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Bulletin No. 20

Pavement Performance

1949

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PAVEMENT PERFORMANCE

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1948

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Washington 25, D. C. September 1949

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ANALYSIS OF SPRING BREAK-UP DATA IN VIRGINIA

T. E. SHELBURNE and A. W. MANER
Virginia Department of Highways

SYNOPSIS

The paper presents an analysis of road performance data collected at the time of the spring break-up on more than 1000 miles of primary and nearly 5000 miles of secondary roads in the Culpeper District. The purpose of the survey was to secure detailed information on road performance and to determine the extent and, insofar as possible, the causes of the major break-up. Results of such a survey are useful in conjunction with the planning of maintenance and construction programs

A uniform system of rating the numerous road sections was devised. Five ratings, depending upon the degree of distress, were employed. The survey was started about the middle of February, 1948 and all field work was completed near the first of April. Thus, all ratings were obtained at a time when subgrade support was at a minimum.

The performance ratings both for the primary and for the secondary roads were summarized by counties. The secondary roads were further divided into hard and non-hard surface types. A map of each county was prepared showing the ratings for each road section.

Despite the fact that primary highways carry approximately 80 percent of the traffic, their performance was considerably better than that of secondary roads. For example, about 43 percent of the primary roads were giving good performance (ratings 1 and 2) as compared to only 20 percent of the secondary roads. Likewise, only 29 percent of the primary highways were rated as poor (ratings 4 and 5), while 47 percent of the secondary roads were in this category.

For the purpose of analysis the Culpeper District was divided into five general soil areas according to parent materials. It was found that the roads in the Coastal Plain Sediments soil area were giving the best performance, while those in the Triassic "Red Beds" soil area were rated the poorest in the District.

The studies revealed that in all five soil areas macadam bases performed much better than selected soil, gravel, stone or stabilized bases. The order of ratings for bituminous surfaces from the best to poorest performance was as follows: plant mix, heavy bituminous mixes and light bituminous mixes.

An analysis of the 30-year weather records (1917-1947) brought to light certain pertinent facts. It was found that pavement break-ups are most severe for those years with low temperatures preceded by subgrade and base saturation (high precipitation). The past winter was second only to the 1935-36 one as regards to climatic conditions favorable to a severe break-up.

In order to design, construct and maintain better roads, highway engineers are making use of research to evaluate the various factors responsible for the behavior of pavements. Road condition surveys, if made at a time when differences in performance are most apparent, can be useful for this purpose. A comprehensive and thorough study at the time of the spring break-up will not only serve the purpose of locating the areas where distress is most prevalent, but also may serve to indicate the best solution to some of the problems.

Good pavement performance is dependent upon a number of factors. Often these factors are so interrelated that it becomes almost a hopeless task to isolate and evaluate each factor individually for some particular section of road. A study of a large number of pavements may bring to light certain facts not otherwise obtainable. Among the important variables are climate, traffic, design and the soil area in which the pavement is located. All highway engineers have observed that the spring break-up is much more severe some years than others. What are the climatic factors which contribute to a major spring break-up? What pavement designs are giving "year-around" good performance for the traffic conditions to which they are subjected? In what areas are the roads distressed the most at the time of the "break-up"?

To answer these questions the Department collected detailed information on weather data, and road performance this past spring to determine insofar as possible the extent and causes of the major break-ups. The survey was extensive since it included the 38,000 mi. in the secondary system and the 9,000 mi. of primary highways. It was started about the middle of February and all field work was completed near the first of April. The survey consisted of an inspection, logging and description of failure types (surface and base) and a performance rating of each individual road section. Pictures were taken to illustrate both poor and good performance and to record actual conditions.

In addition to the field survey, an analysis was made of the 30 yr. weather data throughout the State. For this analysis the reports from 16 US Weather Bureau stations (two per district) were used.

It may be pertinent at this time to note that the field parties were alerted to pay particular attention to road damage being caused by excessive loads. During the time of the break-up load limit restrictions were placed on many highways. Most interstate routes retained their designated 40,000-lb. gross load limit; however, many other primary roads were restricted to 24,000-lb. gross load and practically all secondary routes were posted for 16,000 lb.

Results of the state-wide road condition survey have been summarized by counties and districts; however, because of their bulk they are not included in this report. More detailed analysis of the data was desired to study the factors affecting pavement performance. Thus, the paper presents an analysis of the survey results for the Culpeper District - one of the eight in the State.

