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Bridge over the James River, connecting the north and south sections of the Blue Ridge Parkway, between U.S. 460 and Virginia State Route 130.

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IN THIS ISSUE

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Correlation of Compaction and Classification Test Data of Soils

BY THE OFFICE OF RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported[1] by GEORGE W. RING, III, and JOHN R. SALLBERG, Highway Research Engineers, Physical Research Division, and WEBSTER H. COLLINS, Bridge Engineer, Development Division

This article presents the results of two studies undertaken by the Physical Research Division, Bureau of Public Roads, to correlate the results of laboratory compaction tests with the results of classification tests. The correlations developed in these studies have been proved useful for rapidly checking compaction test results and for denoting unusual soil characteristics that might cause construction difficulties.

Laboratory compaction tests are employed to provide data on maximum dry density and optimum moisture content of soils; results of such tests are used widely in the development of requirements for earthwork compaction. Classification tests consist of grain-size analysis and the determination of plastic and liquid limits. In the two correlation studies described here, the compaction tests were performed in accordance with an AASHO test method; the grain-size analyses were extended to include the 0.001 millimeter size; and the moisture contents, denoting plastic and liquid limits of the soil, were determined in the standard manner.

The first study, to correlate compaction and classification test results was conducted mainly by plotting maximum dry densities and optimum moisture contents against plastic and liquid limits to arithmetic scale. In the second study, multiple linear regression analyses were used; this permitted correlation of compaction test results with several characteristics of the grain-size analysis, as well as with the plastic and liquid limits.

The formulas developed during these studies, incorporating the various factors for estimating compaction tests results, appear to be more reliable for a wide variety of soils than any previously published.

Introduction

AMONG the soil tests required for controlling the quality of highway construction, the compaction test is one of the most important and one of the most time-consuming. A need exists for (1) shortening the time required to perform laboratory compaction tests, and (2) developing interrelationships between compaction test data and other laboratory test data to increase the basic knowledge of compaction. The results of two studies conducted by the Bureau of Public Roads to accomplish these two objectives are reported in this article.

Summary

The optimum moisture content and maximum dry density obtained in compaction tests are known generally to be related to the plasticity and gradation of the soil material. This article shows the relationships of optimum moisture content and maximum dry density—as determined in the test, *The Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop* (AASHO Designation: T 99–57, Method A)—to each of several plasticity and gradation characteristics and to two or more plasticity and gradation characteristics jointly.

The multiple correlations, based on two or more plasticity or gradation characteristics, provided several methods for predicting optimum moisture content and maximum dry density. The simplest of these prediction methods seems to be better than those previously published when a variety of soils from large geographical areas are being considered. The best correlations of compaction data with classification data were obtained when the analyses were made on test data from only one county, which was the smallest geographical unit considered.

Predicted optimum moisture content and maximum dry density are useful for several purposes: (1) determining the amount of water to use in the compaction test for the first moisture-density point; (2) rapidly appraising compaction test results when the classification data are readily available; (3) reducing the number of compaction tests required in areas where the prediction methods have been proved to be sufficiently accurate for construction control purposes, and (4) denoting unusual soils that are different from those generally encountered and that may cause construction difficulties.

Published Correlations

In 1938, Woods and Litehiser (*1*)[2] showed the general interrelation of plastic limit, plasticity index, liquid limit, optimum moisture content, and maximum dry density for 1,367 Ohio soils. They reported that increases in the plastic properties of the soils were accompanied by increases in optimum moisture content and decreases in maximum dry density. In a more recent report, Jumikis (*2*) presented a chart relating optimum moisture content obtained from ". . . standard soil compaction tests . . ." with liquid limit and plasticity index for various New Jersey glacial soils. This chart is shown in figure 1.

Rowan and Graham (*3*) presented two formulas for estimating the maximum dry density and the optimum moisture content as determined in the Proctor test.[3] These formulas are, as follows:

[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 86.

[3] The details of the compaction test used by Rowan and Graham were not given. In the original Proctor test (*4*), 25 firm 12-in. strokes of a 5.5-lb tamper were used on each of three layers of soil in a mold about 4 inches in diameter and 5 inches in height.

Table 1.—Summary of deviations of optimum moisture contents estimated by the PL and LL chart (fig. 4) from those determined by test

State	Predominant soil type (origin) [1]	No. of samples for which the estimated optimum moisture content was less than the test result by amount indicated												No. of samples for which the estimated optimum moisture content exceeded the test result by amount indicated					
		−11	−10	−9	−8	−7	−6	−5	−4	−3	−2	−1	0	+1	+2	+3	+4	+5	+6
Alabama	Residual	----	----	----	----	----	----	----	----	----	----	3	16	12	1	----	----	----	---
Arizona	do	----	----	----	----	----	1	1	2	1	----	----	2	----	----	----	----	1	---
Arkansas	Recent alluvium	----	----	----	----	----	----	----	1	1	2	4	10	8	2	----	----	----	---
Connecticut	Glacial	----	----	----	----	----	----	----	1	1	1	4	7	2	2	1	----	----	---
Florida	Coastal plain sand	----	----	----	----	----	----	----	----	1	1	1	3	4	1	----	----	----	---
Idaho	Non-soil	1	1	2	----	----	4	5	4	4	6	11	20	9	2	----	3	----	1
Illinois	Loessial	----	----	----	----	----	----	----	----	----	1	10	12	5	----	----	----	----	---
Kentucky	Residual	----	----	----	----	----	----	----	----	1	----	6	18	4	1	----	----	----	---
Maryland	do	----	----	----	----	----	----	----	----	----	2	3	6	7	3	1	2	----	---
Minnesota	Glacial	----	----	----	----	----	----	----	2	4	4	4	3	2	1	----	----	----	---
Nebraska	Outwash	----	----	----	----	----	----	1	1	7	7	7	15	7	4	1	1	----	---
Nebraska	Loessial	----	----	----	----	----	----	----	----	2	4	13	8	----	1	----	----	----	----
Nevada	Non-soil	----	----	1	1	2	1	1	2	1	----	----	----	----	----	----	----	----	---
New Mexico	Residual	----	----	----	----	----	----	----	----	----	1	2	4	1	2	----	----	----	---
North Carolina	do	----	----	----	----	----	----	----	----	----	1	3	5	5	1	2	----	1	---
North Dakota	Glacial	----	----	----	----	----	----	1	2	2	2	2	4	4	2	----	----	----	---
Oregon	Non-soil	----	----	----	----	1	2	2	1	1	1	2	7	----	3	----	----	----	---
Tennessee	Residual	----	----	----	----	----	----	1	3	----	10	11	7	6	----	----	----	1	---
Texas	Coastal plain clay	----	----	----	----	----	----	----	----	----	----	2	7	1	2	----	1	----	---
Texas	Residual	----	----	----	----	----	----	----	1	1	2	----	----	1	2	----	----	----	---
Vermont	Glacial	----	----	----	----	----	----	1	----	1	----	3	4	7	----	----	----	----	---
Ohio	Lacustrine	----	----	----	----	----	----	----	1	----	1	1	2	2	----	----	----	----	---
Totals		1	1	3	1	3	8	13	21	28	46	92	160	87	30	5	7	3	1

[1]See reference *10*.

Calculated density, pounds per cubic foot[4] =

$$\frac{D}{1+\dfrac{D-C}{62.5\ G_s}} \qquad (1)$$

Calculated optimum moisture, percent=

$$SL\left(\frac{B}{A}\right) \qquad (2)$$

Where,

$D=\dfrac{CA}{B}$

$C=62.5 \times$ shrinkage ratio, p.c.f.

A=percentage passing No. 4 sieve

B=percentage passing No. 40 sieve

G_s=specific gravity

SL=shrinkage limit, percent.

In comparing the predicted maximum dry densities and optimum moisture contents with test results for 10 soils, Rowan and Graham (*3*) found that the greatest difference between the predicted and the test maximum dry densities was about 5 percent, and that the predicted optimum moisture contents were slightly higher, from about 1 to 5 percentage points, than the test results. They suggested that all calculated (predicted) optimum moisture contents be corrected by subtracting 3 percentage points.

Davidson and Gardiner (*5*) used the Rowan-Graham formulas in an analysis of test data for 210 soils from 11 States. They

[4] Pounds per cubic foot hereafter indicated by p.c.f.

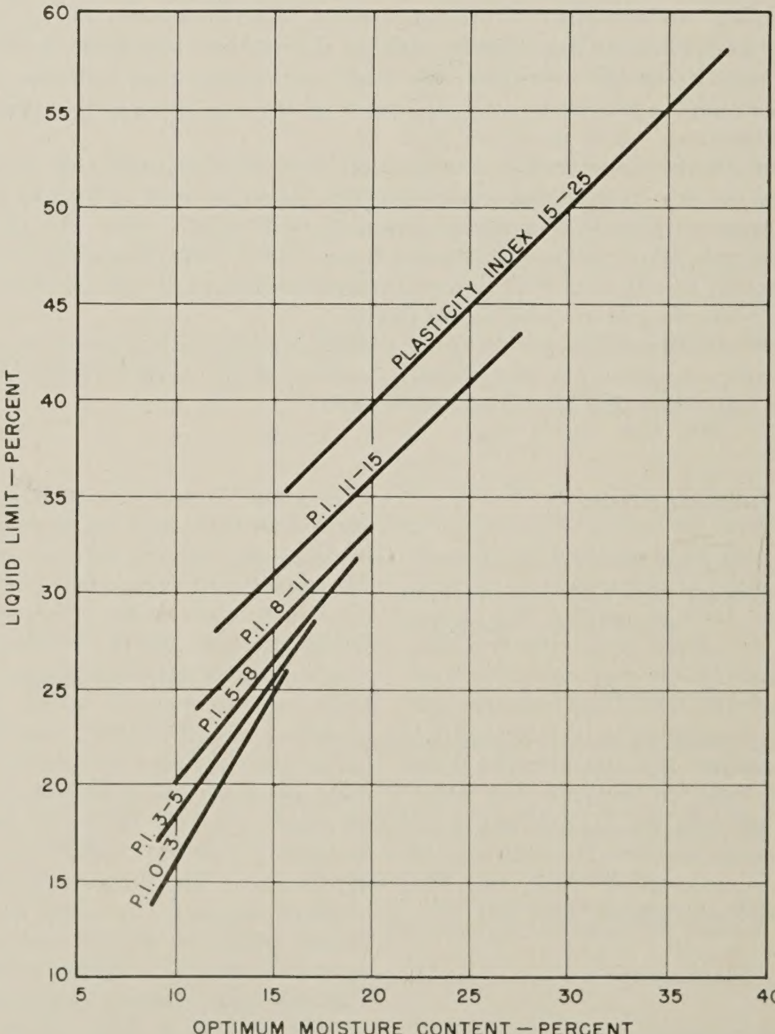

Figure 1.—Optimum moisture content and liquid limit related to plasticity index of various glacial materials (2).

Table 2.—Comparisons of optimum moisture contents and maximum dry densities estimated by PL and LL chart (fig. 4) with those determined by test[1] for soils from Alaska and outside the continental United States

Soil		Optimum moisture content			Maximum dry density		
Location	Kind	Test	Estimated	Deviation	Test	Estimated	Deviation
		Percent	*Percent*	*Percent*	*P.c.f.*	*P.c.f.*	*P.c.f.*
Alajuela, Costa Rica	Lateritic	41	22	−19	77	99	+22
Las Lomas, Panama	do	29	19	−10	88	104	+16
Liberia	do	20	24	+ 4	107	95	−12
Do	do	12	14	+ 2	126	114	−12
Do	do	15	17	+ 2	117	108	− 9
Addis Ababa, Ethiopia	Black Cotton	39	29	−10	80	87	+ 7
Oahu, Hawaii	Kaolinitic	33	22	−11	85	99	+14
Do	do	36	23	−13	80	97	+17
Do	Montmorillonitic	37	30	− 7	79	86	+ 7
Do	do	35	27	− 8	84	90	+ 6
Kodiak, Alaska	Silty alluvium	20	21	+ 1	103	100	− 3
Do	do	24	23	− 1	95	97	+ 2
Do	do	33	30	− 3	80	86	+ 6
Kenai-Kasilof, Alaska	do	10	10	0	131	125	− 6
Do	do	14	14	0	118	114	− 4
Do	do	13	12	− 1	124	119	− 5

[1] Data from Public Roads laboratory except for Hawaiian soils, which are from Kawana and Homes (*11*).

found wide deviations between the predicted and test results obtained from ". . . the standard Proctor control tests . . ." on highly plastic soils. They determined that the size of the deviation was related to the plasticity index of the soil and revised the Rowan-Graham formulas to fit these data more closely. The revised formulas are, as follows:

Calculated density, p.c.f.=

$$\frac{6250\ K_1}{SL\left(\frac{B}{A}-1\right)+\frac{100}{R}} \qquad (3)$$

Calculated optimum moisture, percent=

$$SL\left(\frac{B}{A}\right)+K_2 \qquad (4)$$

Where,

$K_1=\frac{312-2(PI)}{300}$

PI=plasticity index

R =shrinkage ratio

$K_2=\frac{PI}{3}-4.$

Formula (3) does not include the specific gravity term (G_s), which appears in formula (1); Rowan and Graham calculated specific gravity from shrinkage test data, whereas Davidson and Gardiner substituted the shrinkage data directly into the formula.

Turnbull (*6*) of Australia showed that the optimum moisture content is closely related to the gradation of the soil. For a numerical measure of gradation, he used the area above the graph of the grain-size distribution curve and named it the classification area (*7*). The solid, curved line in figure 2 shows the relationship of classification area to optimum moisture content for 101 soils tested by a compaction method very similar to the AASHO T 99–57, Method A. (The compaction effort that Turnbull used was 15 percent greater than the AASHO effort.) The simple curvilinear relationship shown in figure 2 fits the test data for optimum moisture content very closely; 72 percent of the predicted optimum moisture contents were within 1.0 percentage point of the test results. Additional tests made later by Turnbull, on coarse-grained soils with high classification areas, indicated that the curve in figure 2 should be corrected to follow the dotted line shown in the figure. Information on these tests was transmitted to the authors by a letter from Mr. Turnbull dated Oct. 10, 1961.

To simplify the determination of the classification area, Turnbull subdivided the grain-size distribution chart by equally spaced ordinates; figure 3 is an adaptation of Turnbull's chart. The original chart by Turnbull extends to the right five more units to include particle sizes up to 6 inches. The extra five

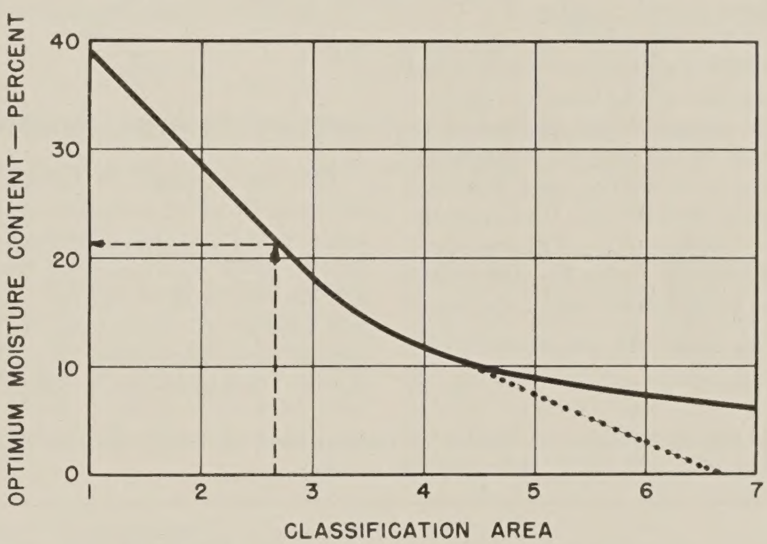

Figure 2.—Optimum moisture content vs. classification area based on test data from 101 soils (6). The dotted line shows position of corrected curve based on additional tests of coarse material.

