additives. The pressure membrane extraction apparatus used in this test is shown in Fig. IV-39. Pressure is applied in the form of compressed air to the top of soil specimens sealed in the chamber. Water is forced from the soil through a cellulose membrane and out a drain line until equilibrium is achieved. At equilibrium there is an exact relationship between soil suction in the specimens and applied pressure in the chamber.

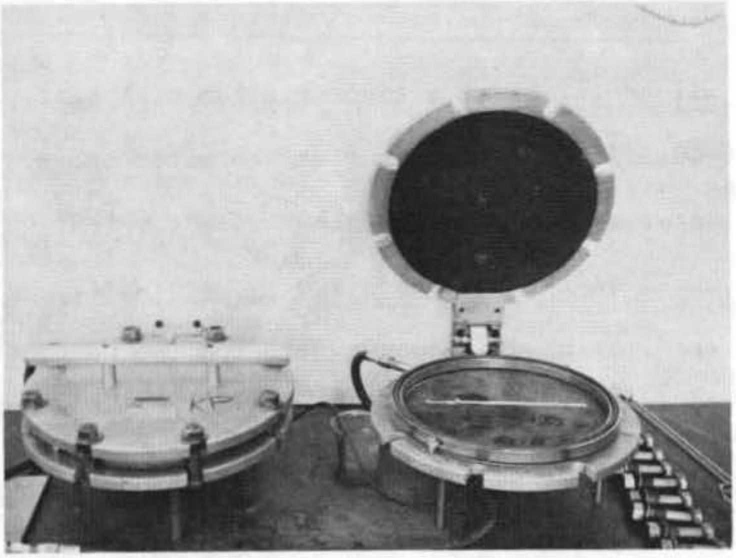


Fig. IV-39. Pressure membrane extraction apparatus.

Sample Preparation and Testing

The moisture tension test was used to evaluate the Central Chemicals, Kelpak, Clapak, Claset, and SA-1 with regard to their moisture control capabilities. Manufacturers' literature indicated that several of these chemicals might be used as moisture control agents, especially in roadway bases and subgrades. If this were in fact, the case, then their potential as a membrane in low lying, wet areas would be realized.

Specimens containing various concentrations of Kelpak, Clapak, Claset, and SA-1 were prepared at moisture contents above the liquid limit of the soil and were hand mixed until all reached approximately identical consistency. Samples were then poured into rubber retaining rings in the apparatus, the lid closed and sealed, and pressure applied. Equilibrium was achieved with the first 16 hours at each pressure with no appreciable change in water content noted beyond that point. However specimens were arbitrarily removed at 24-hour intervals for moisture content determinations. From these moisture samples, curves of moisture content versus applied pressure, or sorption curves were constructed. Applied air pressure, or soil suction, was converted to feet of water for ease of understanding.

Test Results

Sorption curves for all additives at a concentration of 1 part per 1000 are shown in Fig. IV-40. At 0 pressure, or at the water table, Claset-, Clapak-, and Kelpak-treated soils contained slightly less moisture than untreated, while SA-1 contained slightly more water. As the pressure was increased, or as distance upward from the water table increased to about 22 ft, moisture in all specimens except Claset decreased somewhat uniformly. With further increases in height above water table, Claset showed a significant decrease in moisture content when compared with untreated specimens. Above 45 ft, all treated specimens indicated a reduction in moisture content to some degree. In general, there appeared to be some beneficial effect with the addition of trace amounts of these chemicals, especially above a height of 45 to 50 ft above water table.

Sorption curves of soil specimens containing concentrations of 10 parts per 1000 are shown in Fig. IV-41. In general, these curves lie in a tight band almost centered on the untreated curve throughout the entire height range above water table. It may be noted that Claset contained somewhat greater moisture content than untreated specimens throughout. SA-1 seemed to decrease moisture between 25 and 70 ft with the most significant decrease around 45 ft. Overall, there were no other appreciable changes in moisture content with addition of 10 parts per 1000 of Kelpak or Clapak.

Sorption curves for specimens containing 20 parts per 1000 are shown in Fig. IV-42. Again Claset reduced moisture content below that of untreated; this is especially significant at heights between 45 ft and 90 ft. Kelpak also reduced the moisture content somewhat above

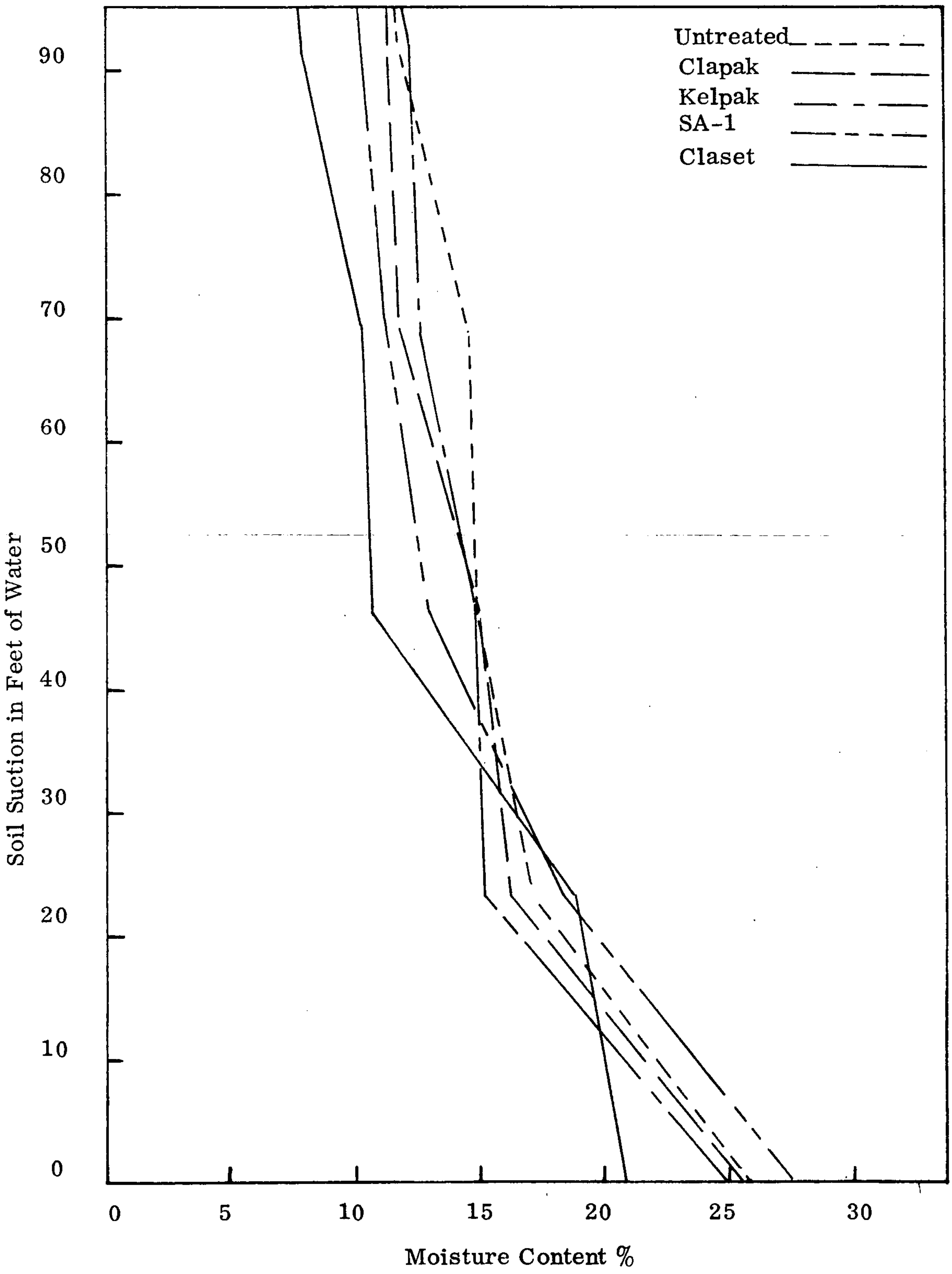


Fig. IV-40. Sorption curves of Central Chemicals at 1/1000 concentration.

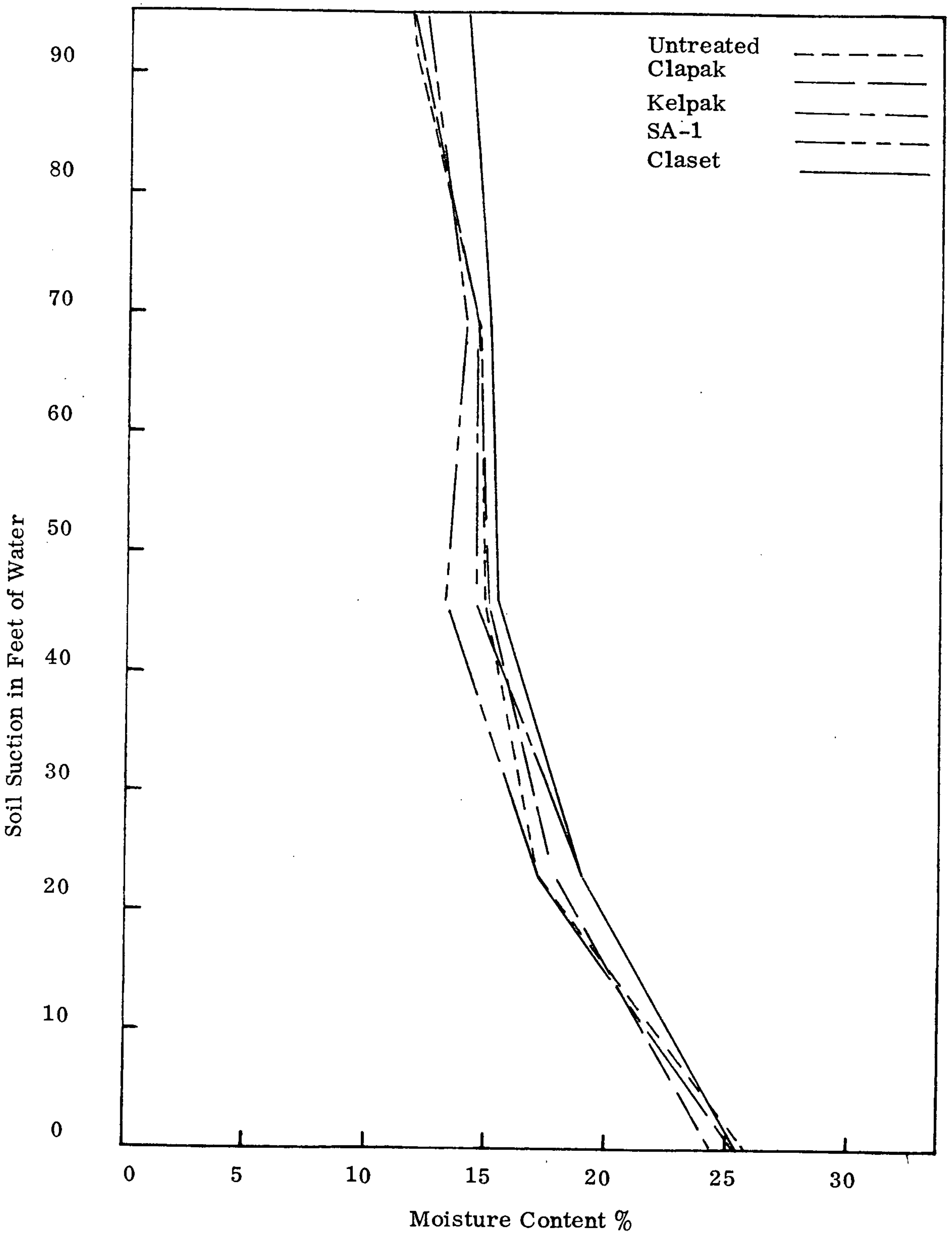


Fig. IV-41. Sorption curves of Central Chemicals at 10/1000 concentration.