DESCRIPTION OF THE CULPEPER DISTRICT

The Culpeper District with an area of 5021 sq. mi. is located in the north central part of the State. It is bounded by Maryland on the north, by the crest of the Blue Ridge Mountains on the west, by the James River and Nelson County on the south, by the Potomac River and Stafford, Spotsylvania, Hanover and Goochland Counties on the east. It includes the following 13 counties: Albemarle, Arlington, Culpeper, Fairfax, Fauquier, Fluvanna, Greene, Loudoun, Louisa, Madison, Orange, Prince William and Rappahannock.

The population in 1930 was 243,000, had increased to 304,500 in 1940 and is now estimated at over 400,000. Two independent cities, Alexandria and Charlottesville, have an estimated population of 58,000 and 25,000 respectively. Several of the rural counties have had a tendency to lose some of their inhabitants while others have gained. Especially noticeable for gains are Arlington-which jumped from 57,000 in 1940 to an estimated present population of 120,000 - and Fairfax which increased from 41,000 in 1940 to 80,000 (estimated) in 1948.

The District is divided into six residencies. The offices of the Resident Engineers are located as follows: Fluvanna and Louisa Counties at Louisa Court House, Albemarle and Greene Counties at

Charlottesville; Culpeper, Orange, and Madison Counties at Culpeper; Fauquier and Rappahannock Counties at Warrenton, Fairfax, Arlington, and Prince William Counties at Fairfax; and Loudoun County at Leesburg. The District office is located at Culpeper.

The primary highway system includes 1175 mi. and the secondary road network has a mileage of 4958. In addition, the County of Arlington maintains its own highway system. Approximately 60 mi. of primary roads have been built to four-lane width, some of them being divided. This last type of mileage will be increased upon completion of the Henry Shirley Memorial Highway.

The principal primary highways in the district are: US Routes 1, 15, 29, 33, 50, 211 and 250. A traffic flow map as prepared by the Division of Traffic and Planning is presented in Figure 1. It will be noted that traffic volume is greatest on the above listed routes and is concentrated particularly in the Washington area. Additional data from the Traffic and Planning Division reveal that while only 13 percent of all roads in the state system are in the Culpeper District, about 17 percent of all traffic on the system is found in this area.

The district can be divided into two major physiographic provinces: the Piedmont Plateau and the Coastal Plain. The latter is extremely limited in extent and stretches along a narrow north-south band of land in Arlington, Fairfax and Prince William Counties bordering the Potomac River. A large part of the Piedmont Plateau in the Culpeper District is occupied by a series of Triassic Basins.

From a geological point of view the formations encountered vary widely. The oldest one is apparently the Catoctin greenstone, an extrusive rock of pre-Cambrian age (most likely Algonkian). It is a basic lava which lies parallel to the eastern flank of the Blue Ridge. Of similar age are the widespread Wissahickon schist (a chlorite-muscovite schist) and several granitic formations such as: the Marshall, Hypersthene, Lovingston and Columbia granites; hornblende gabbro and quartz monzonite are two other igneous pre-Cambrian formations.

In the lower Cambrian are the Loudoun slate and quartzite. Sedimentary rocks are to be found in the Culpeper District. Of Ordivician age are small out-crops of Arvonis and Quantico slate, and Everona limestone. Finally in the Triassic basin are three main groups of sedimentary rocks, namely, conglomerates, sandstones and shales. The conglomerates are usually classified according to component pebbles (limestone, quartz, schist, trap, arkose). The basins as well as the remaining of the plateau are cut by a number of diabase dykes, sills and stocks. In the Coastal Plain are to be found sediments, mostly arkosic, ranging from Cretaceous to Pleistocene in age. At places, along the Potomac, they are wholly unconsolidated and the most recent sediments are sand or peat and muck.

In the northern part of the district the drainage is toward the Potomac River. The central part is drained by the Rappahannock and Rapidan on the one hand and the Anna Rivers on the other. Runoff in the southern part is toward the James River and its tributaries. The drainage pattern is obviously a reflection of the topography. In the Culpeper District the high land lies to the west with a series of foothills stretching about as far as the Triassic basins. In those basins the relief is markedly flatter. The remaining of the district is gently rolling.