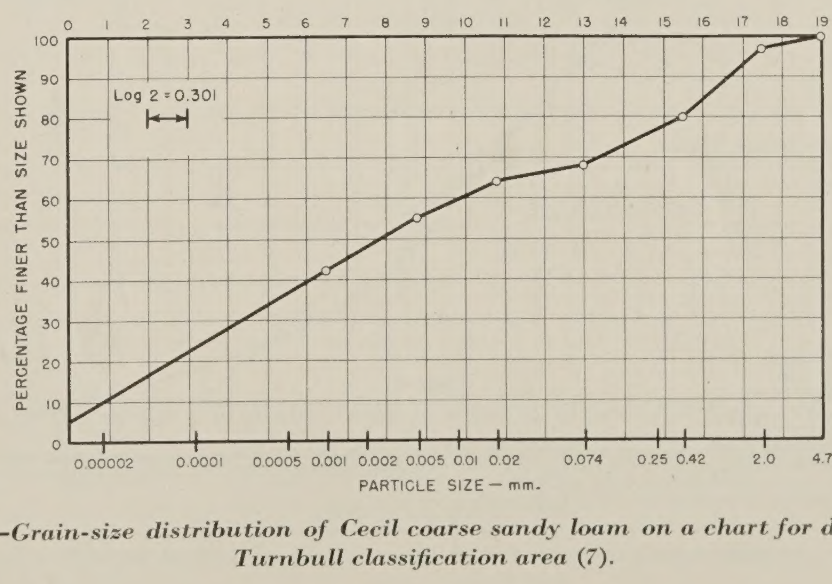

Figure 3.—Grain-size distribution of Cecil coarse sandy loam on a chart for determining Turnbull classification area (7).

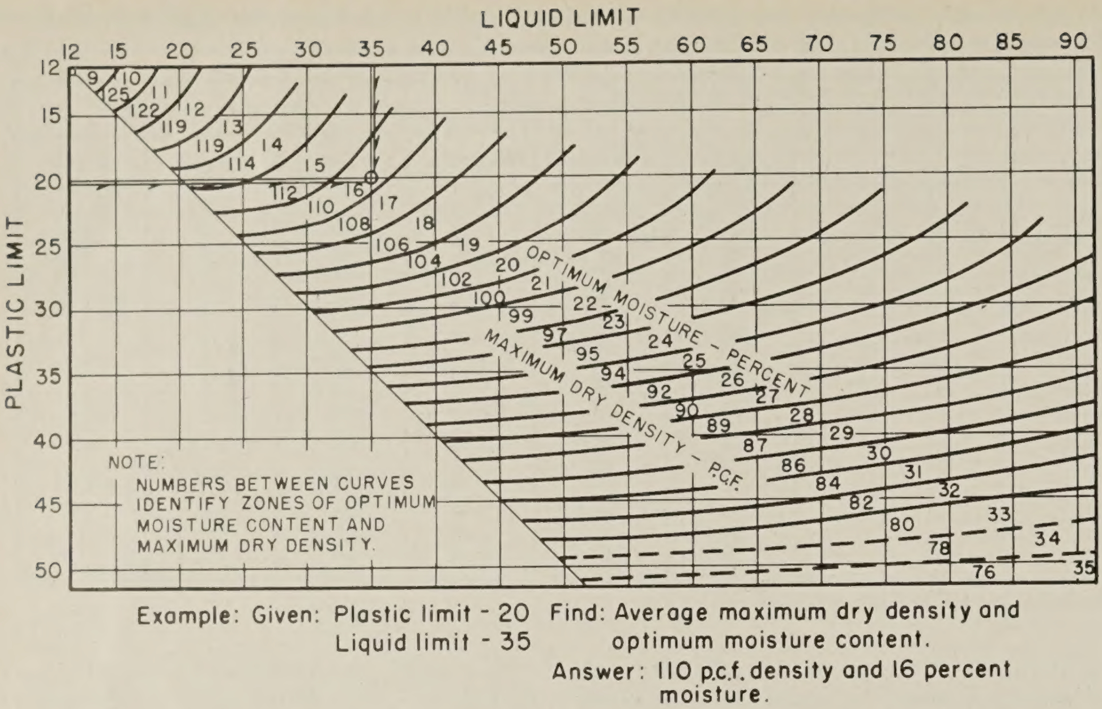

Figure 4.—Relation of average maximum dry density and optimum moisture content to plastic limit and liquid limit.

units were not needed in the Public Roads study because the maximum size of particles used in the compaction test was 4.7 mm. (about three-sixteenths of an inch).

To determine the classification area from the grain-size distribution curve of a given soil material, one-half of the length in percent of ordinate zero above the curve should be added to the sum of the lengths of ordinates 1 through 19 above the curve, and that sum should be multiplied by 0.00301. For example, the gradation of a sample of Cecil coarse sandy loam has been plotted on figure 3. The length of the ordinates to be added above the curve are $\frac{95}{2}$, 89, 84, 78, 72, 67, 61, 55, 50, 44, 40, 36, 34, 32, 27, 23, 16, 8, 2, and 0. The sum of these, 866, when multiplied by 0.00301 yields a classification area of 2.61. By locating the point corresponding to this area in figure 2, the optimum moisture content is determined to be 22 percent.

FIRST PUBLIC ROADS STUDY

The first attempt, in 1958, by the Bureau of Public Roads to correlate optimum moisture content and maximum dry density with classification data was based on test data of 972 soil samples from 31 States. The compaction test used in the first study, as well as in the second, was performed in accordance with AASHO Designation: T 99–49, which is the same as the current AASHO Designation: T 99–57, Method A. In this method, the soil fraction passing the No. 4 sieve is compacted in three layers in a 4-in. diameter mold by dropping a 5.5-lb. rammer from a height of 12 inches, 25 times per layer.

Correlations were made by plotting the test data on rectangular coordinates. The chart shown in figure 4, developed by Yemington (*8*) in the first study, correlates optimum moisture content and maximum dry density with plastic limit and liquid limit. To evaluate the chart, it was used to estimate optimum moisture contents for 510 additional soil samples from a number of States. These estimates were compared to test results; these comparisons are given in table 1. The comparisons show that 81 percent of the predicted optimum moisture contents were within 2 percentage points of the test optimum moisture contents. The correlation was best for eastern soils; 94 percent of the predicted optimum moisture contents for the 222 samples from east of the Mississippi River were within 2 percentage points of the test results. Soils showing the least correlation were from non-soil areas west of the Mississippi River. To evaluate estimates of maximum dry density from the chart (fig. 4), a study was made of test data from 532 samples, which included the 510. Sixty-three percent of the estimates were within 4 p.c.f. of the corresponding test results.

Another appraisal of the chart was made with test data for soil samples obtained from Alaska and places outside the continental United States. The comparisons, given in table 2, ranged from reasonably good for soils from Alaska to extremely poor for those from Costa Rica, Panama, and Hawaii. Whereas, the data clearly show that wide variations occur between the predicted and the actual test results of optimum moisture contents and maximum dry densities for certain soils, the variety of soils studies was too limited to warrant conclusions as to the cause of the variations.

The results of Public Roads' first study proved to be very useful. The chart, figure 4, has been used for several years as a guide in performing compaction tests, particularly to estimate the amount of water to use for the first moisture-density point. It also has been used by the laboratory supervisor to determine whether the optimum moisture contents obtained by technicians were reasonable. The use of the chart is limited, however in that it does not fit a large number of unusual soils. Further analysis was needed to make the correlations applicable to a wider range of soils and to make the estimates more accurate.

SECOND PUBLIC ROADS STUDY

To improve the methods for predicting optimum moisture content and maximum dry density, Public Roads made a second study in 1961 using multiple linear regression analysis. This method of analysis permitted all of several variables to be used jointly for estimating optimum moisture content and maximum dry density.

Table 3.—Correlation between variables as determined by inspection of data plotted on rectangular coordinates

Correlation	Rating [1] of correlation			Relationship on arithmetic scale		Relationship on log log scale	
	Good	Fair	Poor	Linear	Curved	Linear	Curved
Optimum moisture content versus:							
LL	X	------	------	X	------	X	------
PL	X	------	------	X	------	X	------
PI	------	X	------	X	------	------	X
Range	------	------	X	------	------	------	------
D'_{50}	------	X	------	------	X	X	------
FA	------	X	------	------	X	------	X
0.001	------	X	------	X	------	------	X
Maximum dry density versus:							
O.M.C	X	------	------	------	X	------	X
LL	------	X	------	X	------	------	X
PL	X	------	------	X	------	------	X
PI	------	X	------	------	------	------	------
Range	------	------	X	------	------	------	------
D'_{50}	------	X	------	------	------	------	------
FA	------	X	------	------	X	------	X
0.001	------	X	------	------	------	------	------

Legend:
LL, Liquid limit.
PL, Plastic limit.
PI, Plasticity index.
Range, Number of log cycles traversed by the straight line approximating the D_{10} to D_{90} portion of the grain-size distribution curve.
D'_{50}, The average particle size, determined at the midpoint of the straight line referred to in Range definition.
FA, Fineness average, equal to one-sixth of the sum of percentages of particles finer than the following sizes in millimeters: 2.0, 0.42, 0.074, 0.020, 0.005 and 0.001.
0.001, Percentage of particles finer than 0.001 millimeter.

[1] Ratings were based on the degree of scatter of the plotted points about the line of best fit.

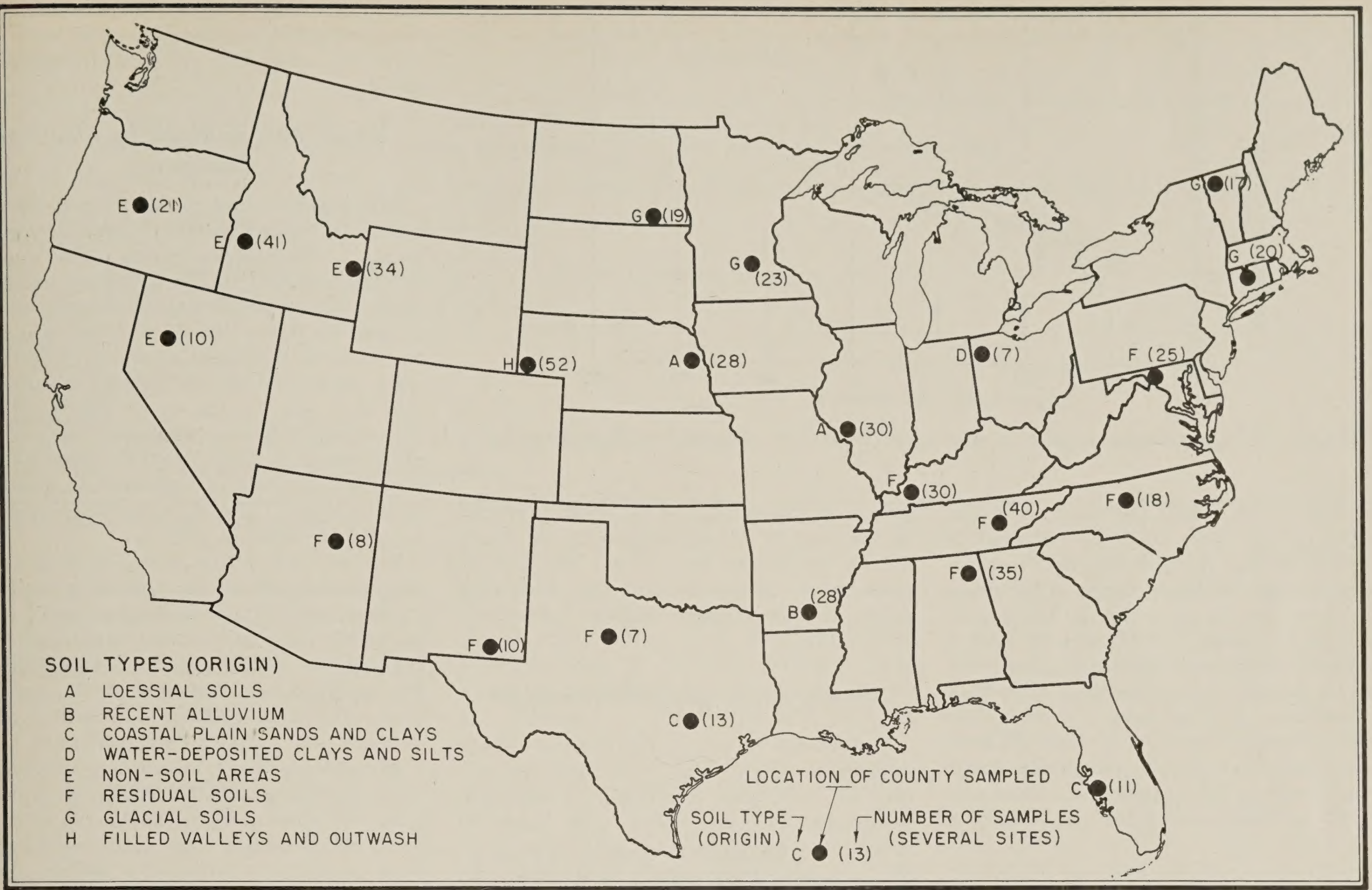

Figure 5.—Primary soil type (origin), location of county samples, and number of samples.

Selection of Samples and Variables

Soil test data, to represent a broad coverage of soils within the continental United States, were selected from the files on the basis of the geographical and geological origin of the soil samples. Initially, 946 samples were selected; many were the same as those used in the first study. This number was reduced to 600 by the use of a set of random numbers. The nonplastic soils were eliminated after preliminary analyses of the test data had shown them to have different interrelationships from those of the plastic soils. The analyses were made on the remaining 527 samples of plastic soil. The general types (origins) of soils represented, their sampling locations, and the number of samples from each location are shown in figure 5.

The independent variables used in the analyses included plastic limit, liquid limit, plasticity index, and several measures of gradation. Specific gravity also was considered but was not used because of insufficient data. Standard AASHO tests were used in determining the plasticity and gradation of the soils; gradation was determined on the fraction passing the No. 4 sieve because that was the fraction used in the compaction test, AASHO T 99–57, Method A. Gradation, as represented by percentages passing specific sieves, could not be used as an independent variable because the regression type of analysis requires that each variable be expressible or measurable by a single number. To represent characteristics of gradation, several measures were tried. Burmister (*9*) reported, "The significant characteristics of grain-size distribution are fineness, range of grain size, and type of grading." All of these characteristics were evaluated in the second Public Roads study.