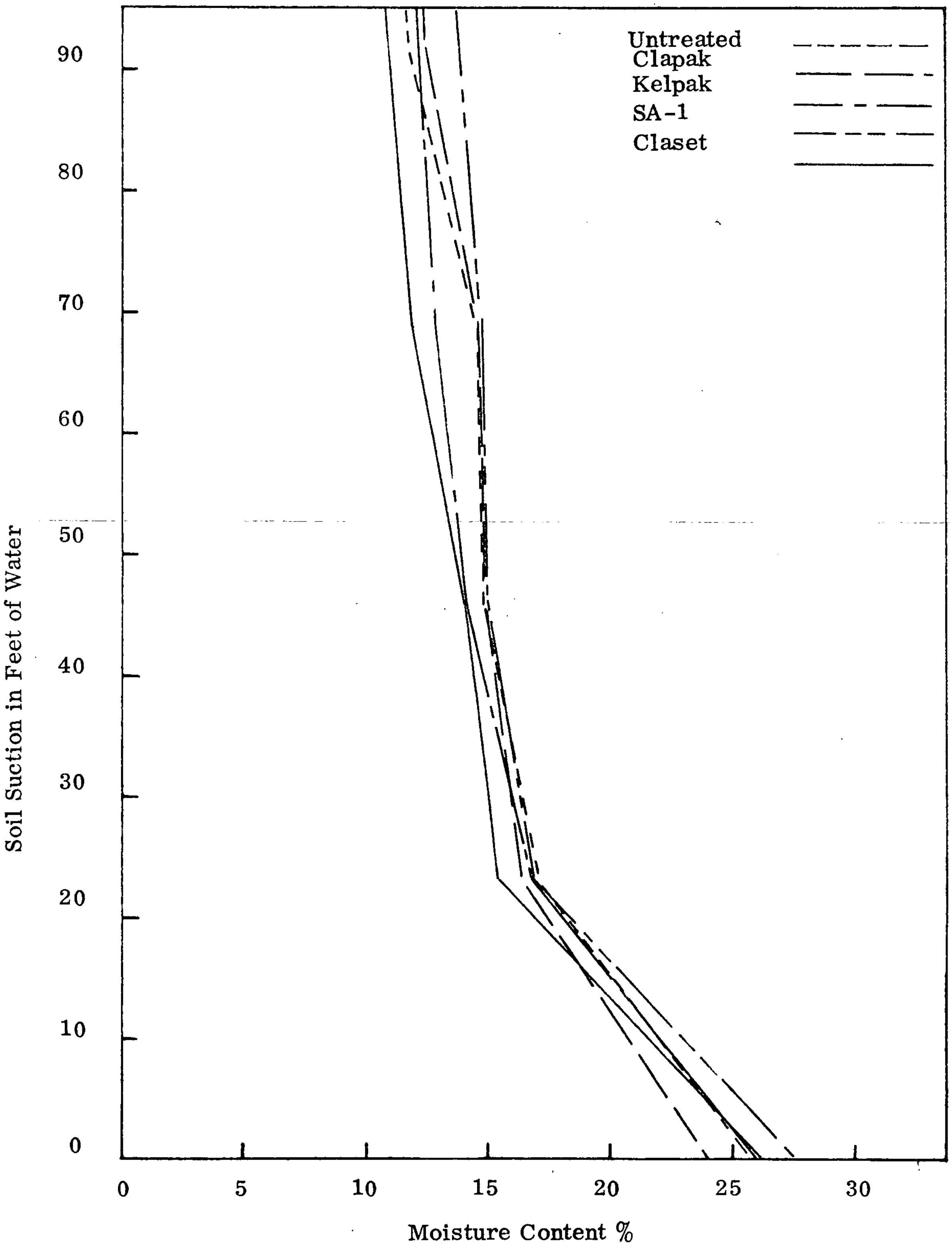


Fig. IV-42. Sorption curves of Central Chemicals at 20/1000 concentration.

25 ft. The moisture content of specimens containing either SA-1 or Clapak did not tend to reduce moisture with the exception of Clapak at the water table where the reduction was about 2% moisture.

Summary of Results

1. Claset, in concentrations of 1/1000 or 20/1000 appears effective in controlling moisture in the subgrade. This is in agreement with the manufacturers' literature and claims.

2. Claset is not an effective moisture control agent at a concentration of 10/1000, the reason being unknown.

3. In trace amounts (1/1000) Claset is beneficial at the water table and at heights of 25 or more ft above the water table while greater concentration (20/1000) are beneficial only above 25 ft.

4. SA-1 is beneficial in concentrations of 10/1000 between 25 and 70 ft above the water table.

5. Kelpak, at a concentration of 20/1000 is beneficial above 25 ft.

6. Other additive concentrations were of limitated beneficiation due to very small moisture decreases or no decreases at all.

X-Ray Diffraction Studies

The Central Chemicals, Kelpak, Clapak, Claset, and SA-1, were subjected to further testing in an attempt to rationalize the stabilization mechanism. Results of unconfined strength and erosibility tests indicated that something was happening even with trace amounts of additives. An investigation utilizing x-ray diffraction was selected in order to evaluate the effects of chemicals on the crystal structure of soil, probably the only way to determine what trace concentrations were doing.

Sample Preparation and Testing

Specimens containing varying concentrations of Kelpak, Clapak, Claset, and SA-1 were investigated using an x-ray diffractometer, model G.E. XRD-5, with filtered copper radiation. A sodium montmorillonite was selected as carrier because of its unique, characteristic first-order basal spacing and because it is the most abundant and active clay mineral in Iowa soils. Due to their proprietary nature, very little information pertaining to the chemical composition of Kelpak, Clapak, Claset, or SA-1 was available. One component of Kelpak, the catalyst which is packaged separately to be added on the job site, was identified using x-ray diffraction as Ammonium Di-Chromate, $(NH_4)_2Cr_2O_7$.

In addition, the pH of each concentration investigated was measured with a pH meter and values are listed in Table IV-9. It should be noted that Clapak, Claset, and SA-1 are very acidic, whereas Kelpak is only slightly basic. The degree of acidity increased with water concentration.

Specimens of montmorillonite containing the chemicals at each of the three concentrations noted in Table IV-9 were x-rayed immediately after mixing. Enough liquid was added to the clay to obtain a very plastic mix which was then placed directly into a sample holder and x-rayed.

Results

Analysis was conducted on the basis of first-order basal spacings (d_{001}) of treated montmorillonite. The curves obtained using the

Table IV-9. pH values for investigated concentrations.

Product	Concentration	pH
Kelpak	1/1000	7.25
	10/1000	8.35
	20/1000	7.40
Clapak	1/1000	1.80
	10/1000	0.95
	20/1000	0.68
Claset	1/1000	3.00
	10/1000	2.18
	20/1000	2.00
SA-1	1/1000	1.70
	10/1000	0.80
	20/1000	0.50

diffractometer are shown in Figs. IV-43-47. However, the data is also tabulated for better ease in understanding and is presented in Table IV-10.

From this data, some generalizations about the activity of chemicals could be inferred, though the evidence was still inconclusive. In specimens for which no peak was recorded, the chemicals may act as dispersing agents, (a) increasing the basal spacings or (b) the atomic structure of chemical molecules is so large that no spacing was recorded.

Therefore, several other treatments were investigated using Claset, Clapak, and SA-1 each at 20/1000 concentration. In addition, a liquid limit test was performed using Kelpak and the A-2-4(0) soil previously noted in this study.

Specimens treated with Clapak, Claset, and SA-1 were x-rayed after drying several days and again immediately after rewetting with distilled water. The resulting basal spacings are shown in Table IV-11. Again