Three measures were devised for fineness, as follows: (1) Percentage of particles finer than the 0.001-mm. size was designated as 0.001 fraction. (2) Fineness average, FA, was determined by taking one-sixth of the total of the percentages of particles, by weight, finer than the following listed sizes in millimeters: 2.0 (No. 10), 0.42 (No. 40), 0.074

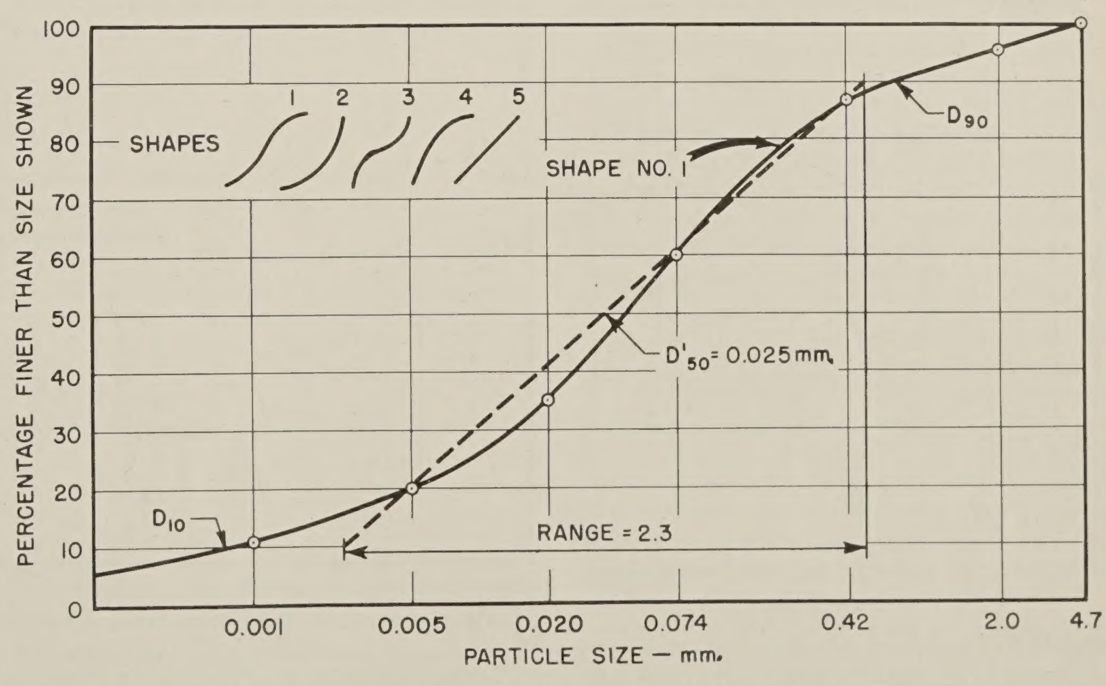

Figure 6.—Typical grain-size distribution curve showing shape, D'_{50}, and range.

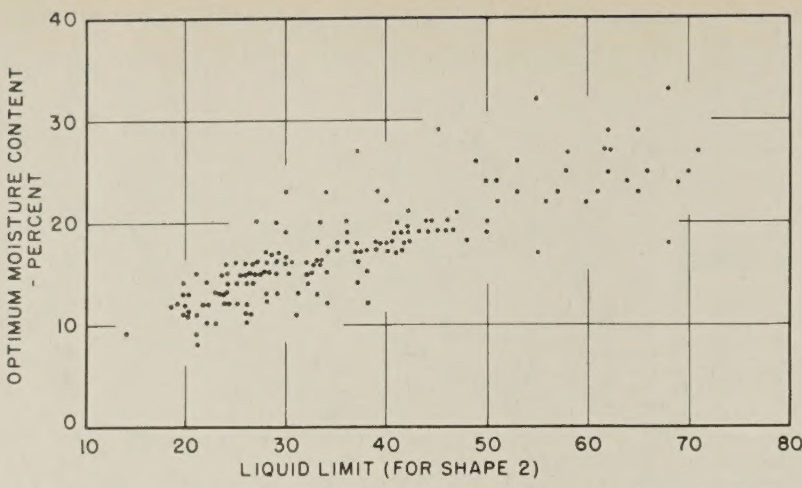

Figure 7.—A preliminary plot of optimum moisture content vs. liquid limit.

(No. 200), 0.020, 0.005, and 0.001. (3) The average particle size, D'_{50}, was obtained at the midpoint of a straight line (fig. 6) drawn to approximate the major portion of the grain-size distribution curve. The dashed line in figure 6 is an example of this straight line; it approximates the grain-size distribution curve from D_{10} to D_{90}. The percentage finer than the 0.001-mm. size was used in the first two measures mainly because it is the finest fraction normally measured in the Public Roads soils laboratory. The range of grain or particle sizes, designated herein as range, is

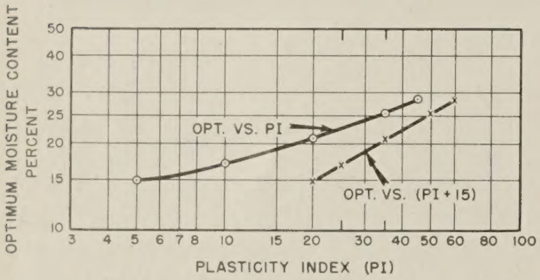

Figure 8.—Logarithmic plots of optimum moisture content vs. PI, and optimum vs. (PI+15), showing effect of adding a constant to the independent variable.

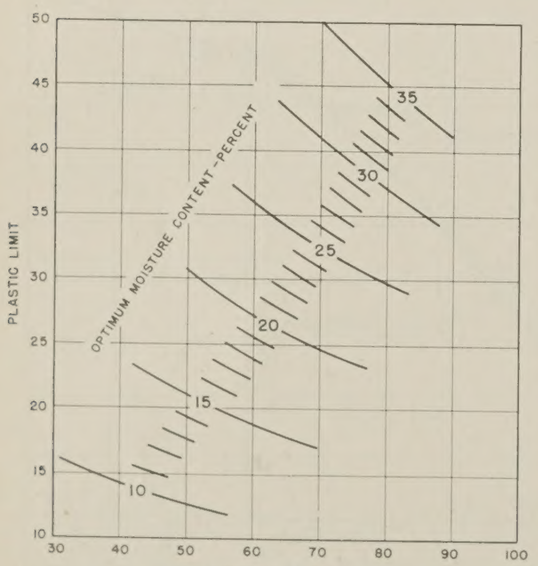

Figure 9.—Relation of optimum moisture content to plastic limit and fineness average, analysis No. 4.

defined as the number of log cycles traversed by the straight line approximating the D_{10} to the D_{90} portion of the curve. The type of grain-size distribution curve was designated shape. The five shapes considered are shown in figure 6.

Study of Simple Relationships

To examine the simple relationships of each dependent variable—optimum moisture content and maximum dry density—with each independent variable, plots were made on rectangular coordinates to arithmetic scale. Separate plots were made for each shape. An example, optimum moisture content versus liquid limit for shape 2, is given in figure 7. The results indicated good correlations of optimum moisture content with liquid limit and with plastic limit, and good correlations of maximum dry density with optimum moisture content and with plastic limit. A summary of the findings is given in table 3. Whenever a definite linear or curvilinear relation was developed on arithmetic scale, that line approximating the data was replotted to logarithmic scale. The type of relationship resulting, linear or curvilinear, also is shown in table 3. An examination of the plotted data for each shape indicated that separating the data on the basis of shape was of little or no value; hence, the data for all the samples were subsequently analyzed together regardless of shape.

Regression Analysis by Electronic Computer

The multiple linear regression analyses were performed on an IBM 650 electronic computer using a computer program (file number 6.0.001) supplied by the machine manufacturer. The multiple regression analysis is a method for obtaining a formula for estimating one variable by means of several other variables. The analysis provides the linear equation that best fits the data. The results of the multiple linear regression analyses are summarized in table 4. The variables and the standard error of estimate[5] are given for each analysis. The formulas that were developed are listed in table 5.

●In the first of five regression analyses, to determine equations for predicting optimum moisture content, a relationship was sought using all six independent variables. The analysis was made without adjustments for the curvilinear relationships of D'_{50} and FA (table 4). The resulting standard error of estimate was ±2.00 percent moisture.

●In analysis No. 2, logarithmic transformations were made of all variables in order tc make the linear program applicable to curvilinear relationships. The resulting standarc error of estimate indicated a slightly poorei correlation than that obtained in analysis No 1. It was found that the logarithmic trans formation, in addition to straightening ou certain curvilinear relationships, had causec some linear relationships to become curvilinear

[5] The standard error of estimate is a measurement of dev ation or degree of scatter of the points (test results) aroun the regression line. If the normal distribution of error hold: 67 percent of the test results will be within one standar error of the predicted result, and 95 percent will be withi 2 standard errors. The unit of measure is the same as th of the predicted variable.

Table 4.—Variables and resulting standard error of estimate for each regression analysi

Analysis No.	Dependent variable	Independent variables[1] O.M.C.	LL	PL	PI	D'_{50}	FA	0.001	Standard error of estimate	Remarks
1	O.M.C.	--------	X	X	X	X	X	X	[2] ±2.00	Data used directly.
2	do	--------	X	X	X	X	X	X	±2.05	Log transformation.[3]
3	do	--------	X	X	X	X	X	X	±1.98	Log transformation with adjustments.[4]
4	do	--------	----	X	----	----	X	------	±2.17	Do.
5	do	--------	----	X	----	----	X	------	±1.13	Do.[5]
6	M.D.D.	--------	X	X	----	----	X	------	±4.44	Data used directly.
7	do	X	X	X	----	----	X	------	±2.52	Log transformation with adjustments.[4]
8	do	--------	----	X	----	----	X	------	±4.32	Do.
9	do	--------	----	X	----	----	X	------	±2.98	Do.[5]

[1] Independent variables are simplified; see table 5 for exact form of each variable. Meanings of abbreviations are giv in table 3.

[2] The standard error of estimate by predicting with PL alone was ±2.45; the predicting equation is O.M.C.=0.811 PI 0.430.

[3] All variables were transformed to natural logarithms.

[4] Constants were added to some independent variables before logarithmic transformation to make all relationships lin with dependent variable.

[5] Analysis is for 40 samples from Loudon Co., Tenn.

Table 5.—Summary of predicting formulas from Public Roads Study No. 2

Analysis No.	Predicting formula
1	$\text{O.M.C.}=1.427\ \text{LL}-0.815\ \text{PL}-1.373\ \text{PI}-0.0007\ \text{D}'_{50}+0.062\ \text{FA}+0.035\ (0.001\ \text{fraction})-1.312$
2	$\text{Log O.M.C.}=0.158\log(\frac{\text{LL}}{10})+0.647\log \text{PL}+0.021\log \text{PI}+0.012\log(\frac{\text{D}'_{50}}{100})+0.354\log \text{FA}+0.248\log(0.001\ \text{fraction})-0.974$
3	$\text{Log O.M.C.}=1.029\log \text{LL}+0.045\log \text{PL}+0.224\log(\text{PI}+15)-0.033\log(\frac{\text{D}'_{50}}{100})+0.229\log(\text{FA}+100)+0.098\log(0.001\ \text{fraction}+40)-3.401$
4	$\text{Log O.M.C.}=0.784\log \text{PL}+1.378\log(\text{FA}+100)-6.586$
[1] 5	$\text{Log O.M.C.}=0.763\log \text{PL}+1.377\log(\text{FA}+100)-6.544$
6	$\text{M.D.D.}=147.525-0.020\ \text{LL}-1.195\ \text{PL}-0.198\ \text{FA}$
7	$\text{Log M.D.D.}=7.126-0.653\log(\text{OMC}+15)+0.059\log \text{LL}-0.120\log(\text{PL}+20)+0.014\log \text{FA}$
8	$\text{Log M.D.D.}=7.247-0.567\log(\text{PL}+20)-0.110\log \text{FA}$
[1] 9	$\text{Log M.D.D.}=7.105-0.518\log(\text{PL}+20)-0.113\log \text{FA}$

Legend:
O.M.C., Optimum moisture content.
M.D.D., Maximum dry density.
LL, Liquid limit.
PL, Plastic limit.
PI, Plasticity index.
D'_{50}, The average particle size on a straight line approximating the grain-size distribution curve.
FA, Fineness average.
0.001 fraction, Percentage of particles finer than 0.001 millimeter.
log, Natural logarithm.

[1] For 40 samples from Loudon Co., Tenn.

●Prior to analysis No. 3, the average linear and curvilinear relationships developed in arithmetic plots of the data were plotted to logarithmic scale to determine the effect of logarithmic transformation on all the variables. When the resulting plot was curved, constants were added to the independent variables to obtain straightline relationships. An example of this type of adjustment is shown in figure 8. The constants were determined by trial and error. The standard error of estimate of ±1.98 percent moisture indicated a slightly better relationship than found in the previous analysis.

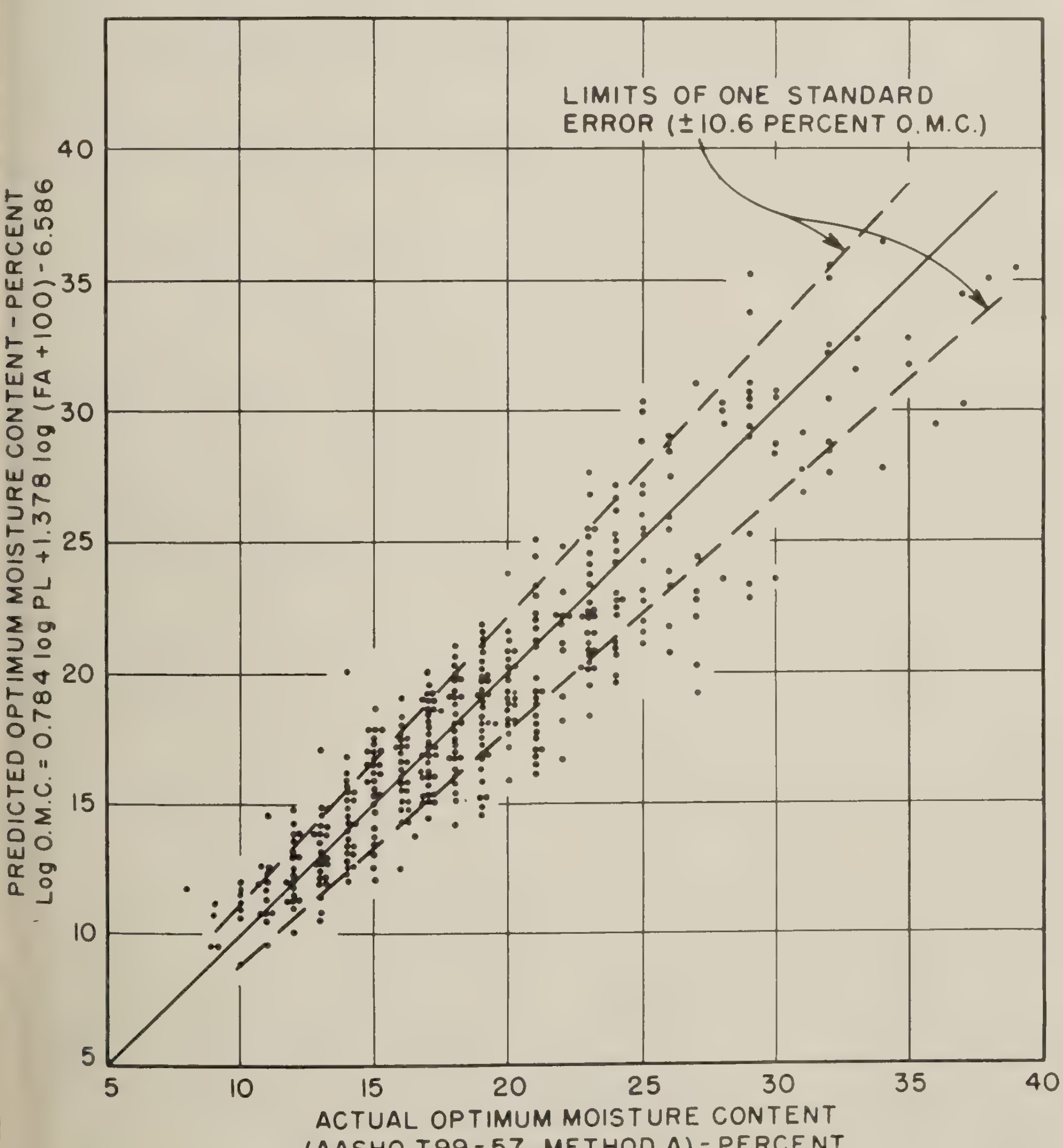

Figure 10.—Actual vs. predicted optimum moisture content from analysis No. 4.

●In analysis No. 4, optimum moisture content was compared with only two independent variables—plastic limit and fineness average. The number of variables was reduced to simplify the predicting equation. Plastic limit and fineness average were used because the partial correlation coefficients from previous analyses had indicated them to be the best two of the independent variables for predicting optimum moisture content. The reduction of independent variables, however, reduced the accuracy of the predicting formula; the standard error of estimate increased to ±2.17 percent moisture. The predicting formula developed in this analysis was:

$$\log \text{O.M.C.}=0.784\log \text{PL}+1.378\log(\text{FA}+100)-6.586 \qquad (5)$$

Where,

log = natural logarithm
O.M.C. = optimum moisture content, percent
PL = plastic limit
FA = fineness average.