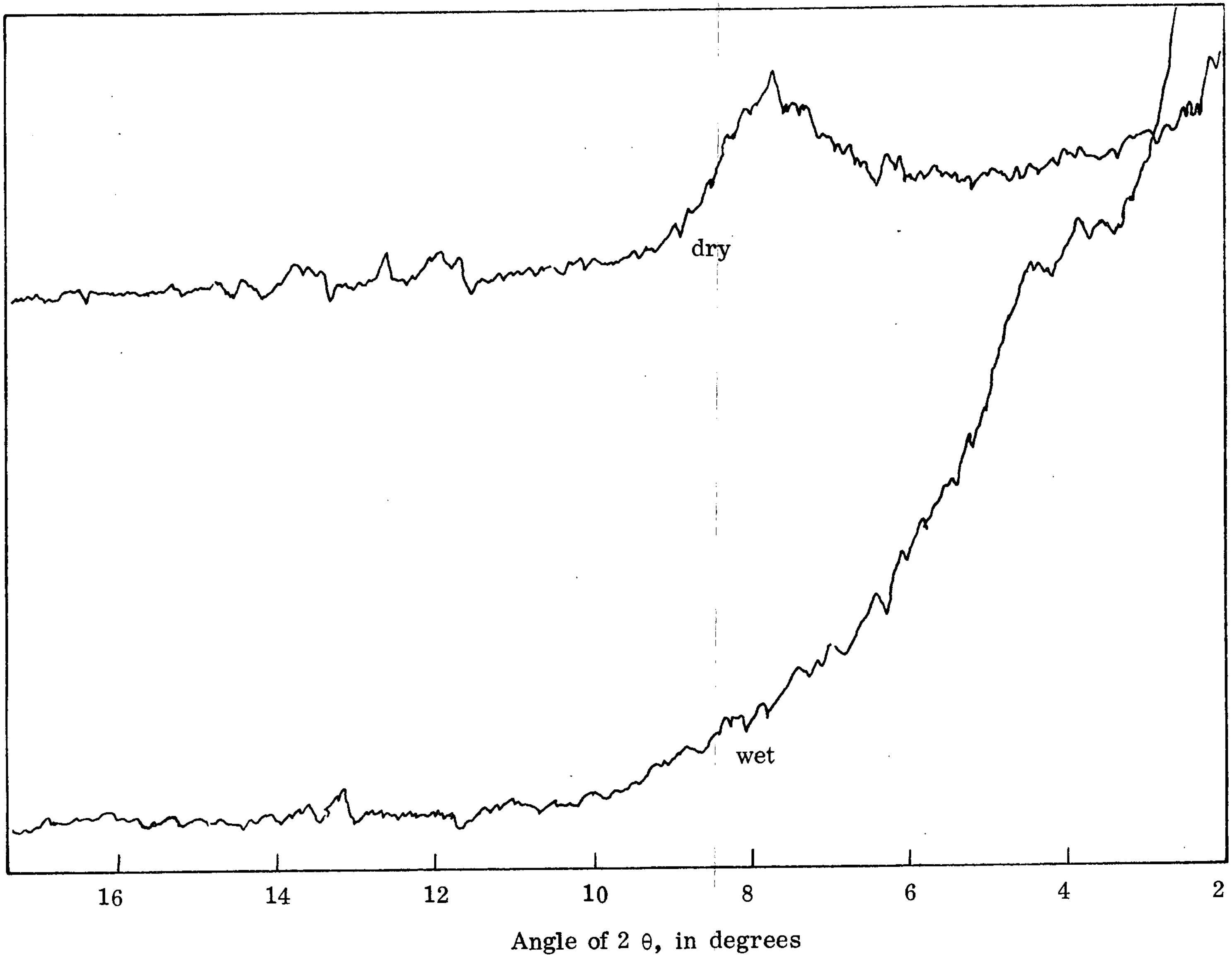


Fig. IV-43. X-ray diffraction curves for montmorillonite.

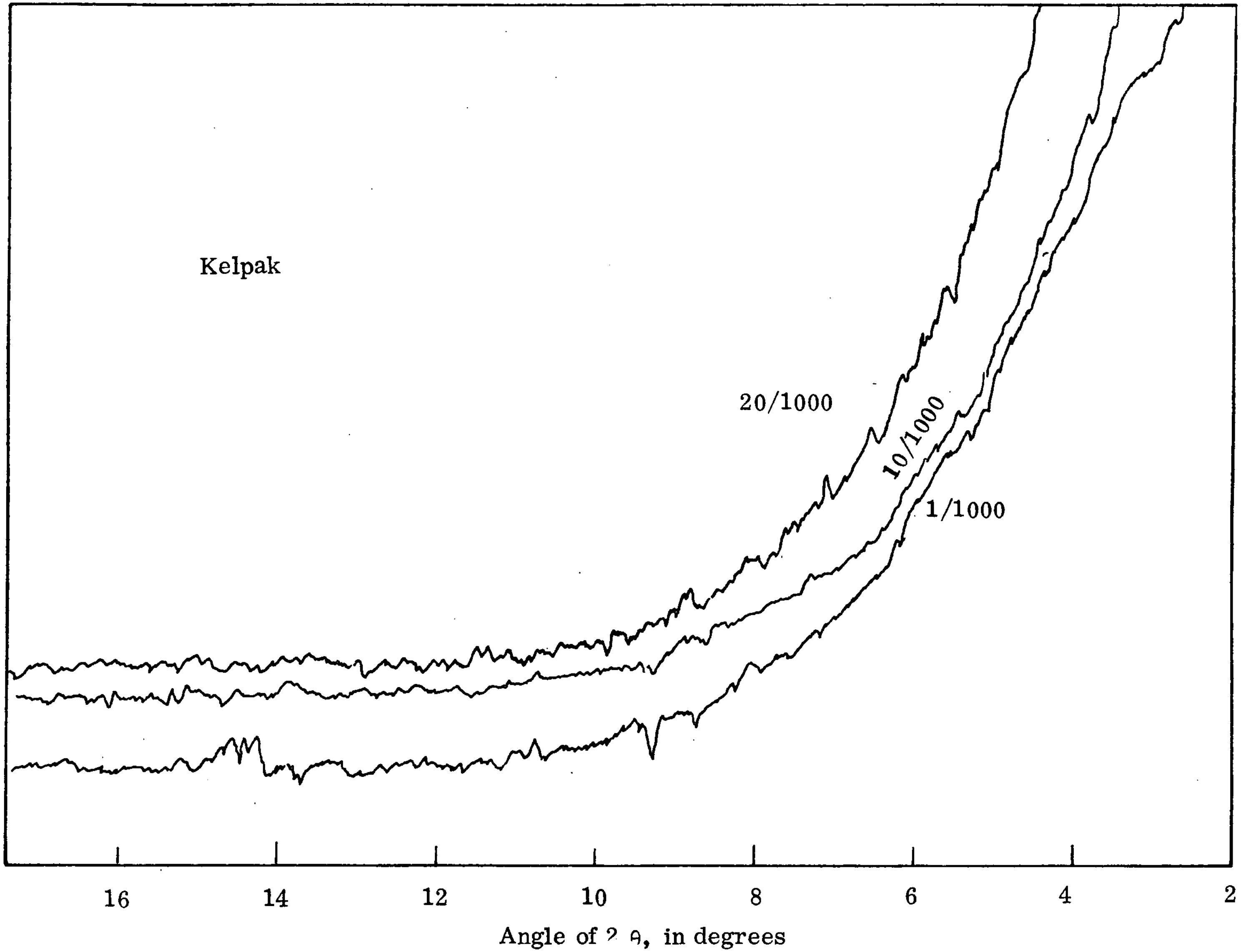


Fig. IV-44. X-ray diffraction curves for Kelpak-treated montmorillonite.

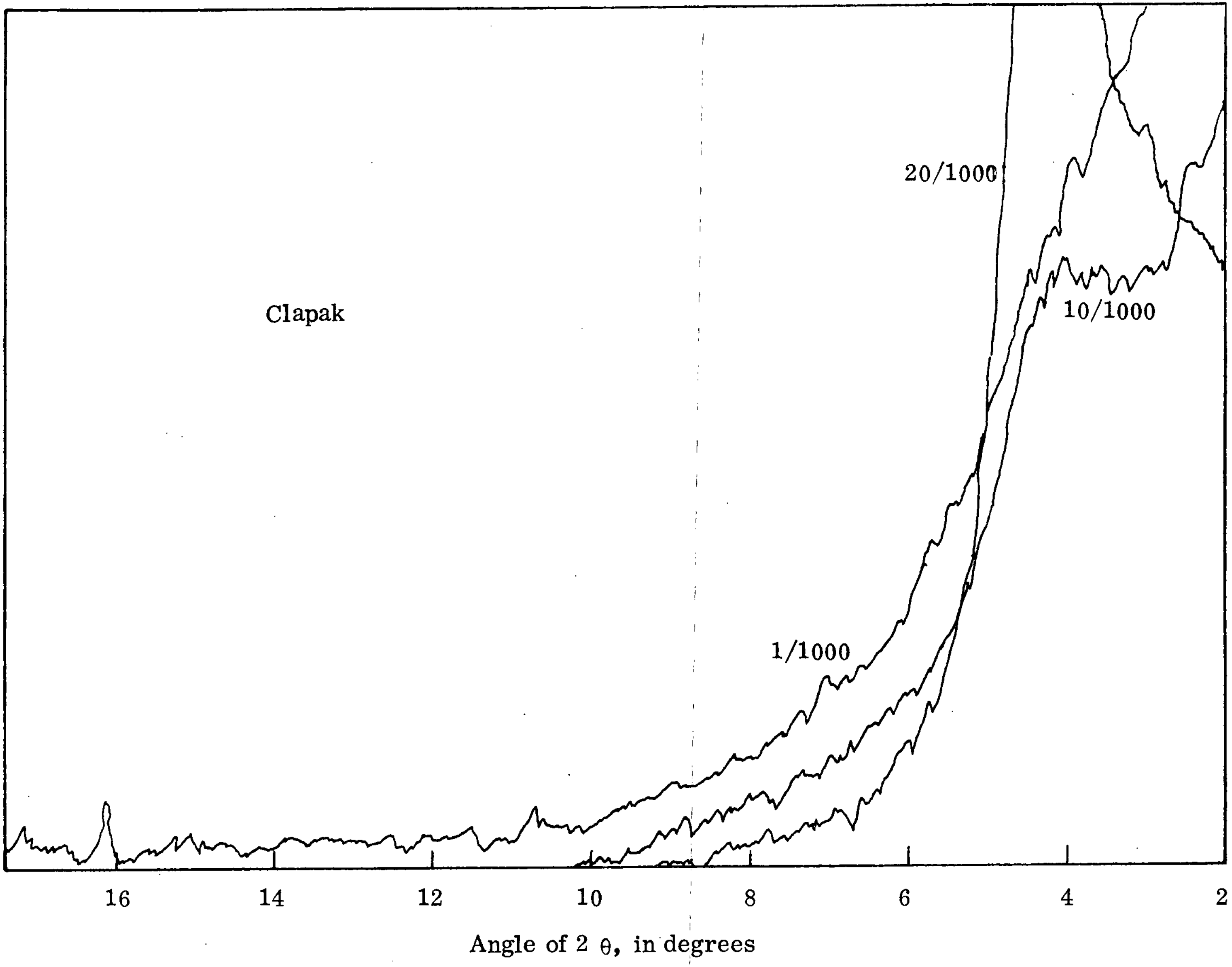


Fig. IV-45. X-ray diffraction curves for Clapak-treated montmorillonite.

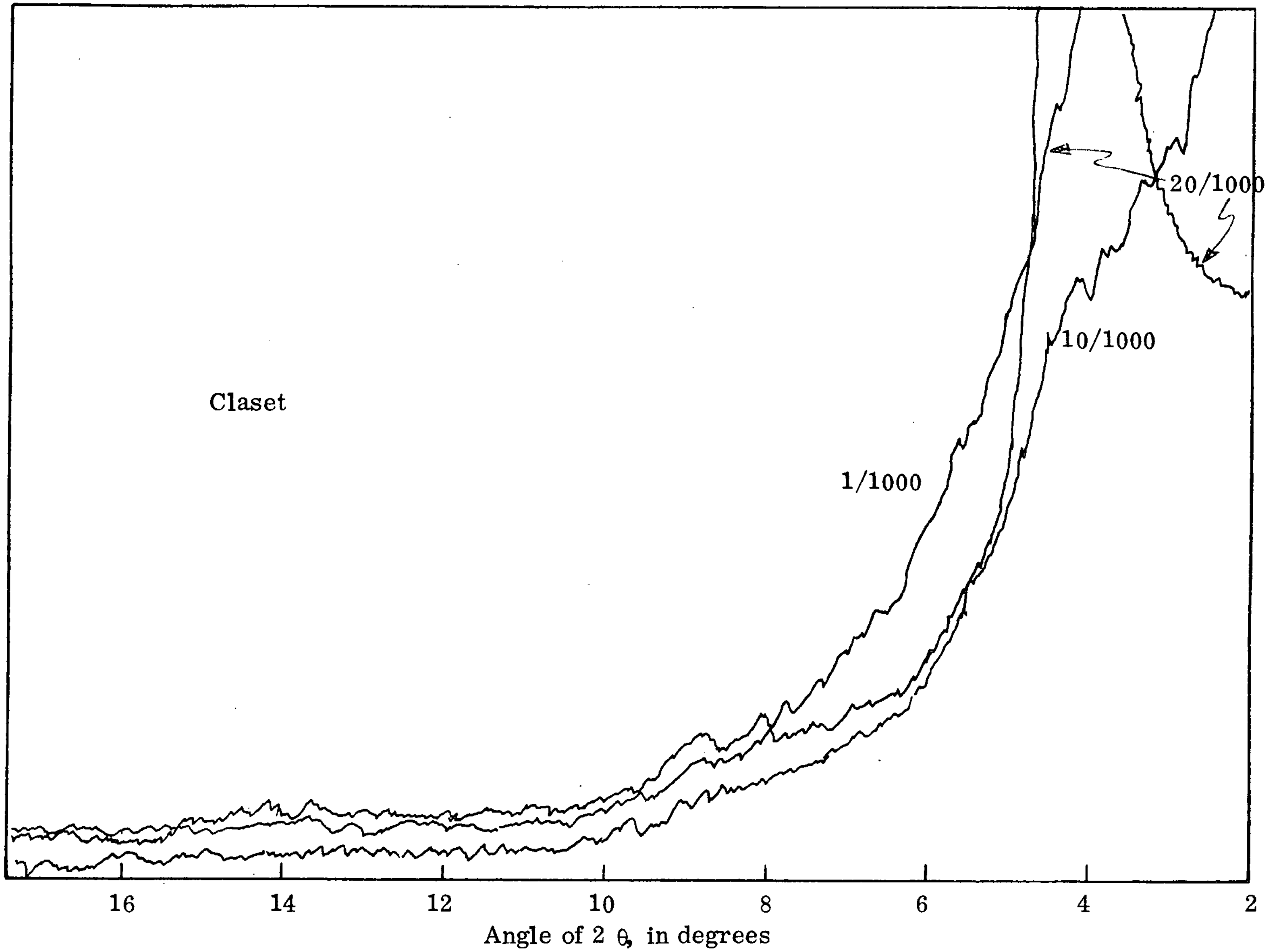


Fig. IV-46. X-ray diffraction curves for Claset-treated montmorillonite.

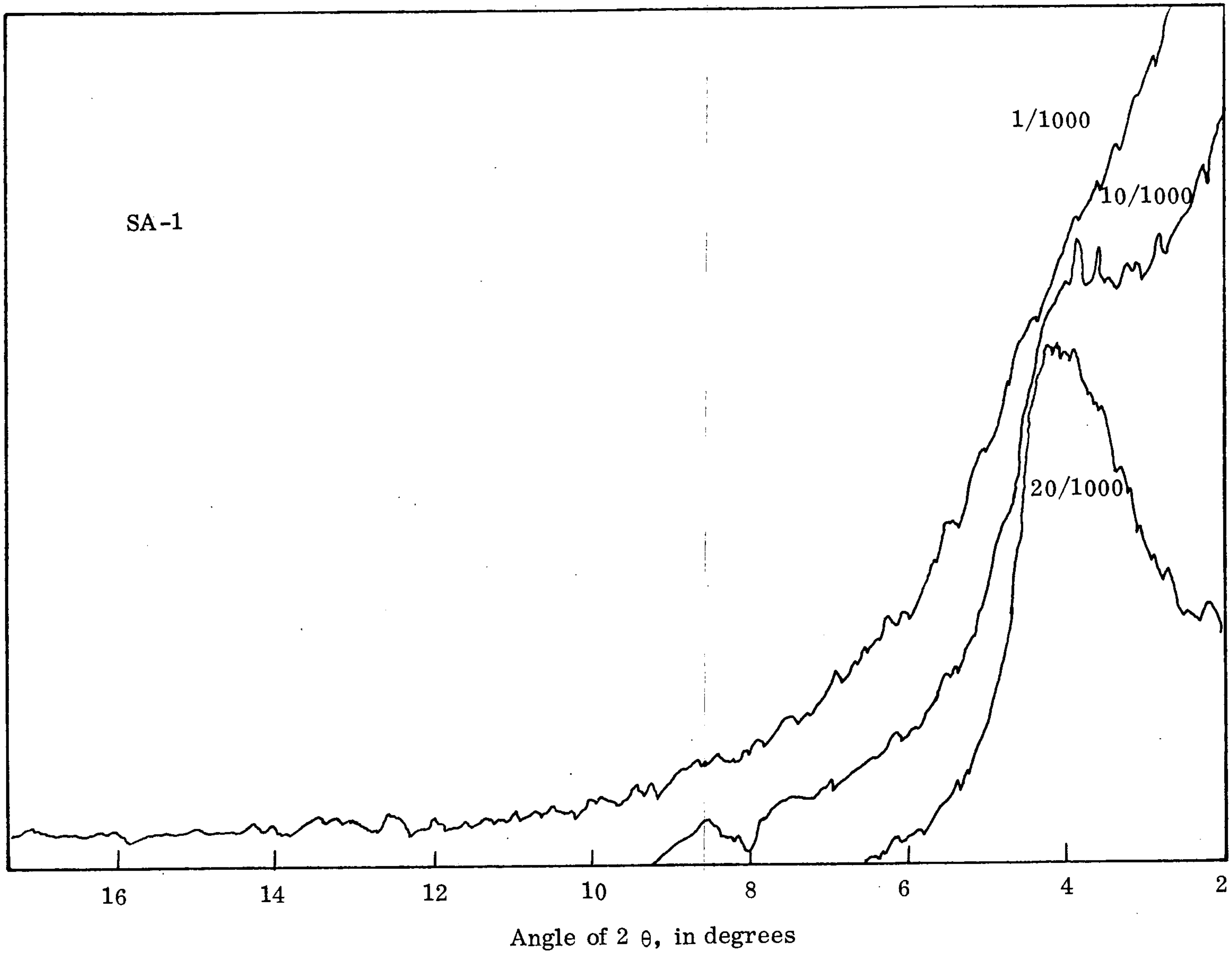


Fig. IV-47. X-ray diffraction curves for SA-1-treated montmorillonite

Table IV-10. X-ray diffraction data of montmorillonite clay.

Product	Treatment level	Basal spacing, angstroms
Untreated	0	~ 9
Kelpak	1/1000	a
	10/1000	a
	20/1000	a
Clapak	1/1000	~ 35 and ~ 21
	10/1000	~ 35 and ~ 21
	20/1000	~ 30 and ~ 21
Claset	1/1000	~ 22
	10/1000	~ 21
	20/1000	~ 19
SA-1	1/1000	a
	10/1000	~ 22
	20/1000	~ 22

[a] No peak recorded — very large d_{001} spacings.

Table IV-11. X-ray diffraction data of treated montmorillonite after drying and rewetting.

Product	Dry	Wet
Claset	~ 13	~ 20
Clapak	~ 13	~ 22
SA-1	~ 13	~ 20

the data was inconclusive due to the unknown nature of chemical and possible soil reaction. Results of the liquid limit test indicated an increase in the liquid limit of several percent upon addition of Kelpak. This, coupled with the absence of any first-order basal spacings of the montmorillonite from x-ray diffraction analysis, suggest that Kelpak may be a potential dispersant, going between the clay layers and expanding the structure. The increase in liquid limit supported this theory in that the further the structure was dispersed, the more room for water between layers and thus more water in the soil.

On the other hand, Clapak, Claset, and SA-1 may be flocculants, reducing or inhibiting expansion of the structure. As the concentrations of Claset increased, d-spacing decreased. With SA-1, the d-spacing reduced from very large (not recorded in Table IV-10) to a constant value.

Clapak definitely exhibited two first-order peaks. The 35 Å spacing could be a combination of 20 Å and 15 Å, alternately spaced, while the 30 Å spacing could be a combination of 19 Å and 11 Å, alternately spaced. Accordingly, the increased concentrations tended to inhibit expansion of the structure.

Summary of Results

1. Kelpak may possibly be a dispersing agent.
2. Clapak, Claset, and SA-1 may possibly be flocculating agents.
3. Information pertaining to actual chemical composition was severely limited, thus the result of x-ray data are inconclusive, and somewhat conjecture as to the reactions between each chemical and soil.

SUMMARY AND CONCLUSIONS

1. Untreated specimens exhibited high dry strength (1700 lb) but very poor durability as shown by erosibility and freeze-thaw tests. Trafficability results indicated that resistance to abrasion of moving loads remained good as long as the specimen stayed dry. Rut depth increased as water penetrated the specimen.

2. Specimens containing Petro D Dust exhibited the following characteristics:

    (a) Unconfined compressive strength was reduced slightly with small amounts of additive. Strength and density decreased as additive content increased although an optimum strength was clearly evident at 0.1%.

    (b) Erosibility index was reduced substantially with the addition of 0.05% Petro D Dust, from 268 to 18, and remained constant with increasing treatment.

    (c) Resistant to abrasion of a moving wheel load during the "rain" portion of traffic simulation with addition of 0.1% and 0.25% Petro D Dust.

    (d) Resistance to freezing and thawing was improved with addition of small amounts of Petro D Dust. Improvement was optimized at 0.1% concentration.

3. Specimens having a surface seal application of Petro D Dust exhibited the following characteristics:

    (a) The erosibility index was decreased and loss of material was mainly attributed to internal instability.

    (b) Surface application of Petro D Dust immediately after compaction did not allow sufficient curing as evidenced by traffic simulation tests.

    (c) Resistance to abrasion by a moving load was improved significantly, especially during "rain," by surface application following curing.