Figure 9, developed from formula (5), shows optimum moisture contents for the range of plastic limits and fineness averages studied.

The standard errors of estimate shown in table 4 were based on the numerical differences between the measured and predicted optimum moisture contents and maximum dry densities. These standard errors of estimate may give the erroneous impression that the optimum moisture content or the maximum dry density can be predicted with the same accuracy through a wide range of values. A more realistic picture of the relationship between predicted optimum moisture content from formula (5) and the test optimum moisture content is shown in figure 10. The deviations, in percent moisture, increased as the actual optimum moisture content increased.

●Analysis No. 5 was made with data representing 40 samples from Loudon Co., Tenn. The test data from a single county were

Table 6.—Soil data used in comparison of methods for predicting optimum moisture content

Location sampled	Soil [1] series	Horizon sampled	Optimum moisture content	Maximum dry density	LL	PL	PI	Specific gravity	Shrinkage limit	Shrinkage ratio	Gradation finer than sizes shown in millimeters [2] 4.7	2.0	0.42	0.074	0.020	0.005	0.001	FA
			Percent	P.c.f.							Percent	Percent	Percent	Percent	Percent	Percent	Percent	
Ottawa, Ill.	(3)		14	119	28	15	13	2.72	13	1.95	100	98	92	78	62	41	21	65
Bayfield Co., Wis.	Gogebic	A_2	14	109	24	22	2	2.71	23	1.68	100	99	90	54	35	13	7	50
Jerome Co., Idaho	Portneuf	B_2	16	107	26	22	4	2.72	19	1.73	100	100	99	97	51	22	13	64
New Castle Co., Del.	Manor	A	17	109	35	26	9	2.71	23	1.61	100	89	77	50	42	30	16	51
DeSoto Co., Miss.	Grenada	C_2	18	106	41	22	19	2.70	19	1.73	100	100	109	100	67	30	23	70
Strafford Co., N.H.	Suffield	C_2	20	107	40	22	18	2.74	21	1.72	100	100	99	98	77	51	32	76
Maricopa Co., Ariz.	Mohave	B_2	20	109	43	20	23	2.66	11	1.96	100	99	91	73	50	40	28	64
Elbert Co., Ga.	Cecil	B_2	22	100	68	34	34	2.74	26	1.53	100	97	80	68	64	55	42	68
Madison Co., Iowa.	Winterset	B_2	25	94	70	30	40	2.73	9	2.02	100	100	99	98	80	52	42	79
Albemarle Co., Va.	Davidson	B_2	31	89	72	37	35	2.89	24	1.60	100	100	99	95	89	82	75	90

[1] Sampled and named by the Soil Conservation Service, U.S. Department of Agriculture.
[2] Gradation percentages are based on the fraction passing the No. 4 sieve, the same fraction used in the compaction test.
[3] AASHO Road Test embankment; test data are average values of a cooperative materials testing program reported by Shook and Fang (*12*).

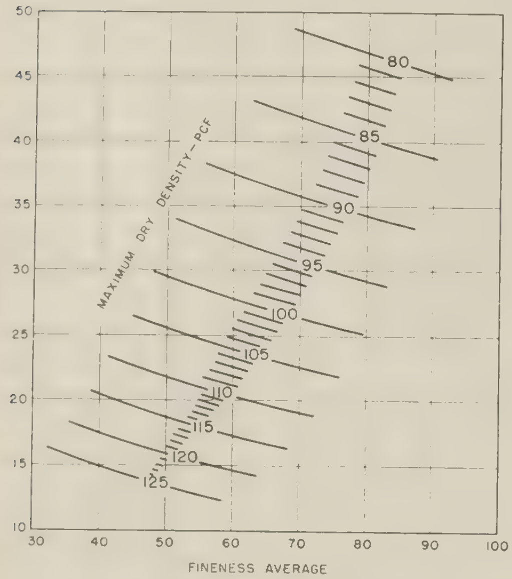

Figure 11.—Relation of maximum dry density to plastic limit and fineness average, analysis No. 8.

Table 7.—Comparison of methods for predicting optimum moisture content

Soil sample [1]	Optimum moisture content: Test [2]	Predicted by— PL and FA	PL and LL	Jumikis	Turnbull	Davidson and Gardiner	Rowan and Graham
	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
AASHO embankment	14	13	14	13	19	12	9
Gogebic	14	15	15	14	13	18	18
Portneuf	16	17	15	15	18	14	14
Manor	17	18	18	[3] 21	14	14	15
Grenada	18	18	18	22	20	21	16
Suffield	20	19	18	21	24	23	18
Mohave	20	16	18	23	19	14	7
Cecil	22	25	26	(4)	22	28	18
Winterset	25	25	24	(4)	28	18	6
Davidson	31	32	27	(4)	38	32	21
Sum of the deviations	0	13	16	(5)	28	37	63

[1] Description and basic data are given in table 6.
[2] Determined by AASHO Designation: T 99–57, Method A.
[3] Extrapolated.
[4] Beyond limits of predicting chart, figure 1.
[5] Insufficient data.

Table 8.—Comparison of methods for predicting maximum dry density

Soil sample [1]	Maximum dry density: Test [2]	Predicted by— PL and FA	PL and LL	Davidson and Gardiner	Rowan and Graham
		P.c.f.	P.c.f.	P.c.f.	P.c.f.
AASHO embankment	119	118	114	118	125
Gogebic	109	110	112	112	109
Portneuf	107	107	112	110	109
Manor	109	104	106	108	111
Grenada	106	106	106	99	105
Suffield	107	105	106	99	108
Mohave	109	110	106	111	125
Cecil	100	92	92	85	105
Winterset	94	94	95	98	126
Davidson	89	87	90	81	100
Sum of the deviations	0	20	30	52	76

[1] Description and basic data given in table 6.
[2] Determined by AASHO Designation: T 99–57, Method A.

selected to show how closely a regression formula would fit actual test results when the soils were from a relatively small area, where a restricted range of soil formation processes might exist. The actual optimum moisture contents ranged from 12 to 34 percent. The standard error of estimate was ±1.13 percent moisture.

- In regression analyses Nos. 6 through 9, the relationship of maximum dry density to several independent variables was investigated. In analysis No. 6, three independent variables that had appeared to be most closely related to maximum density during the examination of simple relationships were involved; these three were liquid limit, plastic limit, and fineness average. The data were used directly in the analysis without adjustment. The standard error of estimate was ±4.44 p.c.f.

- In analysis No. 7, maximum dry density was related to all of the independent variables in the sixth analysis and to optimum moisture content, which was known from the first Public Roads study to have a very good correlation with maximum dry density. All of the data were logarithmically transformed and adjusted where necessary. The standard error of estimate for this analysis was ±2.52 p.c.f.

- In analysis No. 8, maximum dry density was related to plastic limit and to fineness average, the number of variables being reduced to simplify the predicting equation. Although the number of variables was reduced from that used in analysis No. 6, the accuracy was slightly improved, probably because of the logarithmic transformations. The standar error of estimate was ±4.32 p.c.f. The pre dicting formula developed in analysis No. was:

$$\log \text{M.D.D.} = 7.247 - 0.567 \log (\text{PL}+20) - 0.110 \log \text{FA} \quad \text{....(6}$$

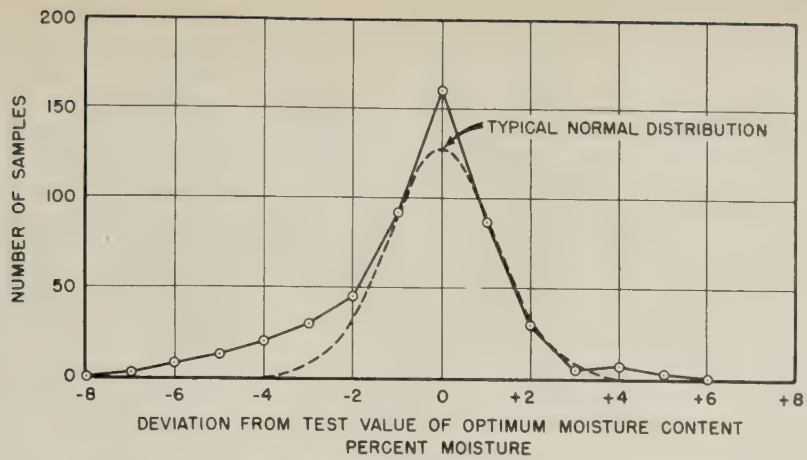

Figure 12.—Distribution of deviations of predicted optimum moisture contents, which was determined by the LL and PL method, from test results.

In figure 11, developed from this formula, maximum dry densities are shown for the range of plastic limits and fineness averages studied.

●Analysis No. 9 was performed in the same manner as No. 8 except that the data were limited to the 40 samples from Loudon Co., Tenn., for which the maximum dry densities ranged from 83 to 119 p.c.f. The standard error of estimate was ±2.98 p.c.f., which was smaller than that of analysis No. 8 and reflected the reduction in number of soil varieties.

Regression analyses summarized

To summarize the results of the regression analyses, the standard errors of estimate listed in table 4 showed that the two formulas developed for Loudon Co., Tenn., in analyses Nos. 5 and 9, fit the data much better than the corresponding formulas developed in analyses Nos. 1, 2, 3, 4, 6, and 8 for several States. Analysis 7 included the test optimum moisture content as an independent variable and, therefore, should not be considered with the other eight analyses. The standard errors of estimate of the Loudon County formulas were 1.1 percent moisture and 3.0 p.c.f. for optimum moisture content and maximum dry density, respectively. To show the relative magnitude of these standard errors of estimate, they may be compared to the averages of the test results examined. The standard error of 1.1 percent moisture was 5.3 percent $\left(\frac{1.1}{20.94}\times 100=5.3\right)$ of the average optimum moisture content (20.94); the standard error of 3.0 p.c.f. density was 2.9 percent $\left(\frac{3.0}{102.3}\times 100=2.9\right)$ of the average maximum dry density (102.30).

The standard errors of estimate of the formulas developed in analyses Nos. 1, 2, 3, 4, 6, and 8 for all the samples were approximately 2 percent moisture and 4.4 p.c.f. density. In terms of the average optimum moisture content and maximum dry density, 18.75 percent and 105.28 p.c.f. respectively, these standard errors of estimates were 10.7 and 4.2 percent. It is possible that combinations of the independent variables different from those used in this study could result in better correlations; only those combinations given in table 4 were analyzed. The selections of variables were based mainly on the data shown in table 3 and on the partial correlation coefficients developed in the analyses.

Comparison of Predicting Methods

Optimum moisture content

To test the formula from analysis No. 4 (see fig. 9) for predicting optimum moisture content from plastic limit and fineness average, data for 10 soils that varied considerably in characteristics and that were from widely

Table 9.—Comparison of actual [1] optimum moisture contents and maximum dry densities with those estimated by figures 9 and 11, respectively, based on plastic limit and fineness average for soils from Alaska and outside the United States

Soil		Optimum moisture content			Maximum dry density		
Location	Kind	Actual	Estimated	Deviation	Actual	Estimated	Deviation
		Percent	*Percent*	*Percent*	*P.c.f.*	*P.c.f.*	*P.c.f.*
Alajuela, Costa Rica	Lateritic	41	32	−9	77	87	+10
Las Lomas, Panama	do	29	32	+3	88	85	−3
Liberia	do	20	21	+1	107	96	−11
Do	do	12	12	0	126	120	−6
Do	do	15	14	−1	117	112	−5
Addis Ababa, Ethiopia	Black Cotton	39	38	−1	80	80	0
Kodiak, Alaska	Silty alluvium	20	21	+1	103	98	−5
Do	do	24	25	+1	95	93	−2
Do	do	33	30	−3	80	84	+4
Kenai-Kasilof, Alaska	do	10	9	−1	131	130	−1
Do	do	14	15	+1	118	114	−4
Do	do	13	12	−1	124	120	−4
Sum of the deviations				23			55

[1] Determined by AASHO Designation: T 99–57, Method A.

Table 10.—Simple correlation coefficients between pairs of variables used in analyses Nos. 1 and 3 for optimum moisture content

VARIABLES—ANALYSIS NO. 1

	O.M.C.	LL	PL	PI	D'_{50}	FA	0.001
O.M.C.	1.00						
LL	0.87	1.00					
PL	0.91	0.84	1.00				
PI	0.72	0.95	0.62	1.00			
D'_{50}	0.47	0.45	0.35	0.45	1.00		
FA	0.75	0.77	0.64	0.74	0.73	1.00	
0.001	0.76	0.90	0.66	0.91	0.50	0.84	1.00

VARIABLES—ANALYSIS NO. 3

	Log O.M.C.	Log LL	Log PL	Log (PI+15)	Log $\frac{D'_{50}}{100}$	Log (FA+100)	Log (0.001 fract.+40)
Log O.M.C.	1.00						
Log LL	0.89	1.00					
Log PL	0.90	0.83	1.00				
Log (PI+15)	0.74	0.94	0.59	1.00			
Log $\frac{D'_{50}}{100}$	0.76	0.82	0.63	0.81	1.00		
Log (FA+100)	0.77	0.77	0.63	0.74	0.92	1.00	
Log (0.001 fract.+40)	0.76	0.88	0.62	0.92	0.90	0.84	1.00

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separated geographical areas were selected from the files. These data are given in table 6. The predicted and test optimum moisture contents are shown in table 7, as are the optimum moisture contents predicted by the first Public Roads study (PL and LL) and by methods developed by Jumikis (*2*), Turnbull (*6*, *7*), Davidson and Gardiner (*5*), and Rowan and Graham (*3*). The results indicated that the PL and FA method is a slightly better predictor than the PL and LL method. The other four methods are at a disadvantage in this type of comparison because they were developed for a more limited range of soil types (origin). Some predictions by each method were quite accurate.

Maximum dry density

To test the formula from analysis No. 8 (see fig. 11) for predicting maximum dry density from plastic limit and fineness average, data for the 10 soils described in table 6 again were used. The actual and predicted maximum dry densities are shown in table 8. This summary also shows the densities predicted by use of the Public Roads method from the first study (PL and LL) and by methods proposed by Davidson and Gardiner (*5*) and by Rowan and Graham (*3*). The densities predicted on the basis of plastic limit and fineness average were generally closer to the actual maximum dry densities, although for a few soils one of the other methods provided a closer estimate.

Another test of the formulas from analyses 4 and 8 was made using data from Alaska and areas outside the continental United States. The comparisons of estimated and actual test optimum moisture contents and maximum dry densities are given in table 9. The soils used in this test were the same, except for the Hawaiian soils, as those used in the first Public Roads study to evaluate the PL and LL method (table 2). Estimates for the Hawaiian soils could not be made because the grain-size analyses were not available. The sums of the deviations shown in table 9 are 23 percent moisture and 55 p.c.f. density, and they represent about half of the corresponding sum of deviations resulting from the estimates made with the methods developed in the first study.

Supplemental Information

To determine whether the deviations of the predicted optimum moisture contents from the actual optimum moisture contents have a normal distribution, the deviations listed in table 1 were plotted in figure 12. The resultant curve closely approximates a standard normal distribution curve; this is evidence that the standard error of estimate is a reasonable measure of the accuracy of the predicting methods. Simple correlation coefficients between pairs of variables used in analyses Nos. 1, 3, 6, and 7 are given in tables 10 and 11; the larger the coefficient, the better the correlation, 1.00 being perfect.