    (d) Resistance to freeze-thaw was improved with a surface application of Petro D Dust although not as much as with internal concentrations of 0.05%.

4. Specimens containing Stypol 40-5020 polyester resin exhibited the following characteristics:

    (a) Strength and density decreased as additive amounts increased although an optimum appeared at 0.5% Stypol.

    (b) Resistance to water abrasion and erosibility improved with additions of small amounts of Stypol. The Erosibility Index was reduced from 268 to 125 at 0.25%, and to 25 at 0.5% Stypol where it remained constant with increased treatment levels.

(c) Trafficability simulation indicated that 0.5% Stypol provided greater resistance to abrasion than 0.25% or 1.0%, although 0.25% approaches 0.5% at the upper end of the "rain" cycle.

(d) Resistance to freezing and thawing is improved by the addition of 0.25% Stypol, although more so by 0.5%.

5. Specimens containing Elvanol 71-30 polyvinyl alcohol exhibited the following characteristics:

(a) Strength initially decreased with small concentrations of additive but increased as additive content increased above 0.1%. Density decreased as additive content increased.

(b) The Erosibility Index decreased to below 1 gram/1000 grams, effectively zero, indicating excellent resistance to water.

(c) Resistance to abrasion of a moving load was not improved, although curing time did affect performance of trafficability specimens.

(d) Resistance to freezing and thawing was improved significantly with the addition of both 0.1% and 0.5% Elvanol 71-30, having overall composite elongations of 1.0% and 1.4% respectively.

6. Specimens containing Elvanol 52-22 polyvinyl alcohol exhibited the following characteristics:

(a) Strength decreased with initial additions but increased sharply as concentration increased. Density decreased as concentration increased.

(b) Resistance to water abrasion is vastly improved as evidenced by reduction of Erosibility Index to below 1, effectively zero.

(c) A marked decrease in trafficability occurred with specimens containing Elvanol 52-22. Rutting depth was severe even during the first 1000 dry cycles. An increase in cure time improved durability slightly.

(d) Resistance to freezing and thawing was not improved and, in fact, became worse than untreated specimens, having overall composite elongations of 6.3% and 6.2% for concentrations of 0.5% and 0.1% Elvanol 52-22, respectively.

7. Specimens containing Kelpak exhibited the following characteristics:

(a) Strength increased as additive content increased although an optimum occurred at 0.1%. After an initial decrease, density increased with additive content.

(b) Kelpak reduced the Erosibility Index significantly, optimizing at 0.1%.

(c) Kelpak appeared to control moisture in the subgrade at heights greater than 25 ft in dilutions of 20/1000 with water.

(d) X-Ray diffraction studies and liquid limit tests suggested that Kelpak acts as a dispersant.

8. Specimens containing Clapak exhibited the following characteristics:

(a) Following an initial decrease, strength increased with additive content, although an optimum was slightly evident at 0.1% Clapak. Density remained constant.

(b) The Erosibility Index was reduced by the addition of trace amounts of Clapak and decreased further as additive content increased.

(c) Clapak did not appear to control subgrade moisture in the height range investigated.

(d) X-Ray diffraction studies indicated that Clapak might be a flocculating agent.

9. Specimens containing Claset exhibited the following characteristics:

(a) Strength decreased with increased amounts of additive while density appeared to be optimized at 0.1% Claset.

(b) The Erosibility Index was reduced slightly with initial amounts of Claset but increased to over twice the untreated value as additive content increased.

(c) Claset appeared to control subgrade moisture at the water table and above 25 ft for concentrations of 1/1000, only above 25 ft for a concentration of 20/1000, and not at all for a concentration of 10/1000.

(d) X-ray diffraction studies suggested that Claset acted as a flocculating agent.

10. Specimens containing SA-1 exhibited the following characteristics:

(a) Strength and density were optimized at 0.1% SA-1 with the strength increasing 300 lb above untreated.

(b) The Erosibility Index was reduced with slight amounts of SA-1 and optimized at 0.1% SA-1.

(c) At a concentration of 10/1000, SA-1 appeared to control subgrade moisture between 25 and 70 ft above the water table.

(d) X-ray diffraction studies suggested that SA-1 might act as a flocculating agent at concentrations above 10/1000.

11. The full potential of Central Chemicals may not yet be realized and more research should be conducted. Of critical importance are the chemical compositions of these products and their corresponding reaction with soil.

12. Only an actual field application of the products investigated in this study will provide a true evaluation. The following products and treatment levels are recommended for consideration for field trials:

(a) Petro D Dust at 0.1% to 0.25% concentration.

(b) Stypol 40-5020 polyester resin at 0.5% concentration.

(c) Kelpak at concentrations ranging from 0.1% to 0.2%.

(d) SA-1 at 0.1% concentration.

Elvanol 71-30 polyvinyl alcohol, while providing excellent improvement over untreated specimens throughout the test is not recommended for field trials due to the requirement of hot mixing water and the extreme difficulty in mixing with soil.

PART V

APPENDIX

MINERALOGICAL ANALYSES OF DUST SAMPLES

by

R. L. Handy

CONTENTS

MINERALOGICAL ANALYSES OF DUST SAMPLES

Differential Thermal – Thermogravimetric Analyses

Differential thermal analyses (DTA) and simultaneous thermogravimetric analyses (TGA) were performed with a Regaku-Denki apparatus on selected dust, soil, and chemical samples. In this analysis a small powder sample (usually 40 to 60 mg) is packed into a platinum dish which is covered and heated at a linear rate from room temperature to 1000 °C. For TGA, the sample is continuously weighed on a microbalance, and for DTA, the endothermic and exothermic reactions are shown by differences in temperature between the sample and a comparison dish filled with inert material, ordinarily Al_2O_3.

Calcite

An analysis of pure calcite ($CaCO_3$) showed a large DTA endotherm accompanied by a weight loss beginning at 630 °C and ending at 910 °C. The reaction is $CaCO_3 = CaO + CO_2$, the corresponding molecular weights being $100.09 = 56.08 + 44.01$. The last figure represents the weight of CO_2 lost, a reduction of $44.01 \div 100.89 = 44\%$. This weight loss may be converted to the original weight of $CaCO_3$ by multiplying by the reciprocal of 0.44, which is 2.27. In the test of pure calcite 27.0 mg of weight was lost, which corresponds to 61.3 mg of $CaCO_3$. The original weight measured on an analytic balance was 61.1 mg, a difference of only 0.3%. The TGA test therefore should be applicable to measurement of the percent calcite to within ± 0.5%.

Lignin

Tests of a pure powdered lignin, and of a mixture of Al_2O_3 with 16.67% by weight lignin, gave an endothermic peak indicative of water loss in the range 50-200 °C, followed by an exotherm indicative of oxidation in the range 200-700 °C. The total weight lost by 700 °C exactly corresponded to the initial weight of lignin in the sample. In the diluted 16.7% lignin sample, the loss is complete by 530 °C, and with smaller amounts the reaction will probably finish at a lower temperature. The temperature of completion thus is variable but may be estimated from the end of the exothermic DTA reaction.

Clay

A test of a commercial sodium bentonite gave a large endotherm in the range 30-140 °C indicative of loss of water, and a second endotherm and weight loss in the range 500-750 °C, corresponding to the loss of clay mineral structural (OH) water. The latter can be used similar to the loss of CO_2 from calcite to calculate the amount of total clay minerals in the sample. Idealized formulas, and dry formula weights for common soil clay minerals are as follows:

Name	Ideal formula	Formula weight	OH weight loss multiplier
Montmorillonite	$(Al_{3.34}Mg_{0.66})Si_8O_{20}(OH)_4 \cdot Na_{0.66}$	734.1	20.4
Beidellite	$Al_{4.34}(Si_{6.34}Al_{1.66})O_{20}(OH)_4 \cdot Na_{0.66}$	743.2	20.6
Nontronite	$Fe_4(Si_{7.34}Al_{0.66})O_{20}(OH)_4 \cdot Na_{0.66}$	850.6	23.6
Musconite	$K_2Al_4(Si_6Al_2)O_{20}(OH)_4$	796.7	22.1
Biotite	$K_2(Mg,Fe)_6(Si_6Al_2)O_{20}(OH)_4$	929.2	25.0
Verniculite	$Mg(Mg,Fe)_6(Si_7Al)O_{20}(OH)_4$	876.4	24.3
Kaolinite	$Al_4Si_4O_{10}(OH)_8$	516.4	7.16

All minerals listed above with the exception of the last involve the partial loss of $(OH)_4$ as $2H_2O$, formula weight 36.04. Kaolinite loses twice this much, or 72.08. The weight loss thus ranges from 14% for kaolinite to 4.91% for montmorillonite to 3.88% for biotite. The reciprocals of those and the other percentages are indicated above, and multiplied times the weight loss will give the corresponding weights of the original minerals. In the test on montmorillonite the weight loss was 2.0 mg, which times 20.4 gives 40.7 mg of montmorillonite. The sample weight at 110 °C was 43.05 mg, giving a purity of 95%, a rather typical figure for a commercial bentonite clay.

Unfortunately not only are soil clays usually mixtures of various clay minerals, the loss of OH structural water from clays and related layer minerals occurs over a wide range of temperatures. This limits the utility of this method for determining clay when other materials, such as organic matter, may be losing weight in the same temperature range. In this particular case a distinction may be made from the DTA, since organic matter oxidation is exothermic and clay OH loss endothermic.

Quartz

Quartz does not undergo a weight loss but does show a small crystal inversion DTA peak at 573 °C, the area of this peak being proportional to the amount of quartz. Preliminary calibration data show that the quartz peak is considerably reduced by and affected by the other materials present. As a rough approximation, $Q = 6h + 10$, where h is the peak height in mm (0.5 uV/mm scale) and Q is the quartz weight in mg.

Summary

Based on the above considerations:

1. $CaCO_3$ will be measured by multiplying the terminal weight lost above 630 °C by 2.27. Anticipated precision, ± 0.5%.

2. Lignin or other organic matter on a dry weight basis will be measured from the weight lost from 200 to 520 °C. Precision will depend on separability of the clay reaction, and is probably in the range ± 0.5%.

3. Clay amount will be estimated by multiplying the weight lost from 520 to 630 °C by 12, recognizing that the true amount present may be higher or lower by a factor of 2, depending on the kinds of clay minerals present. Montmorillonitic and illitic minerals will be underestimated and kaolinitic minerals overestimated by this method. The separation temperature from the organic oxidation may be adjusted based on DTA evidence.

4. Quartz amount in mg will be estimated from 6 X h, where h is the DTA peak height in mm (50 uV scale). Anticipated precision, no better than ± 10%.

5. Weights will be converted to percentages on an oven-dry basis by dividing by the sample weight at 110 °C. The latter may be adjusted to compensate for drying delay due to wet samples, based on DTA evidence.

Petrographic Microscope

Analyses of the dust samples were made with a Leitz polarizing microscope with a Rosiwal six-spindle integrating stage. In this method a small amount of sample is mounted in immersion oil on a glass slide and the slide moved in a series of linear traverses under the microscope. Each grain encountered is visually identified by a skilled petrographer, and traversed using the micrometer spindle assigned to this mineral. Upon completion of traverses of total length exceeding 100 times the maximum grain diameter, the spindles are read and summed, and readings converted into volumetric percentages. Expression as a weight percentage may be made by multiplying by corresponding mineral specific gravities. Accuracy and reliability of the method depends on the skill and experience of the petrographer and on the type of sample. Samples containing clays are difficult to accurately analyze because of the occurrence of clay as coatings and aggregates.

Minerals identified include the following:

Quartz	SiO_2	G = 2.65
Calcite	$CaCO_3$	2.72
Feldspars	Var.	2.60
Clays	Var.	2.70
Organic matter	Var.	2.00

Since most specific gravities are within a rather small range, the conversion to weight percentage was not made.

RESULTS AND DISCUSSION

Data from the compositional analyses of dust samples are summarized in Table V-1.

Table V-1. Compositions of dust samples.

Section and treatment	Sample No.	Distance from C_L, ft	TGA, wt %			Microscope, vol %		
			Carbonates	Clay minerals	Organic water	Carbonates	Clay aggregates	Organic matter
Clinton, Chemplex	1	50S	23.4	12	6.2	30.3	33.1	10.8
	2	50N	17.2	11	8.3	20.2	29.3	16.5
Clinton, Lignin	3	75N	42	30	14	42.6	30.0	8.2
	4	100N	49	34	9	20.3	35.5	13.3
	5	125N	49	30	11	38.5	28.8	9.5
Clinton, Lignin + lime	6	25W	26	6	3	19.6	27.8	8.2
	7	50W	21	19	6	20.6	32.6	6.0
Linn, No treatment	W8	350	57	16	6	56.3	25.6	4.1
	W7	300	50	23	13	64.3	23.9	3.6
	W6	250	48	31	11	60.3	21.5	7.1
	W5	200	56	23	10	59.7	24.0	6.5
	W4	150	54	29	10	67.9	18.3	4.2
	W3	100	51	30	12	61.1	24.0	3.2
	W2	50	74	17	6	69.8	13.7	4.8
	W1	12	75	11	3	73.1	13.1	2.9
	E1	12	75	11	2	71.2	14.6	3.2
	E2	50	66	20	5	66.4	19.3	4.5
	E3	100	58	15	8	62.4	23.4	4.6
	E4	150	55	28	9	64.6	25.8	4.2
	E5	200	55	22	9	62.2	24.6	5.3
	E6	250	38	40	13	62.6	22.2	3.6
	E7	300	51	32	11	58.0	28.9	4.5
	E8	350	30	27	14	57.7	30.0	3.9

Table V-1. Continued.

Section and treatment	Sample No.	Distance from C_L, ft	TGA, wt % Carbonates	TGA, wt % Clay minerals	TGA, wt % Organic water	Microscope, vol % Carbonates	Microscope, vol % Clay aggregates	Microscope, vol % Organic matter
Poweshiek No treatment	N10-N7	Missing	a	a	a	a	a	a
	N6	200				29.1	31.9	13.8
	N5	100				42.3	29.6	6.0
	N4	50				52.0	24.4	6.3
	N3	36				63.3	18.4	2.6
	N2	26				63.2	12.7	3.5
	N1	16.5				62.7	11.8	2.7
	S1	16.5				57.8	18.0	5.5
	S2	34				a	a	a
	S3	50				50.8	20.9	4.1
	S4	75				49.6	20.3	4.6
	S5	100				36.4	38.7	11.1
	S6	200				36.2	34.1	12.2
	S7	300				43.8	32.4	11.0
	S8	400				29.7	33.2	22.2
	S9	500				24.5	41.1	22.3
	S10	900	a	a	a	2.2	41.0	50.4
MTR 2, 400 T/M, 1% lignin	W3	28.0	1.0	19	8.4	0.8	46.3	2.9
	W2	20.3	0.9	13	5.0	0.7	46.1	3.2
	W1	12.5	1.6	10	3.9	1.0	43.0	2.3
	E1	14.5	2.6	11	2.4	1.0	45.7	2.6
	E2	23.0	0.7	14	5.4	0.6	42.6	5.0
	E3	31.0	a	a	a	1.3	47.7	3.1
MTR 3, 400 T/M	3R	Road surf.	27.4	28	3.6	35.1	26.8	3.5
	W3	30.9	25.6	10	3.6	14.8	36.7	2.2
	W2	21.2	25.9	8	3.6	18.8	28.2	1.9
	W1	11.5	23.6	7	1.9	16.2	34.4	3.8
	E1	13.0	30	8	1.9	18.6	33.1	2.1
	E2	23.0	30.5	8	2.4	22.2	25.5	2.9
	E3	33.0	35.8	10	2.5	22.7	29.0	2.7

Table V-1. Continued.

Section and treatment	Sample No.	Distance from C_L, ft	TGA, wt %			Microscope, vol %		
			Carbonates	Clay minerals	Organic water	Carbonates	Clay aggregates	Organic matter
MTR 5, 400 T/M, 1% lignin, 0.5% lime	W3	31.4	1.3	20	6.3	1.0	38.6	3.8
	W2	21.2	1.5	21	5.2	1.9	32.4	2.9
	W1	11.0	2.4	13	3.0	1.7	37.2	3.7
	E1	12.5	1.3	15	2.6	3.2	40.8	4.4
	E2	23.4	0.9	15	2.6	3.0	27.5	4.4
	E3	32.3	1.7	16	5.0	2.6	32.6	4.6
MTR 6, 1000 T/M, 1% lignin	W3	31.0	0.6	12	15.4	1.7	35.9	4.6
	W2	22.0	0.9	10	6.6	2.1	25.2	6.3
	W1	13.0	3.2	24	4.9	3.8	29.0	10.1
	E1	14.5	6.7	27	3.8	1.8	25.9	4.5
	E2	22.5	7.3	33	6.2	1.4	31.4	4.3
	E3	30.5	a	a	a	1.4	27.3	6.2
MTR 8, 1000 T/M, 1% lignin, 0.5% lime	W4	60.0	1.3	21	4.7	3.8	30.1	3.9
	W3	44.0	1.0	16	5.1	5.4	33.1	4.7
	W2	28.0	2.8	12	4.3	5.6	36.2	3.7
	W1	12.0	7.7	11	3.1	9.7	31.9	3.5
	E1	14.5	5.0	11	2.8	4.7	33.5	3.6
	E2	23.0	1.4	14	3.1	6.3	34.4	5.0
	E3	32.0	1.4	23	5.7	5.5	34.8	4.8
MTR 9, 1800 T/M	9L	Road surf.	88.1	6.2	0.3	87.9	5.7	1.7
MTR 10, 1800 T/M, 1% lignin	W3	30.5	a	a	a	18.7	28.2	6.9
	W2	21.0	14.8	22	1.8	26.0	26.2	3.6
	W1	11.5	32.7	20	3.8	46.7	19.2	3.7
	E1	13.0	a	a	a	30.3	23.3	3.9
	E2	22.0	19.2	25	5.6	21.1	29.7	5.1
	E3	32.0	a		a	15.7	31.8	5.4

Table V-1. Continued.

Section and treatment	Sample No.	Distance from C_L, ft	TGA, wt %			Microscope, vol %		
			Carbonates	Clay minerals	Organic water	Carbonates	Clay aggregates	Organic matter
MTR 12, 1800 T/M, 1% lignin, 0.5% lime	W3	30.0	15.2	22	5.5	22.7	36.5	6.3
	W2	21.0	27.4	15	5.5	33.9	32.0	5.4
	W1	11.5	39.2	14	3.1	47.2	26.2	4.2
	E1	13.0	43.0	11	2.7	46.9	23.6	6.0
	E2	22.0	[a]	[a]	[a]	[a]	[a]	[a]
	E3	32.0	17.0	46	9.3	31.5	34.7	7.0

[a] Sample insufficient for analysis.

Comparisons of Methods

Carbonates

A plot of carbonate contents obtained by microscopic analyses versus those from TGA is shown in Fig. V-1. The microscopic and TGA analyses usually agree to ± 10%, neither being consistently higher or lower than the other. The agreement therefore is rather good considering the