Table 11.—Simple correlation coefficients between pairs of variables used in analyses Nos. 6 and 7 for maximum dry density

	VARIABLES—ANALYSIS NO. 6			
	M.D.D.	LL	PL	FA
M.D.D.	1.00	----------	----------	----------
LL	0.81	1.00	----------	----------
PL	0.89	0.84	1.00	----------
FA	0.70	0.77	0.64	1.00

	VARIABLES—ANALYSIS NO. 7				
	Log M.D.D.	Log (O.M.C.+15)	Log LL	Log (PL+20)	Log FA
Log M.D.D.	1.00	----------	----------	----------	----------
Log (O.M.C.+15)	0.97	1.00	----------	----------	----------
Log LL	0.81	0.89	1.00	----------	----------
Log (PL+20)	0.90	0.90	0.83	1.00	----------
Log FA	0.67	0.74	0.75	0.61	1.00

Tabulation Available

The test data used in the multiple linear regression analyses have been tabulated and are available to other researchers who would like to make additional studies of the data. This eight-page tabulation is available from the Chief, Physical Research Division, Bureau of Public Roads, U.S. Department of Commerce, Washington 25, D.C. In addition to the basic classification and compaction test data for 527 soils, the tabulation lists the location from which each soil was sampled, the soil series name, the soil type or textural classification of the "A" horizon, and the horizon actually sampled.

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APPENDIX I

Computer Curves for the PL and LL Analyses

Two additional analyses have been made to check the chart, figure 4, developed in the first Public Roads study. The analyses provided the following equations:

M.D.D. = 139.233 − 0.1344 LL − 1.185 PL
O.M.C. = 0.8151 + 0.1358 LL − 0.5290 PL

The standard errors of estimate were 4.69 p.c.f. for maximum dry density and 2.15 percent for optimum moisture content.

The charts developed from the two equations are given in figures 13 and 14. The curves were drawn within the range of the data examined. A comparison of these new curves with those given in figure 4 shows that the new curves generally have the same slope and spacing as the left-hand, straight-line portions of the old curves and that, for soils with maximum dry densities greater than 100 p.c.f., the curves almost coincide.

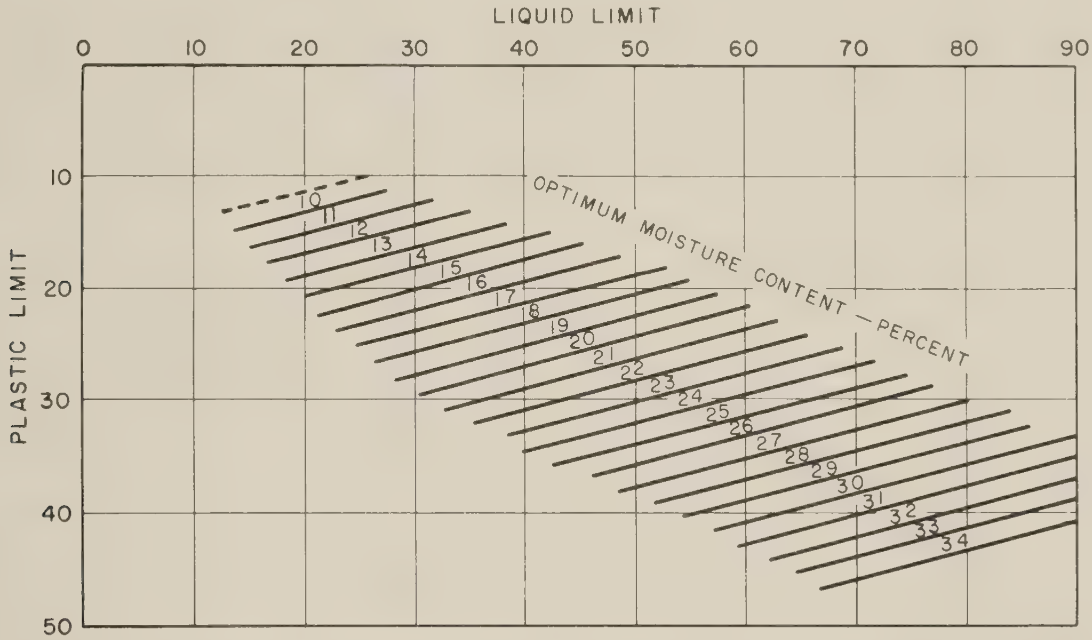

Figure 13.—Relation of optimum moisture content to plastic limit and liquid limit.

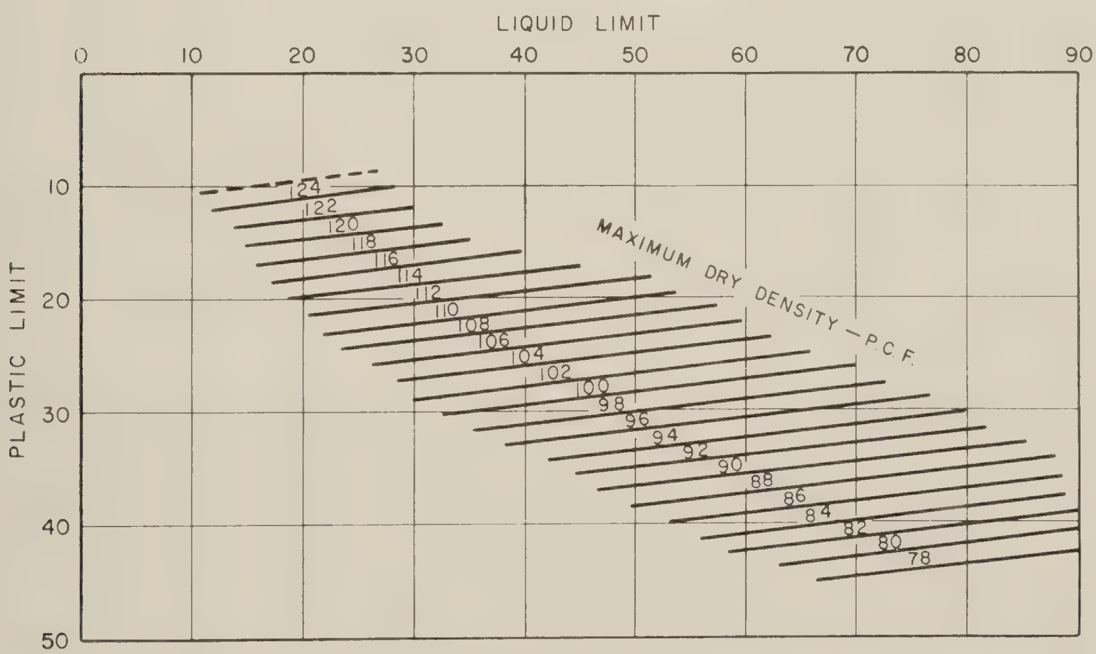

Figure 14.—Relation of maximum dry density to plastic limit and liquid limit.

Comparison of Properties of Coal-Modified Tar Binder, Tar, and Asphalt Cement

BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by WOODROW J. HALSTEAD, Supervisory Chemist, EDWARD R. OGLIO, Chemist, and ROBERT E. OLSEN, Highway Research Engineer

This article presents information on a study conducted to determine whether a newly developed product, coal-modified tar binder, could be used in the same manner as asphalt cement for a binder in pavement construction. Results of the laboratory study of the properties of this binder, a material produced by the high-temperature digestion of finely divided coal in tar and high boiling tar oils, are given. Also discussed is the comparison made of the coal-modified binder's properties and those of an ordinary tar pitch, RT-12 tar, and 85-100 penetration grade asphalt cement. Properties of laboratory mixtures containing each of the three binders also were compared.

This study indicated that coal-digestion increased the tar's resistance to hardening and apparently made it somewhat less brittle than an ordinary tar pitch. The improvements in these properties were not considered sufficient to warrant considering the coal-modified tar binder as a substitute for asphalt cement. Immersion-compression tests indicated that mixtures made with the coal-modified tar binder had greater resistance to deterioration from water than similar mixtures made with asphalt cement.

On the basis of this work, it is suggested that the coal-modified tar binder be considered as an improved tar and that it be used under the same conditions and for the same general purposes as unmodified tars.

Introduction

CONSIDERABLE interest was aroused when the Curtiss-Wright Corp. announced the development of a new coal-based road binder as the result of a research program that had been conducted in an effort to find new uses for coal and coal products. The great interest in this material by groups in several States, especially those seeking ways to use more coal in distressed, coal-producing areas, and the potential effect of such a binder on the national highway program prompted the Bureau of Public Roads to consider this development carefully. This article summarizes the results of the tests made and the findings of Public Roads concerning this material.

The basic principle used in the preparation of the new binder is the simultaneous digestion of powdered coal in coal tar and tar oils at a temperature of 500° to 600° F. It was claimed that, by adjusting the proportions of tar, tar oils, and coal, binders could be prepared covering the same penetration range as asphalt cements. It was the intent of the developer that the modified binders would be used in the same manner as asphalt cements in hot plant mixtures.

The digestion of powdered coal in tars and pitch has been used for a number of years in pipeline coatings and in pitches for steep, built-up roofs. No previous attempt has been made in this country to use this principle in the manufacture of a binder for pavements, but studies of the effects of powdered coal on the properties of road tar have been conducted recently in South Africa (*1*).[2] The details of the manufacture of the coal-modified tar binder and background information concerning its development are discussed in a report issued Dec. 30, 1960, by the Curtiss-Wright Corp., Research Division (*2*), and the description of the construction and performance of the first experimental pavements built with the new binder is given in two reports by the Kentucky Department of Highways (*3, 4*).

Through the courtesy of the Curtiss-Wright Corp. and the Kentucky Department of Highways, samples of the coal-modified tar binder and various materials were obtained for this study. Laboratory studies were conducted to determine the physical properties of the new binder and to compare these properties with the properties of ordinary tar and of asphalt cement. Two series of tests were made: In the first, a sample of the coal-modified tar binder obtained directly from the Curtiss-Wright Corp. was compared with a waterproofing tar pitch. In the second, tests were made to compare the coal-modified tar binder, the base tar, RT-12, and the asphalt cement used in one of the experimental pavement sections constructed in Kentucky.

Summary and Conclusions

These comparative tests showed that th coal digestion and the addition of high boilin tar oils, employed in the manufacture of th coal-modified tar binder, reduced the viscosity temperature susceptibility of the tar and mac it somewhat more resistant to hardening a high temperatures. However, comparison of this binder with asphalt of a similar softenir point (85-100 penetration grade) showed th the coal-modified tar binder retained chara teristics more nearly equal to those of the t than those of the asphalt. In particular, tl volatile loss and hardening in heat tests of tl two tar products were about the same, b these heat tests results were significantly d ferent from the results of similar tests asphalts. At all test temperatures, stabiliti as indicated by compressive strengths, f mixtures made with the coal-modified t

[1] Presented at the 41st annual meeting, Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 95.

binder were higher than stabilities of similar mixtures made with asphalt. However, the stabilities of the coal-modified tar binder obtained at low temperatures were such that a lack of flexibility and possible brittleness might be suspected. The subsequent behavior of the materials in the Kentucky experiments verified this brittleness and lack of flexibility.

The results of this study, in general, showed that coal-modified tar binders, such as the Curtiss-Wright material, should be considered as an improved tar. It would be expected to perform better than unmodified tars in a number of applications. However, it also was indicated that the precautions normally employed when using tars should be employed when using this material and that an attempt should not be made to substitute coal-modified tar binder for penetration grade asphalts.

Comparison of a Modified Tar Binder with a Tar Pitch

In the first series of tests, the properties of the coal-modified tar binder were compared to the properties of a tar pitch meeting an AASHO specification, *Standard Specification for Coal-Tar Pitch for Roofing, Dampproofing, and Waterproofing*, AASHO Designation: M-118, Type B. Table 1 gives the results of these tests. The tar pitch used in these comparisons was selected so as to have the same softening point as the modified binder. The penetration at 77° F. also was about the same for both materials. However, the absolute viscosity results, determined at different temperatures by the Koppers vacuum-operated capillary tube viscometer (*5*) showed the coal-modified tar binder to have a somewhat lower viscosity-temperature susceptibility. The viscosity-temperature coefficient, as indicated by the slope of the line obtained when log log absolute viscosity in centipoises was plotted against log absolute temperature in degrees Rankine (° F. plus 459.7), was −4.71 for the coal-modified tar binder and −4.99 for the tar pitch. Penetration tests made at different temperatures also indicated lower susceptibility for the coal-modified tar binder, the slope of the log penetration versus temperature curves being 0.0258 and 0.0427 for the coal-modified tar binder and the tar pitch, respectively.

In the thin-film oven test (⅛-in., five hours) run at the standard temperature of 325° F. (*Standard Method for Thin-Film Oven Test*, AASHO Designation: T 179–60), both materials had very high losses and exhibited extreme hardening. As this temperature is higher than one that likely would be used for materials of this nature, tests were repeated at a temperature of 250° F. At this lower temperature, the coal-modified tar binder showed a greater resistance to hardening than did the tar pitch. Impact resistance, as measured by the height at which a one pound steel ball produced fracture of a rigidly supported half-inch cube of material at the indicated test temperature, was greater for the modified binder than for the tar pitch. Except for ductility, the results of tests

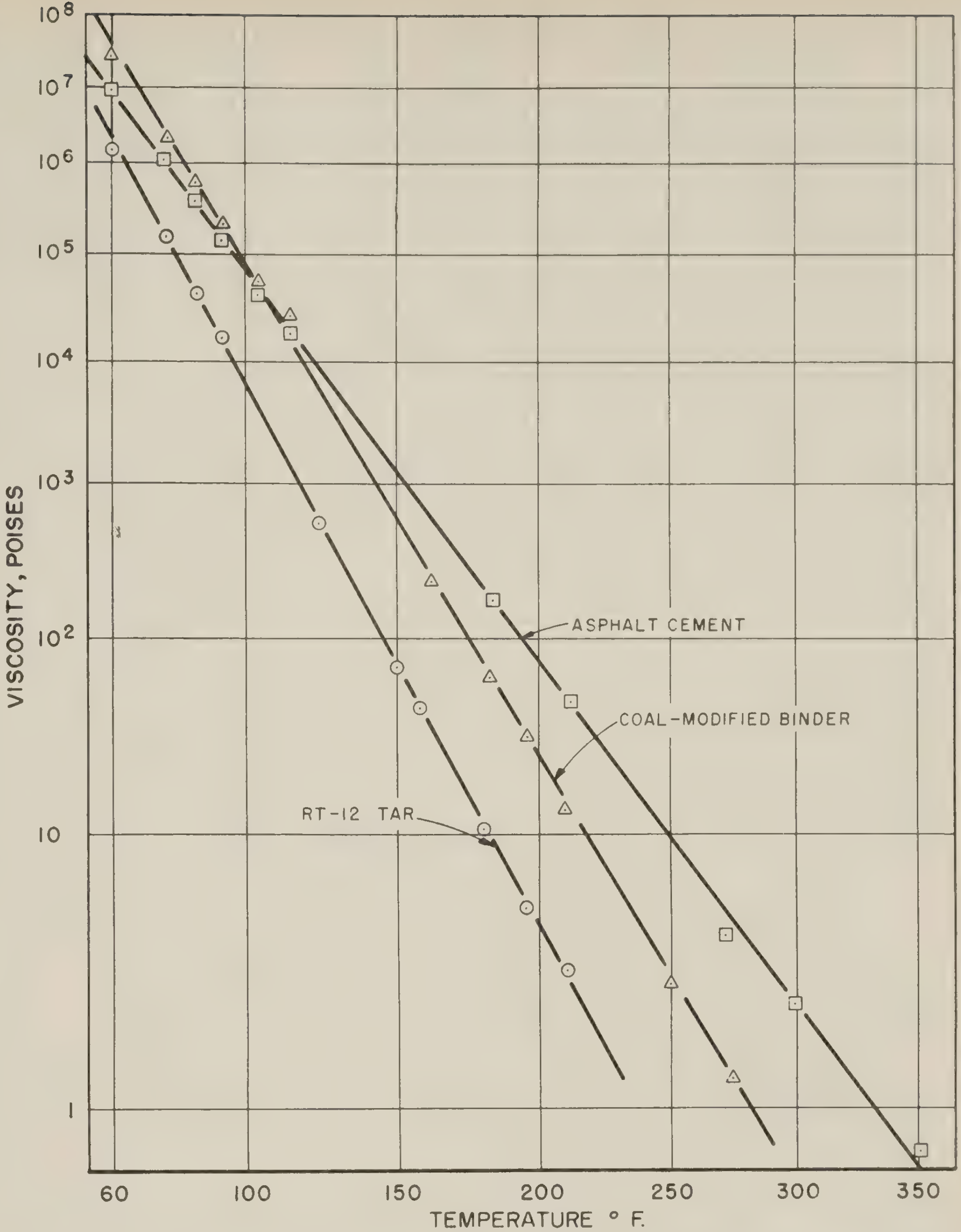

Figure 1.—Viscosity-temperature relationships for binders used in Kentucky experimental project.

made on the residues from the thin-film oven tests at 250° F. showed the same general trends as the results of tests on the original materials. No specific conclusions regarding comparable ductility can be drawn. The original ductility of the modified binder was higher at 77° F. than that of the tar pitch, but at 60° F. the reverse was true. At 77° F., the 250° F. residue of the tar pitch was more ductile than the coal-modified tar binder even though it was harder, as judged by the penetration test. At 60° F., the tar pitch had no ductility compared to a ductility of 8 centimeters for the coal-modified tar binder.

This series of tests confirmed the claim that the coal-modified tar binder has properties that are an improvement over those for tar pitch; they also indicated that further research was warranted to compare the properties of the new binder with asphalt cement and to determine the relative behavior of these two materials when used in paving mixtures.

Comparison of Properties of Binders

The second and more extensive series of tests was made on samples of materials used in the Kentucky experimental paving project 11 (urban) located on U.S. 25 in London, Ky. The materials included the coal-modified tar binder, the RT-12 tar used as the base for the binder, the 85–100 penetration grade asphalt cement used in the control sections, the powdered coal, and the aggregate used in the mixtures. The properties of the three

Table 1.—Comparison of properties of coal-modified binder with tar pitch

	Coal-modified tar binder	Waterproofing pitch
Softening point (R&B) ° F.	117	117
Specific gravity at 77°/77° F.	1.248	1.269
Solubility in CS_2 percent	74.6	81.1
Distillate to 572° F. (AASHO T 52) do	2.4	6.7
Absolute viscosity, poises, at:		
140° F.	1335	747
149° F.	594	310
158° F.		160
167° F.	130	80.1
173.5° F.	92.7	
176° F.		44.0
Viscosity-temperature susceptibility [1]	−4.71	−4.99
Penetration at:		
77° F., 100 g., 5 sec	63	62
53.4° F., 100 g., 5 sec	15	6
39.2° F., 200 g., 60 sec	23	11
Penetration-temperature susceptibility [2]	0.0258	0.0427
Ductility, 5 cm./min., cm., at:		
77° F.	51	41
60° F.	30	59
Impact resistance,[3] inches to fracture, at:		
77° F.	9+	5
60° F.	5	2
53.5° F.	5	2
Thin-film tests at 325° F. (⅛-in. film, 5 hrs.):		
Loss percent	11.4	9.4
Softening point of residue ° F.	170	162
Penetration of residue, 100 g., 5 sec. at 77° F.	0	0
Thin-film tests at 250° F. (⅛-in. film, 5 hrs.):		
Loss percent	1.4	2.9
Penetration of residue, at:		
77° F., 100 g., 5 sec	27	15
53.4° F., 100 g., 5 sec	5	1
39.2° F., 200 g., 60 sec	8	1
Retained penetration at 77° F. percent	43	24
Penetration-temperature susceptibility [2]	0.0310	0.0498
Ductility, 5 cm./min., cm., at:		
77° F.	38	61
60° F.	8	0
Impact resistance,[3] inches to fracture, at:		
60° F.	2	¾
53.4° F.	1¼	1

[1] Viscosity-temperature susceptibility $= \dfrac{\log \log v_2 - \log \log v_1}{\log T_2 - \log T_1}$

v=viscosity in centipoises.
T=Temperature in degrees Rankine (° F.+459.7).

[2] Penetration-temperature susceptibility $= \dfrac{\log P_2 - \log P_1}{t_2 - t_1}$

P_1=Penetration at temperature t_1.
P_2=Penetration at temperature t_2.
t=temperature, degrees F.

[3] One pound steel ball on ½-inch cube.

binders were determined to show their relative viscosity-temperature susceptibilities and their degree of hardening when heated. Comparisons were made of the properties of laboratory mixtures prepared with each binder and the aggregate used in the Kentucky experiment. Comparisons also were made for mixtures of each of the three binders and other selected aggregates.

Table 2 gives the results of the tests made on the binders. These include normally determined physical characteristics and absolute viscosities, in poises, at various temperatures ranging from about 60° F. to 350° F. To determine viscosities over this wide range of temperatures, three instruments were used: the Shell sliding plate microviscometer (*6*) at the lower temperature range of 60° F. to 115° F.; the Koppers vacuum-capillary viscometer in the intermediate temperature range of 115° F. to 200° F.; and the Saybolt furol viscometer, with conversions being made to absolute viscosity in the higher temperature range of 200° F. to 350° F.

Figure 1 shows the viscosity-temperature relations of the three binders used in the Kentucky project. Although viscosity in poises and temperature in degrees Fahrenheit are indicated in the figure, these curves were obtained by plotting log log viscosity in centipoises against log of absolute temperature in degrees Rankine. A good, straightline relation is indicated by the data points for all the binders even though the viscosities were obtained by the different instruments. The difference in slope of the curves for the asphalt and the coal-modified tar binder is significant. The RT–12 tar had substantially lower viscosities for the same temperatures, but it was only slightly more susceptible to temperature than the coal-modified material. The calculated slopes of these lines are: −4.78, −4.38, and −3.51 for the tar, coal-modified tar binder, and the asphalt cement, respectively.

Figure 2 shows the furol viscosities at various temperatures for the three binders. These curves were used to determine the various mixing temperatures for the studies of laboratory mixtures to be discussed later. It should be noted that furol viscosity here is directly related to kinematic viscosity in stokes rather than to absolute viscosity in poises. The data points shown in these curves are the basis for the absolute viscosities, in poises, given in table 2 for the temperatures indicated.

Resistance to hardening

One of the claims of the developers, that coal-modified tar binder could be used in the same manner as asphalt cement in bituminous concrete construction, was of much interest because it is well known that ordinary tars are more volatile than asphalts and should not be subjected to the same high temperatures as asphalt during the production of hot paving mixtures. Because of this claim, the relative resistance of these two binders to hardening when subjected to heat was of considerable interest. To compare this heat resistance, the thin-film oven test (⅛-in., 5 hours), which is commonly used as a specification test to evaluate the hardening characteristics of asphalt cements, was employed. Tests were made at temperatures ranging from 210° F. to 325° F. on the three binder materials. In addition, the oven-loss test (*Standard Method of Test for Loss on Heating of Oil and Asphaltic Compounds*, AASHO Designation: T 47–42) was made on the coal-modified tar binder and the asphalt cement. The results of these tests are given in table 3.

Figure 3 shows the comparison of the weight loss during the thin-film tests of the asphalt cement, the coal-modified tar binder, and the RT–12 tar at the various test temperatures. Although this particular asphalt cemen showed small gains in weight, most asphal cements of this grade have small weight losse in this test. In the study of asphalt cement recently conducted by Public Roads (*7*), th highest loss shown by any of the 85–100 pene tration grade asphalts was 2.18 percent Eighty-seven percent of the 85–100 material in that investigation had losses of 0.5 percen or less. The coal-modified tar binder wa different from asphalts with respect to vola tility, as evidenced by its loss of 10 percent a 325° F. The loss for the RT–12 tar was 11. percent at 325° F.; thus, the results for th two tar materials generally were comparable.

Figure 4 illustrates the relative hardenin characteristics of the three binders as measure by the percent of original penetration retaine after the thin-film test at various test tem peratures. The wide differences between th tar products and the asphalt again wa illustrated. Different asphalts, of cours show different percentages of retained penetra tion, but the percentage never would be a low as that shown by the tar products. Fo example, the asphalt showing the highest los encountered with 85–100 penetration grad materials, which was described in the pre ceding paragraph, had a retained penetratio of 33 percent. This is considerably highe than the 1 and 3 percent shown by the tw tar materials under the same conditions.

Note that, although the hardening asphalt in the thin-film oven test has bee related to the hardening in an asphalt durin mixing in a hot plant, no information available concerning the relation of th hardening of tar materials in such tests to th actual hardening occurring in the mixing an laying operations. Because of the relativel arge amount of volatile matter in tar products, ; is possible that the long time of oven xposure, compared to the time the mixture is t high temperature prior to compaction on he road, would produce considerably greater ardening in these materials than actually ould be encountered in construction operaions. Nevertheless, it is believed that the elationships shown are important: They mphasize the fact that the coal-modified tar inder has properties different from those of sphalt and, therefore, it is not realistic to onsider this material as an alternate to sphalt in all respects.

Studies of Laboratory Mixtures

To further compare the behavior of the oal-modified tar binder, RT–12 tar, and sphalt cement, the characteristics of laboraory prepared mixtures made with each binder vere compared under several conditions. The hanges in stability after various periods of ven aging and after various periods of mmersion in water were determined. Another series of stability tests, at different emperatures, was made to evaluate the effect of the previously observed differences in the viscosity-temperature relations of the binders.

Because of the relatively wide differences in he specific gravities of the three binders, all of the laboratory mixtures prepared for the various phases of this part of the study were designed using equal volumes of binder per mit weight of aggregate (5.85 milliliters of inder per 100 grams of aggregate). This was considered to provide a better basis for comparison than test mixtures prepared with percentages of binder on an equal weight basis. Because of the differences in temperature susceptibility, the mixing and molding temperatures were controlled so that the furol viscosities of the binders were approximately the same for each individual phase of the study.

The aggregates for all of the mixtures were heated overnight at the predetermined mixing temperatures. The binders were heated to this mixing temperature just prior to the mixing with the preheated aggregates in a modified Hobart mechanical mixer. The mixing period was 2 minutes. Marshall specimens were prepared in a mechanical compactor with 50 blows of the hammer applied to each face of the specimen. The specimens for the unconfined compression test were molded by the double-plunger method with a 3,000 p.s.i. load held for 2 minutes.

Marshall Stability

To determine the effect of laboratory aging tests on the strength of mixtures as measured by Marshall stability, specimens were prepared with the three binders, as received. In addition, specimens were prepared with the RT–12 tar in which 10 percent of the tar was replaced by powdered coal, in order to obtain some measure of the filler effect of the powdered coal. As stated previously, all of the mixtures were proportioned with the

Table 2.—Test properties of Kentucky binders

	RT–12 tar	Coal-modified tar binder	Asphalt cement
Specific gravity at 77°/77° F.	1.265	1.267	1.019
Penetration at 77° F., 100 g., 5 sec.	235	71	92
Softening point (R&B) ° F.	96	115	116
Ductility 77° F., 5 cm./min. cm.	------	54	182
Solubility in CS_2 percent	79.5	71.3	[1] 99.9
Organic insoluble do	20.5	28.0	0.1
Distillation test (AASHO T 52) to:			
455° F. percent	0.0	0.0	------
518° F. do	0.3	0.3	------
572° F. do	3.8	2.7	------
Residue do	95.7	97.2	------
Softening point of residue ° F.	113	128	------
Absolute viscosity, poises, at:			
61.5° F.	1,500,000	24,500,000	8,800,000
77° F.	160,000	2,300,000	1,200,000
86° F.	42,000	660,000	418,000
95° F.	17,500	200,000	169,000
105° F.	------	56,000	44,000
115° F.	------	28,000	20,000
123.6° F.	492	------	------
149.4° F.	65.7	------	------
157.9° F.	40.9	------	------
160.3° F.	------	217	------
180° F.	10.3	[2] 60.1	[2] 169
195° F.	4.92	[3] 29.0	[4] 43.2
210° F.	2.77	12.8	[4] 43.2
250° F.	------	2.52	------
275° F.	------	1.26	4.35
300° F.	------	------	2.18
350° F.	------	------	.747
Viscosity-temperature susceptibility [5]	−4.78	−4.38	−3.51

[1] Solubility in carbon tetrachloride.
[2] Actual temperature, 180.8° F.
[3] Actual temperature, 195.9° F.
[4] Actual temperature, 211.7° F.
[5] $\frac{\log \log v_2 - \log \log v_1}{\log T_2 - \log T_1}$

v = viscosity in centipoises.
T = Absolute temperature in degrees Rankine (° F. + 459.7).

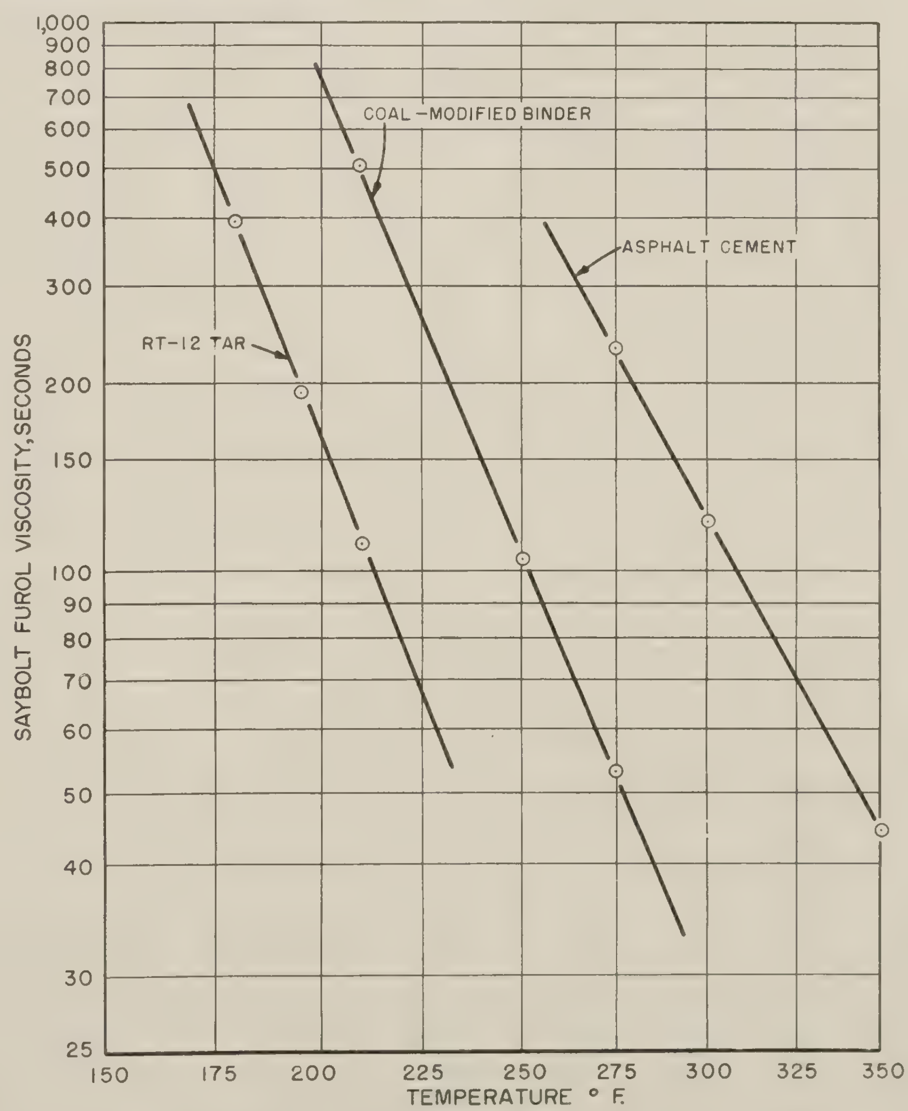

Figure 2.—Furol viscosity of binders at various temperatures.