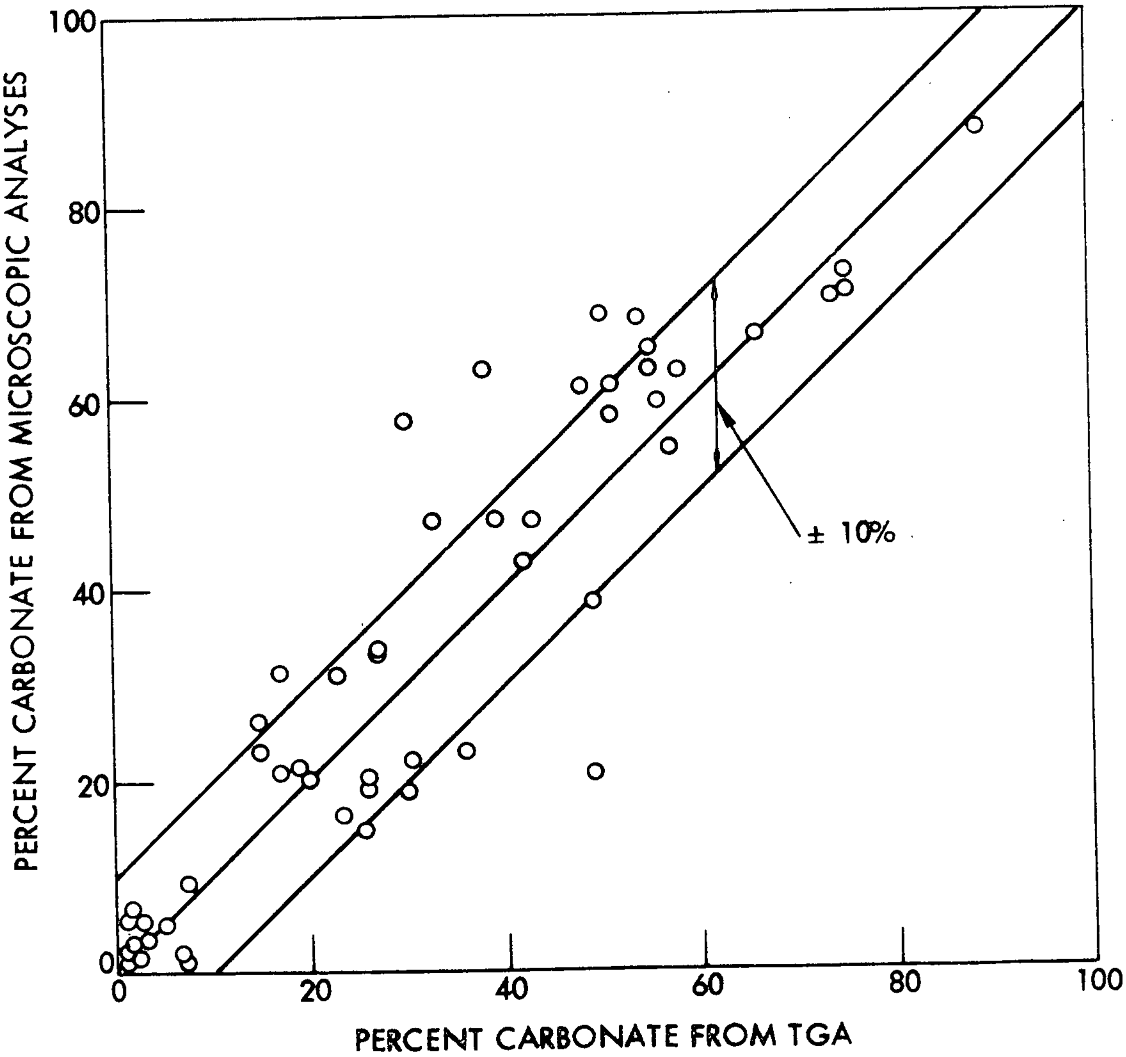


Fig. V-1. Microscopic versus thermogravimetric analyses for carbonates.

difficulties in microscopic identification of these fine powders, particularly when single grains contain more than one component, as was frequently the case. Since as previously discussed the TGA carbonate determination is rather precise, Fig. V-1 suggests that other mineral percentages obtained microscopically also may deviate from the true values by ± 10%.

A few DTA's showed the presence of dolomite, $Ca{\cdot}Mg(CO_3)_2$, molecular weight 184.42 which reduces to 96.4 upon loss of CO_2, a reduction of 47.73%. This gives a multiplier of 2.095 instead of 2.27, and thus a dolomite percentage will be overestimated 8.3% if reported as calcite. This was not a significant factor in most of the analyses.

Organic Matter

Organic matter contents could not be precisely measured with the microscope, but tended to be very roughly the same as obtained from DTA-TGA analyses.

Clays

Two problems are involved in determining clay mineral percentages from the TGA weight loss associated with the 500-700 °C endotherm. First, as previously indicated, the result may be off by a factor of two due to the unknown mineral composition. Secondly, the weight loss involved may overlap simultaneous weight losses due to oxidation of organic matter up to 520 °C, and due to loss of CO_2 from carbonates above 630 °C. The clay range was therefore arbitrarily defined to be between these two temperatures unless the DTA indicated one or both should be altered. Part of the clay reaction thus may be incorrectly assigned to organic matter or carbonate measurements, but this was believed preferable since the clay weight loss is small compared to those other weight losses. Furthermore any weight loss incorrectly assigned to clay would be multiplied by 12, in effect multiplying the clay content error by the same factor.

An appreciable error also is present in identification of clays by the microscope, since clay aggregates inevitably contain nonclay minerals, and carbonate grains almost inevitably contain clay. A graph for clay content similar to Fig. V-1 shows that agreement of the methods is poor. The TGA data for clay minerals, while not accurate, do show consistent trends which also appear in the TGA carbonate and organic matter data. These trends are less evident in the microscope data, and the TGA was therefore preferred.

Quartz

The quartz DTA data are very approximate. The microscope data also are approximate, and the correlation between the two data sets is poor.

Data Trends

Untreated Roads

The Linn County road dust analyses, Fig. V-2, shown decreasing carbonate contents, roughly 75% down to 40%, with distance east from the road. Simultaneously, the organic matter and clay contents tended to increase. These trends also occurred but are less consistent to the west. Two possibilities exist to explain the observed trends: 1. sorting action due to selective sedimentation from air, smaller and lighter particles being carried farther before settling, and 2. contamination

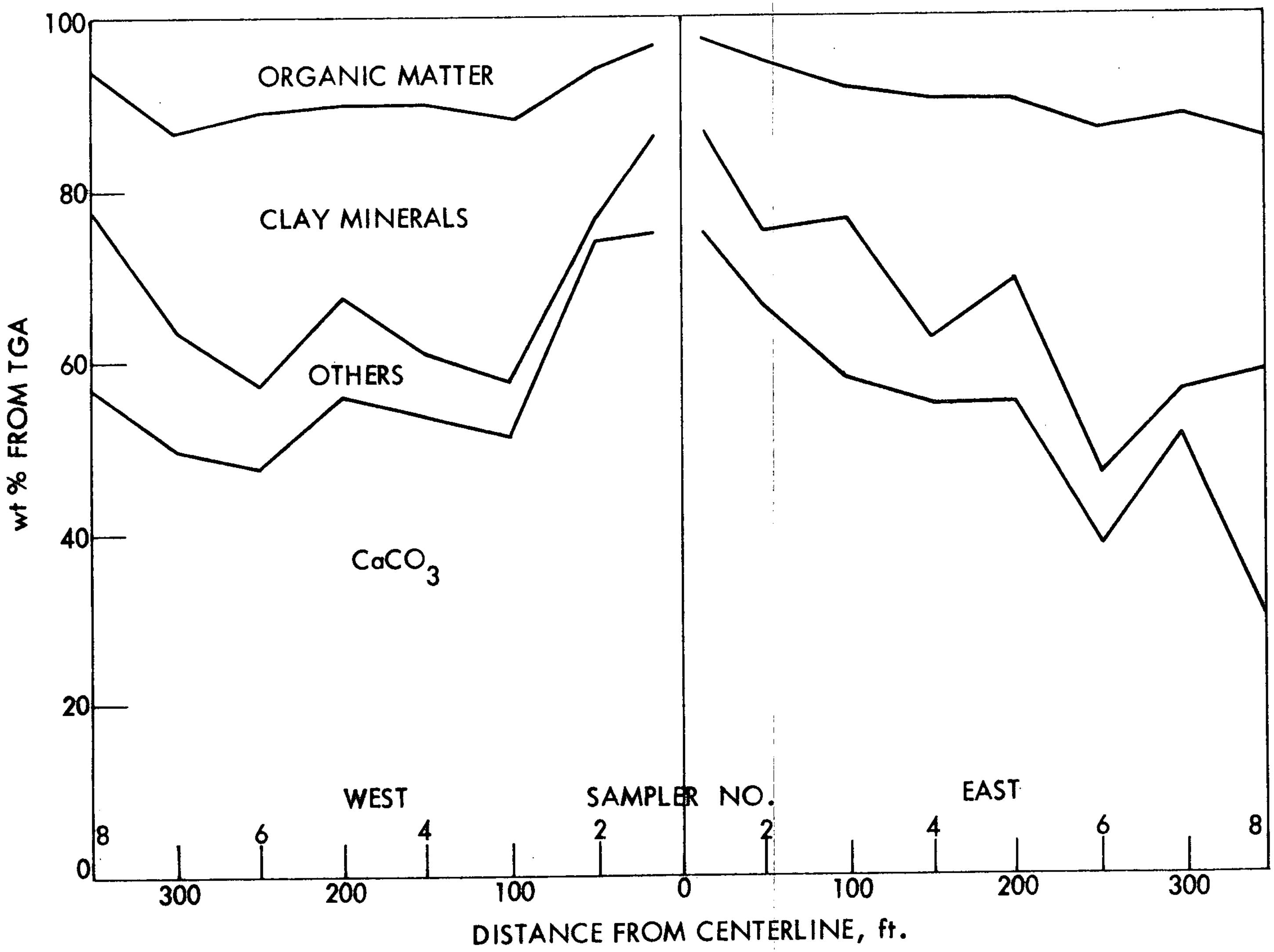


Fig. V-2. Dust compositions from Linn County, Iowa, road (untreated).

from other dust sources. The latter would ideally add the same amount of contaminant to each sampler, and the composition of the contaminant should be relatively constant. The Linn County road-dust samplers set to the east were in a pasture, where contamination should be slight. The samplers to the west ran parallel to and south of a farm lane; contamination from this source would have the greatest compositional effect farthest from the main county road source, i.e., at the west-end samplers. This argument is consistent with higher carbonate percentages shown at the left in Fig. V-2, apparently reflecting contamination.

Marion test road section 3 (MTR 3) had the addition and mixing in of 400 tons/mile of crushed rock with no other additives. Unlike Fig. V-2, the carbonate contents of the dust samples showed little change, but clay and organic matter contents showed a slight tendency to increase with distance. However, in this and other MTR sections the maximum dust sampler distance from the road centerline was only 33 ft, about one-tenth as far as for the Linn sampler traverse.

Samplers for the Poweshiek County road were set as far as 1000 ft from the road, and many samplers had too little dust for analysis. Microscopic studies had to be relied on because of the small amounts of some samples. Results are shown in Fig. V-3, and show a 30-fold reduction in carbonate content with distance from the road, accompanied by increases in clay and organic matter contents. A local source of contamination is suggested near sampler 7, which was 300 ft south from the road. The trends in Fig. V-3 confirm that a selective sorting of mineral species does occur with distance from a dusting road.