Table 3.—Effect of oven exposure at various temperatures

	RT-12 tar	Coal-modified tar binder	Asphalt cement
Oven-loss test (AASHO T-47):			
Loss ... percent	------------	2.36	0.02
Penetration of residue at 77° F., 100 g., 5 sec	------------	40	80
Retained penetration ... percent	------------	56	65
Thin-film tests (⅛ in. film, 5 hrs.):			
At 210° F.:			
Loss ... [1] percent	1.46	1.05	+0.08
Penetration of residue, at 77° F., 100 g., 5 sec	128	50	79
Retained penetration ... percent	54	70	86
Ductility at 77° F., 5 cm. per min ... cm	73	46	228
Softening point ... degrees F	103	121	119
At 250° F.:			
Loss ... [1] percent	3.60	2.68	+0.23
Penetration of residue at 77° F., 100 g., 5 sec	71	31	75
Retained penetration ... percent	30	44	82
Ductility at 77° F., 5 cm. per min ... cm	151	42	239
Softening point ... degrees F	114	129	121
At 275° F.:			
Loss ... [1] percent	5.67	4.47	+0.25
Penetration of residue at 77° F., 100 g., 5 sec	41	18	68
Retained penetration ... percent	17	25	74
Ductility at 77° F., 5 cm. per min ... cm	75	27	230
Softening point ... degrees F	122	137	122
At 325° F.:			
Loss ... [1] percent	11.35	9.98	+0.11
Penetration of residue at 77° F., 100 g., 5 sec	3	2	60
Retained penetration ... percent	1	3	65
Ductility at 77° F., 5 cm. per min	1	(2)	247
Softening point ... degrees F	150	164	126

[1] The + sign indicates gain in weight.
[2] Not run, specimen could not be prepared properly.

binder on an equal volume basis and were mixed and molded at an approximately equal furol viscosity. After molding, one group of specimens was tested immediately and the balance was aged in an oven at a temperature of 140° F. for periods ranging from 1 to 30 days. After each aging period, Marshall stability was determined at a temperature of 140° F. The stabilities obtained, along with pertinent information on proportioning and mixing, are given in table 4. The effect of aging, illustrated in figure 5, shows that the stability for all materials increased with aging in the oven, and that the increase in stability for the coal-modified tar binder was significantly greater than that obtained with either the asphalt or the RT–12 tar.

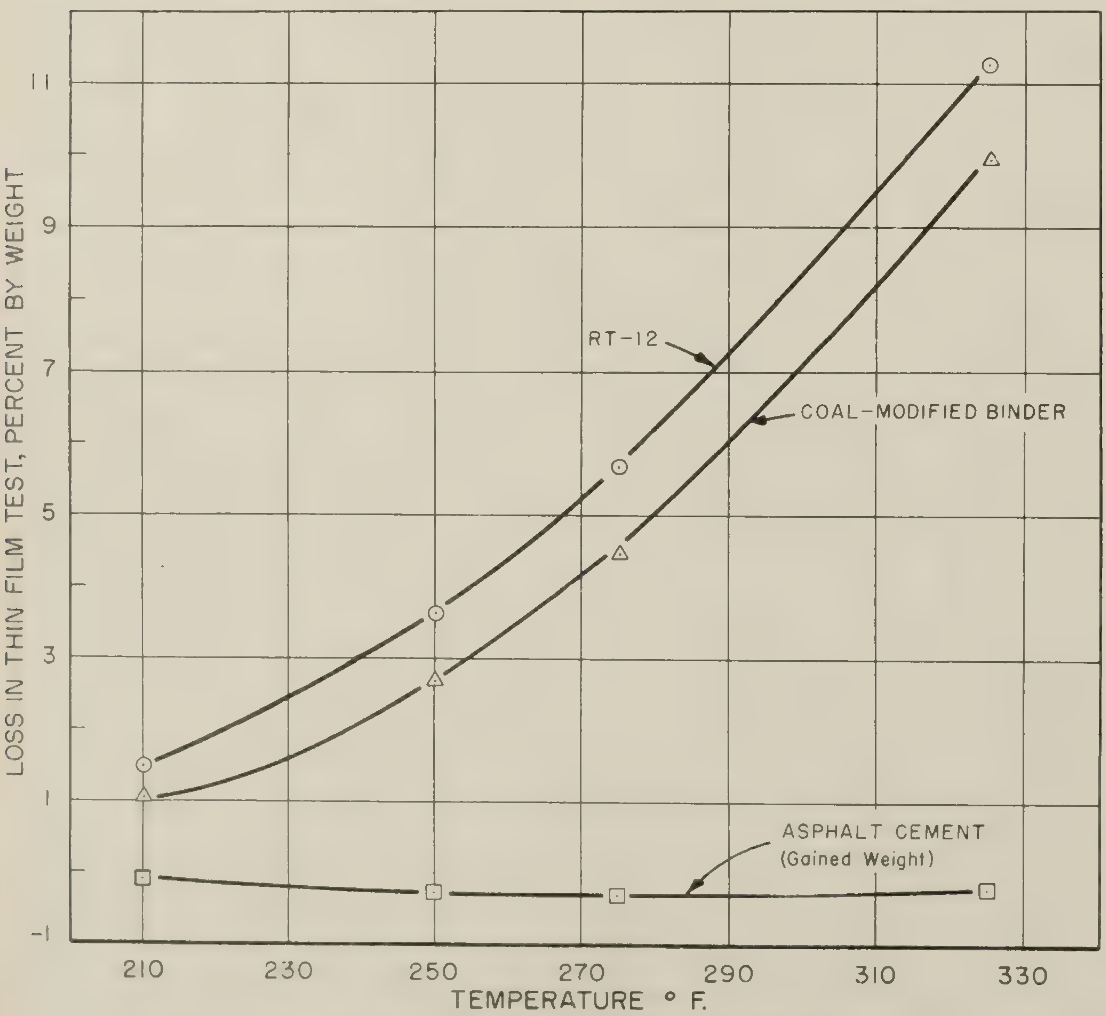

Figure 3.—Weight loss in thin-film test at various temperatures.

The previous oven-loss tests had indicated that the RT–12 tar had slightly greater volatility than the coal-modified tar binder and, therefore, a somewhat greater increase in stability would be expected for the RT–12 mixture if such an increase were attributed to volatility alone. Also, if the effect of the powered coal were primarily a mechanical effect of increased fine material, the stabilities of the RT-12 tar mixtures to which the powdered coal had beeen added would be expected to more nearly equal those obtained with the mixtures of coal-modified tar binder than the stabilities of the RT–12 mixtures. However, as can be seen, the data did not support such suppositions. Although the mixtures containing the powdered coal did have higher stabilities for the same conditions, this increase in stability accounted for only a very small proportion of the total increase in stability shown by the coal-modified tar binder mixtures. These results indicated that some factor, such as the development of internal structure, contributes to a considerably greater increase in the stability of the coal-modified tar binder than can be accounted for by the increase in viscosity caused by loss of volatile matter or by the effect of coal as a filler.

Effect of aging

To further explore the relative tendency of the three binders to exhibit a difference in characteristics upon aging, a special series of viscosity tests was conducted, as follows.

Sliding plate viscosity specimens were prepared in the usual manner for viscosity determination with the microviscometer, and the initial viscosity was determined at 77° F within 15 minutes after preparation of the specimens had been completed. The sides of the glass plates then were taped with a nonpermeable plastic tape to minimize volatile losses and the specimens were placed in an oven at a temperature of 110° F. Absolute viscosity determinations at 77° F. were made on the same specimen after periods of 1, 5, 8, and 13 days. Immediately after the viscosity determination on the 13th day, the specimens were heated above their softening points to a temperature of 150° F. and the viscosity at 77 F. was redetermined. This procedure was used to obtain a measure of the nonpermanent hardening that had occurred. Results of these tests are given in table 5 and are shown graphically in figure 6. The vertical components of the lines in the figures at the 13-day period indicate the change in viscosity that occurred after the specimens were heated.

Although there are some inconsistencies in the data, viscosity of all three binders increased during oven aging. For the RT-12 tar, the viscosity increased when the specimen was heated above its softening point after 1 days of aging. Thus any possible development of structure or reversible hardening was masked by a permanent hardening, which probably was caused by an unavoidable loss of volatile matter. However, for both the asphalt cement and the coal-modified tar binder, viscosity at 77° F. decreased significantly after heating of the specimens to a temperature above the softening point and recooling. This decrease amounted to approximately 64 percent of the total increase for the coal-modified tar binder and to approximately 22 percent of the increase for the asphalt cement.

The tendency for bituminous materials to exhibit such hardening through the development of internal structure with time is well-recognized. Lee and Dickinson (*8*) attributed this phenomenon in tars to the possible crystallization of constituents or to a change in degree of the dispersion of colloidal constituents. Brown, Sparks, and Smith (*9*) in discussing this phenomenon in asphalts attributed it to ". . . internal physical reorientation and reorganization at the atomic, molecular, and micelle levels, of the components of the asphalt." Although this study was not conclusive, the results of oven-aging tests on both the mixtures and binders indicate that the coal-modified tar binder may exhibit the tendency to develop structural hardening to a much greater extent than either an ordinary tar or an asphalt. The effect of this phenomenon on the behavior of the materials in pavements should be considered in any further research on such materials.

Effect of Water on Compressive Strength

An important property of a bituminous mix is its ability to resist loss of strength in the presence of water. To show this characteristic, immersion-compression tests were made using two types of aggregate, a quartzite and a granite, both of which were relatively sensitive to water, as had been shown by previous Public Roads studies. Each of the aggregates was crushed and then recombined to the same grading as that of the aggregate obtained from the experimental project in London, Ky. Only the coal-modified tar and asphalt binders were used in this phase of the study.

Table 6 gives the results of compressive strength tests on the specimens before immersion and after immersion in water at 120° F. for periods up to 18 days. The retained strengths and percentages of swell after each period also are given along with information concerning the mixture composition and mixing temperatures.

The results of the compressive strength test at 77° F. and the percent of retained strength after various periods of immersion are plotted in figure 7. These results show that the modified tar binder had higher initial strengths than the asphalt cement with each aggregate. The percentages of retained strengths were also higher for specimens of each of the mixtures containing the coal-modified tar binder than for the comparable specimens containing

Table 4.—Effect of oven aging at 140° F. on Marshall stability

	RT-12 tar	RT-12 + coal [1]	Coal modified tar binder	Asphalt cement
Specific gravity of binder at 77°/77° F.	1.265		1.267	1.019
Weight of binder per 100 g. of aggregate ... g.	7.40		7.41	5.96
Volume of binder per 100 g. of aggregate ... ml.	5.85		5.85	5.85
Mixing temperature [2] ... ° F.	210	210	243	300
Marshall stability at 140° F. after aging in oven at 140° F. for: [3]				
0 days ... pounds	132	139	321	238
1 days ... do.	180	210	532	292
3 days ... do.	243	304	817	336
10 days ... do.	311	456	1,334	410
30 days ... do.	571	856	2,569	574

[1] Ten percent by weight of the tar was replaced by powdered coal used in manufacturing coal-modified binder.
[2] Mixing temperature was adjusted to give approximate equal viscosity for all binders (120 Saybolt furol seconds).
[3] Average of 4 specimens. Aggregate was same as that used in Kentucky experiment and was graded, as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200
Percent passing:	100	98	72	47	34	17	3	1.4

Table 5.—Increase in viscosity with time of oven aging at 110° F.

Aging period [1]	Viscosity at 77° F.		
	RT-12 tar	Coal-modified tar binder	Asphalt cement
Days	*Poises*	*Poises*	*Poises*
0	163×10^3	1.890×10^3	1.230×10^3
1	380	2,690	1,520
5	392	2,830	1,870
8	609	4,220	1,840
13	921	6,770	2,480
[2] 13-X	1,050	3,660	2,210

[1] Sealed specimens aged in oven at 110° F.
[2] Specimens heated to 150° F. after 13-day determination and viscosity determination repeated.

Table 6.—Effect of immersion in water at 120° F. on compressive strength

	Quartzite [1]		Granite [1]	
	Coal-modified tar binder	Asphalt cement	Coal-modified tar binder	Asphalt cement
Weight of binder per 100 g. aggregate ... g.	7.41	5.96	7.41	5.96
Volume of binder per 100 g. aggregate ... ml.	5.85	5.85	5.85	5.85
Mixing temperature [2] ... ° F.	240	300	240	300
Compressive strength before immersion [3] ... p.s.i.	258	151	363	257
Results [3] after 4 days of immersion:				
Compressive strength ... p.s.i.	223	100	237	163
Retained strength ... percent	86	66	65	63
Swell ... do.	0.1	0.5	1.0	1.0
Results [3] after 11 days of immersion:				
Compressive strength ... p.s.i.	239	93	212	131
Retained strength ... percent	93	62	58	51
Swell ... do.	0.2	0.8	1.3	1.5
Results [3] after 18 days of immersion:				
Compressive strength ... p.s.i.	237	84	209	129
Retained strength ... percent	92	56	58	50
Swell ... do.	0.2	1.4	1.4	1.5

[1] Gradation of both mixtures was as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 40	No. 80	No. 200
Percent passing:	100	98	72	47	17	3	1.4

[2] Mixing temperature was adjusted to give approximate equal binder viscosity (120-150 Saybolt furol seconds).
[3] All results based on average for three test cylinders, 4 inches in diameter and 4 inches high, tested at 77° F.

Table 7.—Variation of compressive strength with test temperature

	RT-12 tar	Coal-modified tar binder	Asphalt cement
Specific gravity of binder at 77°/77° F.	1.265	1.267	1.019
Weight of binder per 100 g. aggregate ... g.	7.40	7.41	5.96
Volume of binder per 100 g. aggregate ... ml.	5.85	5.85	5.85
Mixing temperature [1] ... ° F.	220	265	320
Compressive strength,[2] p.s.i., at:			
0° F.	5,197	5,446	2,568
36° F.	1,503	2,358	912
75° F.	220	544	291
100° F.	86	236	143

[1] Mixing temperature was adjusted to give approximately equal viscosities for all binders (70-100 sec.).
[2] Each result represents the average for three tests on cylinders, 3 inches in diameter and 3 inches high. Aggregate was composed of 58 percent crushed granite (½ in. to No. 10); 37 percent river sand (passing No. 10); and 5 percent limestone dust. Gradation was, as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200
Percent passing:	100	90	57	42	23	20	7	5.2

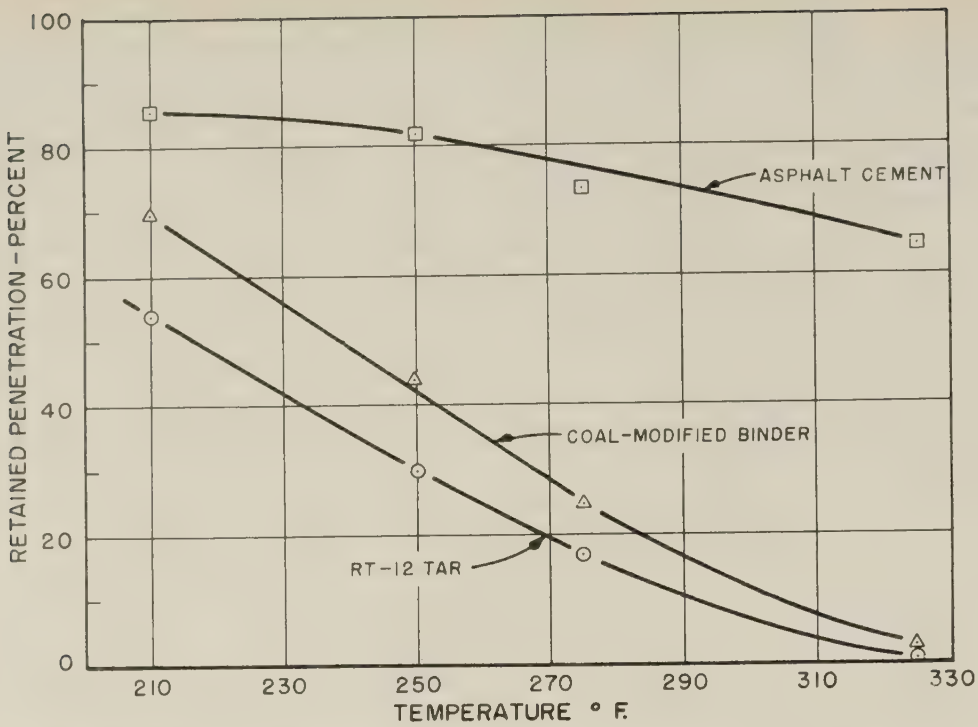

Figure 4.—Resistance to hardening at various test temperatures (1/8-in. film, 5 hours).