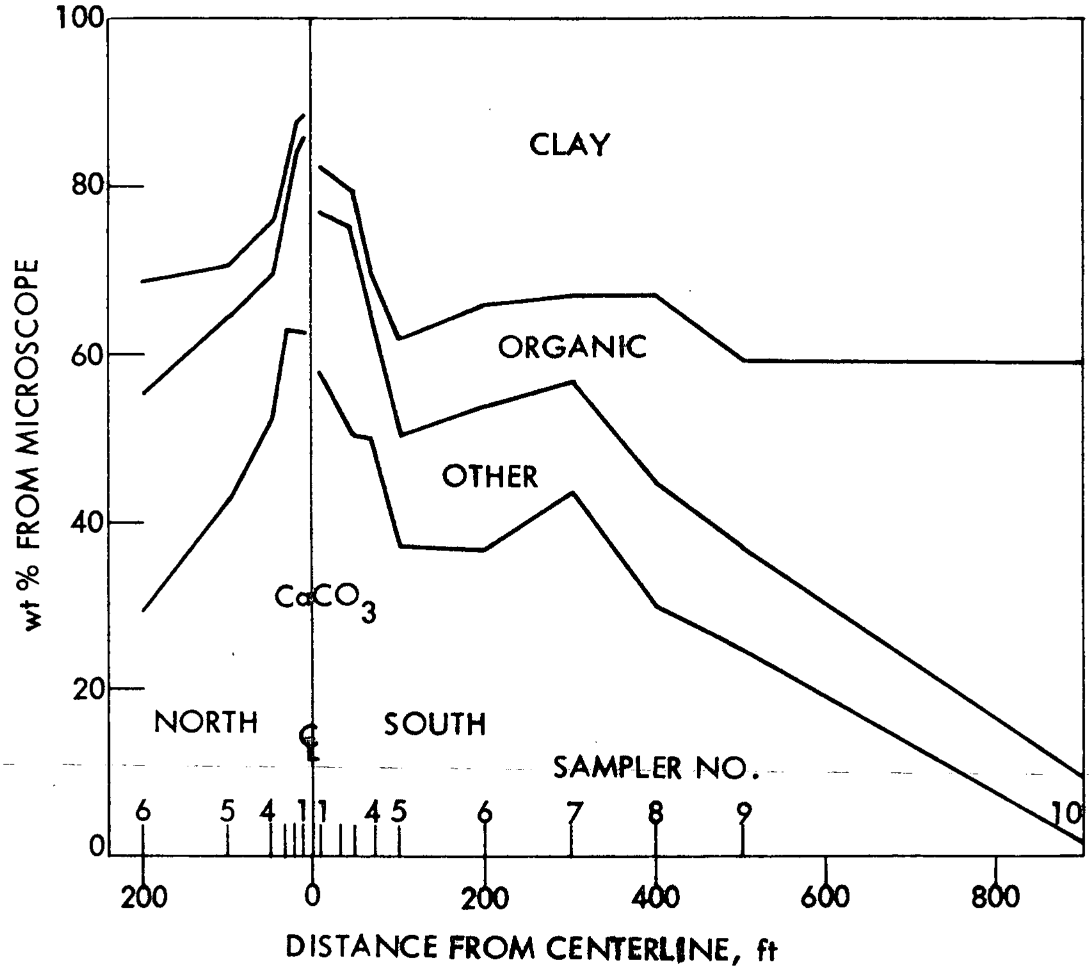


Fig. V-3. Dust composition from Poweshiek County, Iowa, road (untreated).

Treated 400 T/Mile Roads

MTR 2 is similar to MTR 3, with rock added at 400 T/mile, but with the additional additive of 1% lignin. This had a dramatic effect on carbonate contents (Fig. V-4), reducing them by a factor of 10 or more. The only sample with appreciable carbonate was that to the east immediately adjacent to the road. Clay and organic contents are higher in dust from the lignin-treated road, and show a tendency to increase away from the road.

MTR 5 is similar to MTR 2 and 3, but with 1% lignin plus 0.5% lime. Results are practically the same as with 1% lignin alone.

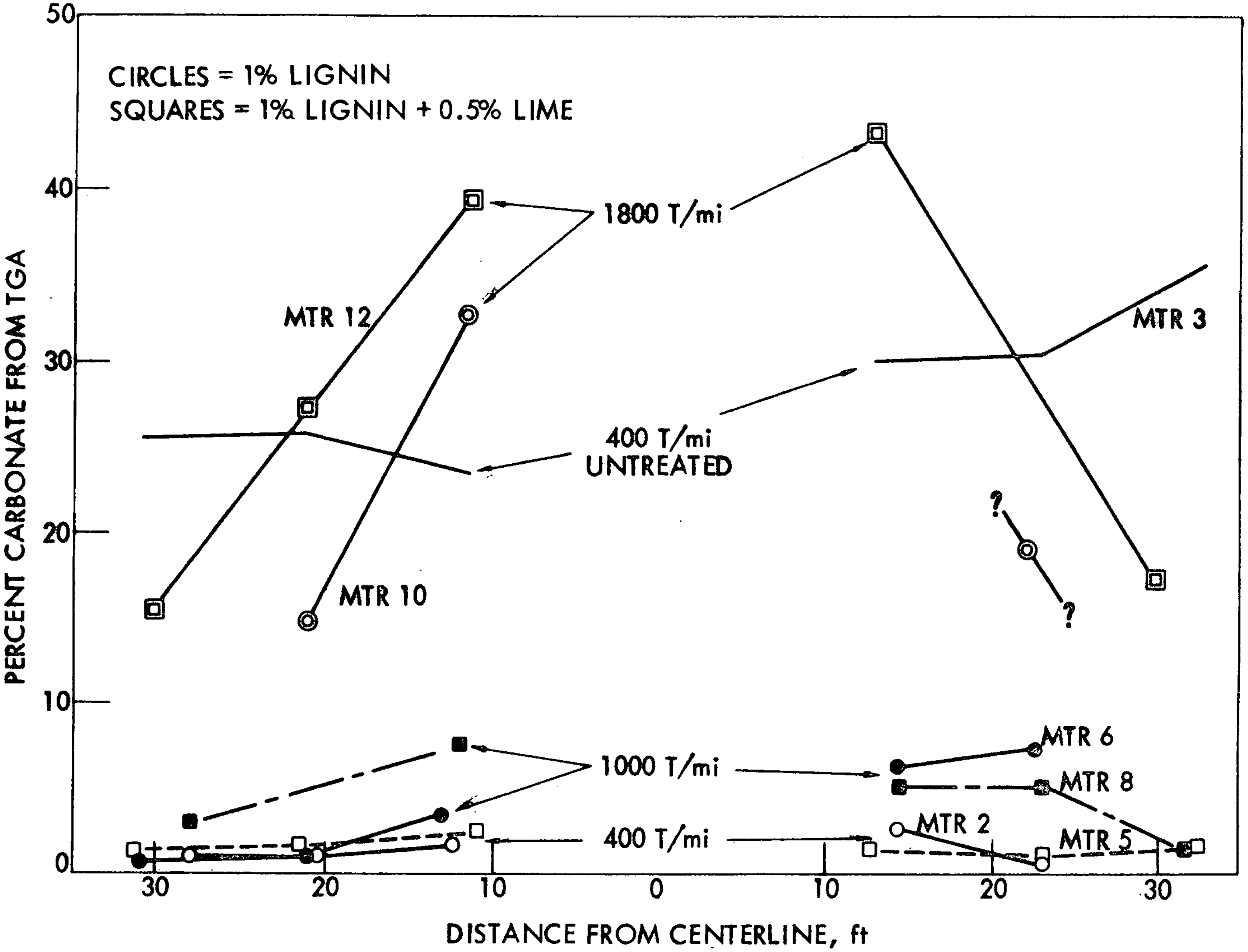


Fig. V-4. Effects of chemical treatments and added rock amount on dust carbonate.

Higher Rock Application Roads

MTR 6, with 1000 T/mile crushed rock plus 1% lignin, showed up to 7% carbonate in the dust samples, indicative that the lignin percentage was insufficient to prevent limestone dusting with this higher rate of rock application. This is confirmed by samples from MTR 10, 1800 T/mile plus 1% lignin, where carbonate contents in the dust ran from 15 to 32%.

The addition of lignin plus lime, MTR 8, gave essentially the same results as with lignin alone when the rock application rate was 1000 T/mile, but resulted significantly higher carbonate contents when the rate was 1800 T/mile, in MTR 12. In this case hydrated lime appears to have had a deleterious effect on carbonate dust control with lignin.

Clinton County Test Road

Dust samples were obtained adjacent to three treated sections of the Clinton test road. Unfortunately the soils varied between the sections, and no untreated sections were included for controls. Furthermore many samplers were lost, so no conclusions can be drawn in regard to composition. Data are given in Table V-1.

Significance of Lignin Treatment

The marked reduction in carbonate percentage of dust samples as a result of lignin treatment is not accompanied by an equal reduction in weight of dust obtained, as previously presented. In fact, the amounts of noncarbonate dust collected along the untreated section, MTR 3, only slightly exceed noncarbonate dusts obtained from the comparison treated sections, MTR 2 and 5. Thus the role of lignin in the 400 T/mile rock sections is primarily to prevent the loss of limestone dust. This could occur by an upward migration of lignin and clay during drying, giving a protective patina. The dust samples were taken almost a year after construction, after sufficient time to generate such a surface film. If this postulated mechanism of formation is correct, the film would be expected to renew itself and help keep the road intact so long as sufficient leachable lignin remains in the road. At rock application rates of 1000 T/mile and 1800 T/mile the effect appears to have been weakened, but the degree cannot be precisely evaluated due to the lack of no-chemical sections with these rock amounts.

CONCLUSIONS

With Regard to Methodology

1. Thermogravimetric analysis offers a rapid, accurate method for the measurement of carbonates and organic matter in 50 mg dust samples, and gives a reliable semi-quantitative indication of the amount of clay minerals.

2. Analysis by petrographic microscope is less accurate but requires less sample, and therefore is a valuable supplemental technique for very small samples.

With Regard to Untreated Roads

3. Dust analyses show conclusively that selective sorting is operative, calcium carbonate dominating in dust samples collected near the road, and gradually decreasing to near zero several hundred feet away from the road. Simultaneously, the contents of clay and organic matter increase with distance from the road. The efficiency of the sorting indicates that it is size-selective rather than specific

gravity-selective (i.e. carbonate contents are higher close to the road because the grains are larger and settle faster).

With Regard to Chemical Treatments

4. Lignin treatment will drastically reduce the carbonate percentage in road dust, by more than a factor of 10, provided the added rock amount does not exceed 1000 T/mile. The total amount of dust is less drastically reduced, and the amount roughly coincides with the amounts of noncarbonate components in dusts from untreated roads. This suggests that lignin and clay build up a protective patina at the road surface, by migration upward due to evaporation. This patina protects the limestone and is worn off by traffic; however it should renew so long as sufficient lignin remains in the road. (Alternatively it probably can be renewed by surface treatments.) The existence of such a coating, while greatly reducing the limestone aggregate loss from the dry road, also may contribute some slipperiness on a wet road, and probably acts to reduce bonding between the road metal and a bituminous overlay.

5. Rock added in excess of about 1000 T/mile is not effectively protected by a 1% lignin treatment, or lignin plus lime. In addition the higher rock application levels, added $Ca(OH)_2$ lime tends to further reduce the protective effect on carbonates, probably by flocculating the clay and reducing migration and development of a film.

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