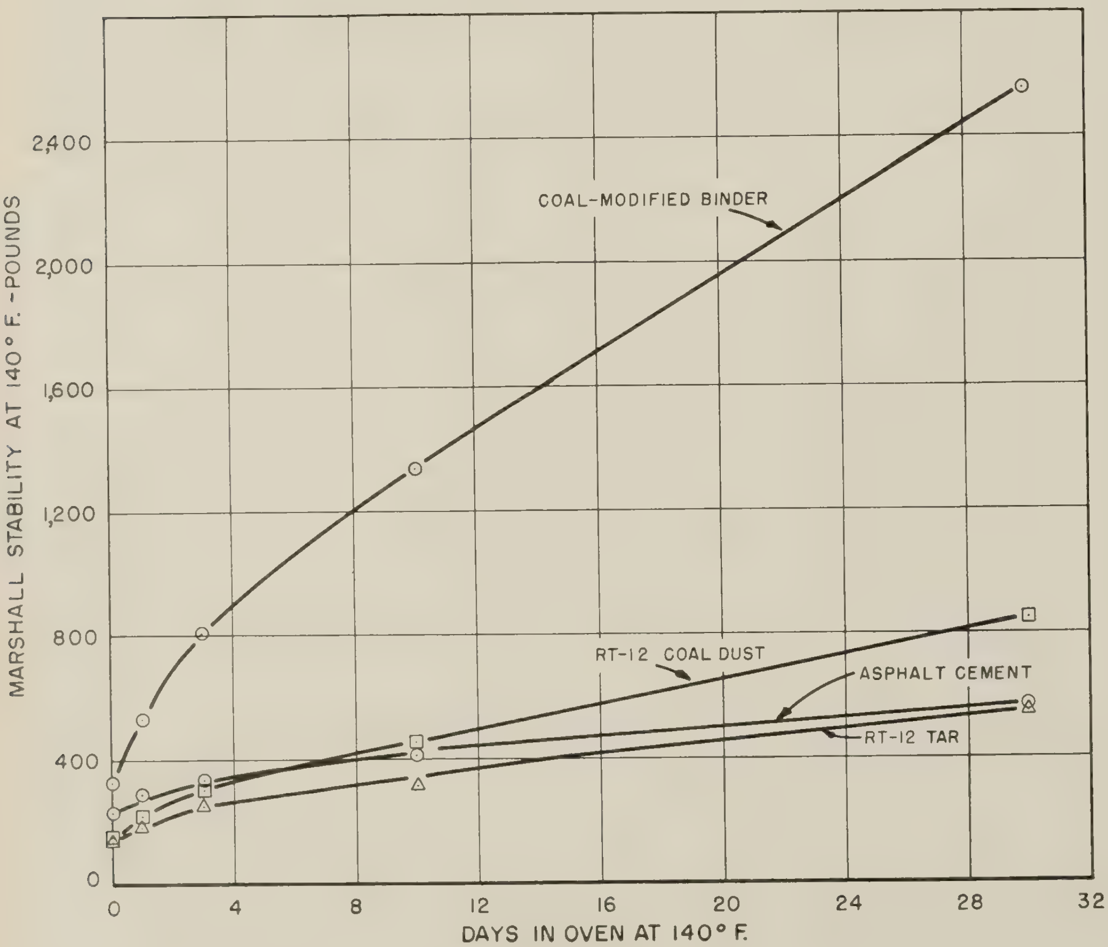

Figure 5.—Increase in Marshall stability during time of aging in oven.

asphalt and the same aggregate. Although the results for the mixtures made with quartzite aggregate and coal-modified tar binder were somewhat erratic, the strengths at 4 days being less than those at 12 and 18 days, these mixtures appeared to have been affected very little by water.

Effect of Temperature on Unconfined Compressive Strength

Because of the differences in viscosity-temperature susceptibility of the coal-modified tar binder, RT–12 tar, and asphalt binders, the relative change in stability with temperature was of interest. For this series of tests, compressive strength cylinders, 3 inches in diameter and 3 inches high, were used. As with the other test series, the binders were added on an equal volume basis and the mixtures were prepared at equal furol viscosities. The aggregate for this series of tests consisted of crushed granite as the coarse aggregate, river sand as the fine aggregate, and limestone dust as the mineral filler. The specimens were tested for stability by unconfined compressive strength tests over a range of temperatures from 0° F. to 100° F. The results of these tests, mixture composition, and mixing conditions are given in table 7.

The relationships of strength to temperature for the three binders are shown in figure 8. These results followed the generally expected pattern; they were in the same order as viscosity-temperature susceptibility of the binders themselves. Compressive strength decreased as the temperature increased, with the rate of decrease being greatest for the RT–12 tar and least for the asphalt cement. It is of interest to note that, when the curves for the asphalt and coal-modified tar binders were extended, the coal-modified tar binder showed higher strength at 140° F., the maximum temperature usually found in pavements. Thus it was indicated that the coal-modified tar binder of the consistency used in these tests would provide higher stabilities than the 85–100 penetration asphalts at any temperature that would be encountered in service. This was not true for the RT–12 tar; the curve intersects the asphalt curve at a temperature of 67° F., thus indicating that this material would have significantly lower stability at 140° F. It is of further interest to note the extremely high compressive strengths shown for the coal-modified tar and the RT–12 tar at 0° F. These strengths are approximately double those of the asphalt and are in the range of the strength of portland cement concrete.

The relation of the data obtained in these laboratory tests to stability of the pavement after construction is not known. However, the conditions of laboratory mixing, compaction, and testing were such that the hardening occurring would most likely have been less than actually occurs in construction. Also, the tar materials would be expected to exhibit greater differences between laboratory and field specimens than asphalts. Thus, it can be surmised that actual differences in the stabilities of pavements made with the different materials would be greater than those indicated by these tests.

As no evaluation of the brittleness or resistance to abrasion of the various mixtures was made, compressive strengths of the magnitude indicated by the tar materials at low temperatures should not necessarily be construed as being advantageous. It is likely that pavements containing such mixtures would be subject to abrasion losses and cracking at low temperatures, and that such distress would be accelerated by any hardening of the binders in service. The performance of these materials in the Kentucky experiments seemed to indicate such deficiencies.

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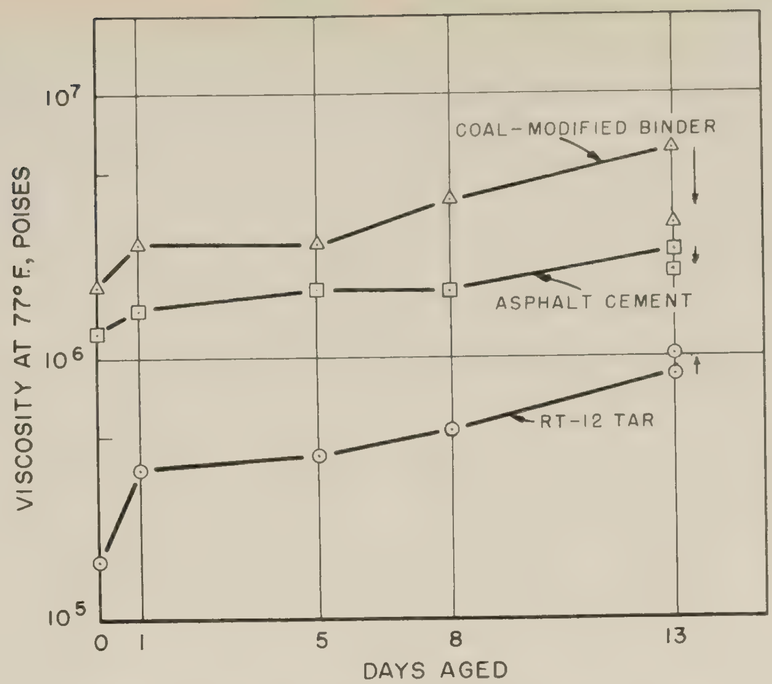

Figure 6.—Effect of aging at 110° F. on binder viscosity.

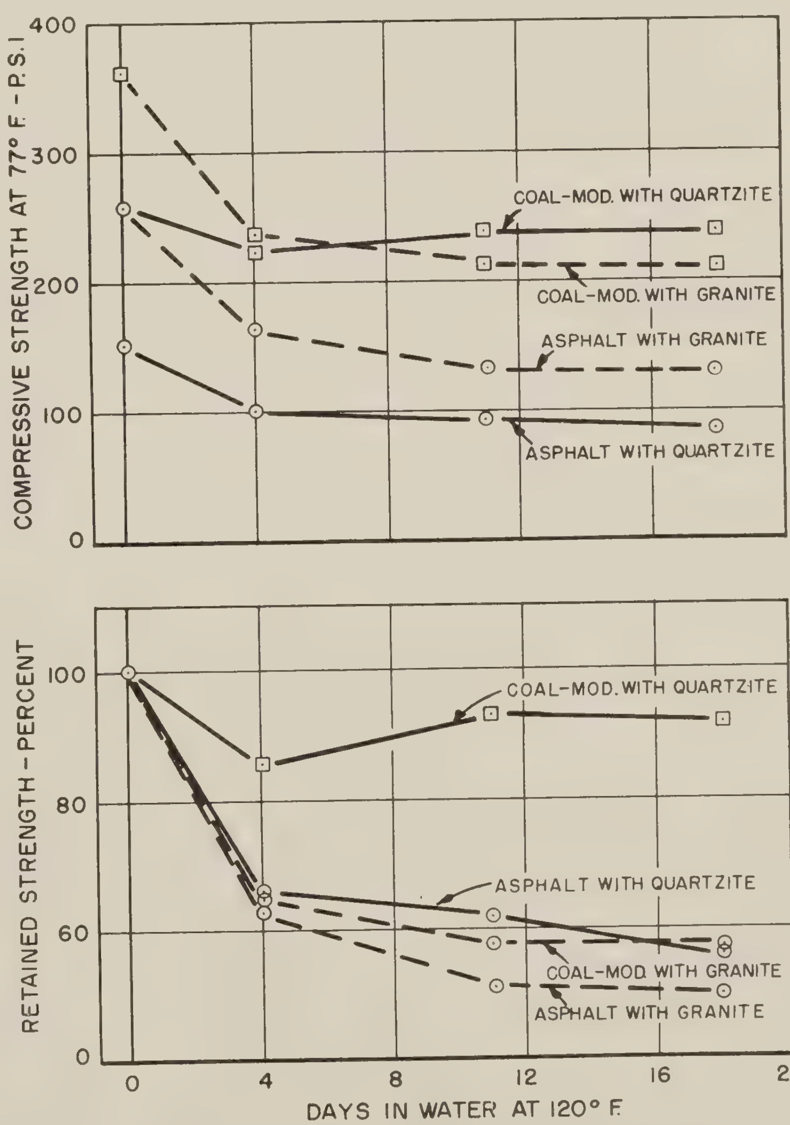

Figure 7.—Effect of water immersion on stability.

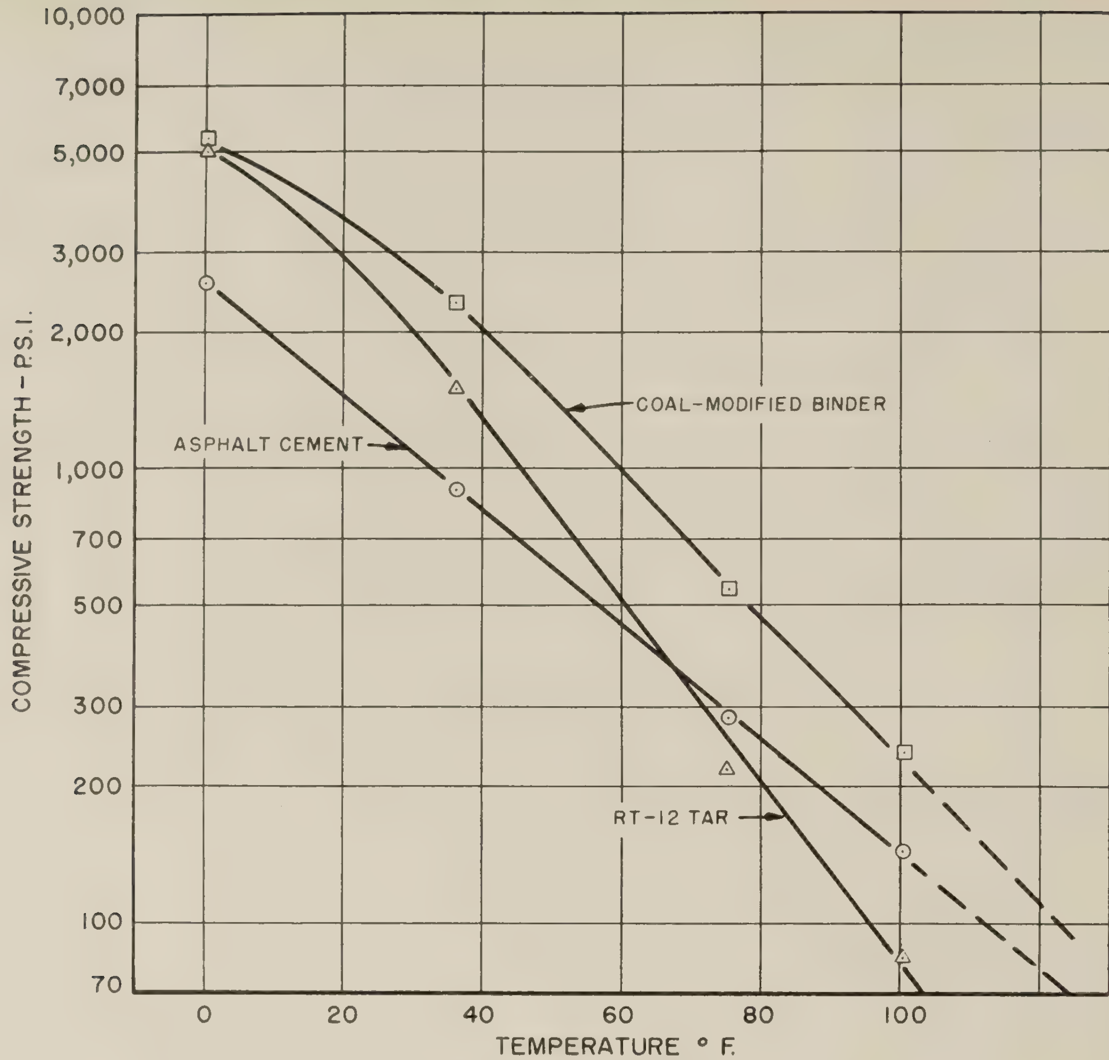

Figure 8.—Variation of compressive strength with temperature.

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