Weather conditions in the district, particularly those affecting the spring break-up, may best be illustrated by Figure 2. Data secured from the US Weather Bureau Station at Charlottesville for the months of November, December, and January are presented graphically for all years since 1917. Average mean temperature and total precipitation are plotted and compared with the 35-yr. normal. It will be observed for this 30-yr. period that the winter of 1917-1918 was the coldest with an average mean temperature of 33 F. Next come the winters of 1935-36 and 1939-40 each with a mean temperature of 37 F. The past winter was fourth with a mean temperature of 37.2 F.

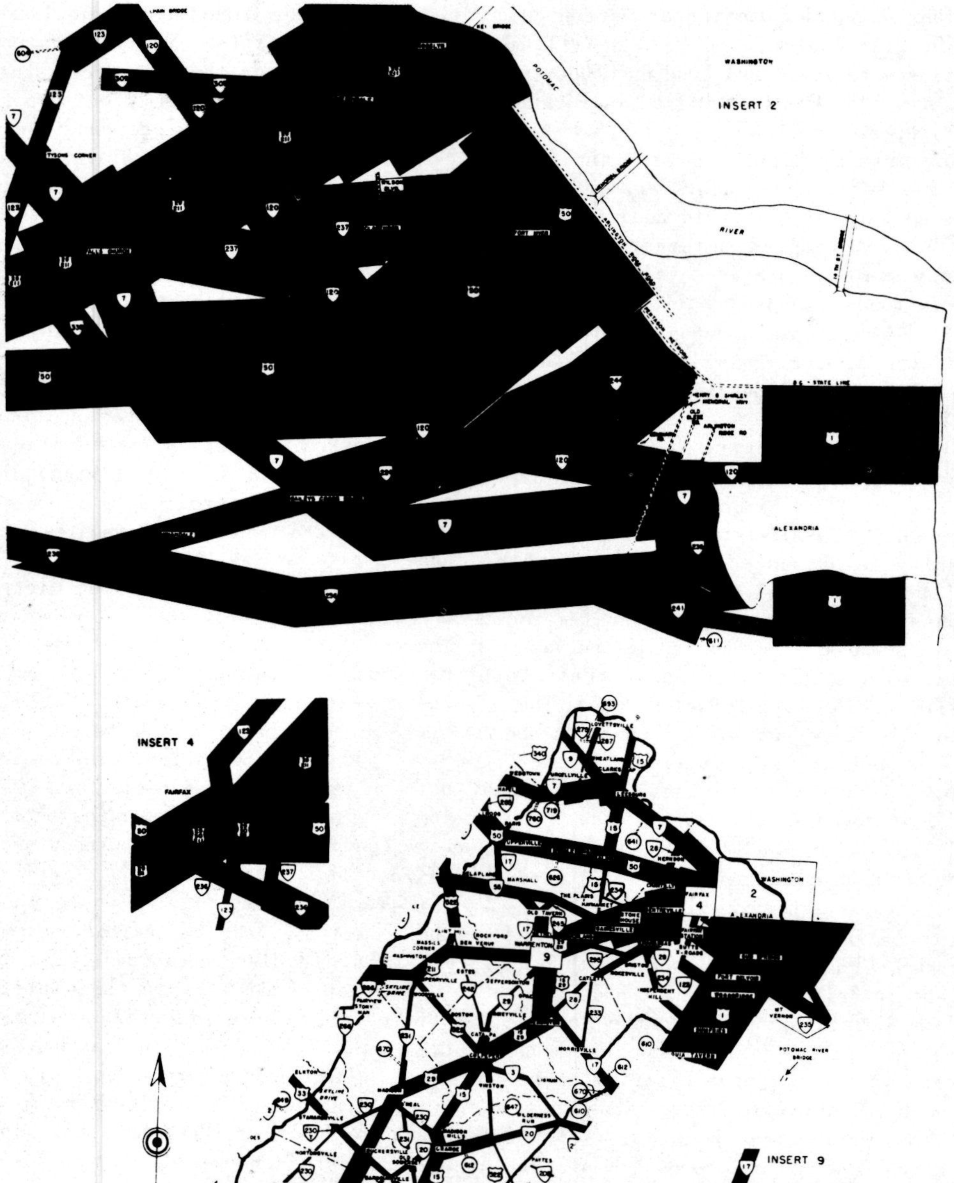

Figure 1. Culpeper District - Traffic Flow Map, 1947 - Traffic Scale: 1/4-in. width = 5000 Vehicles.

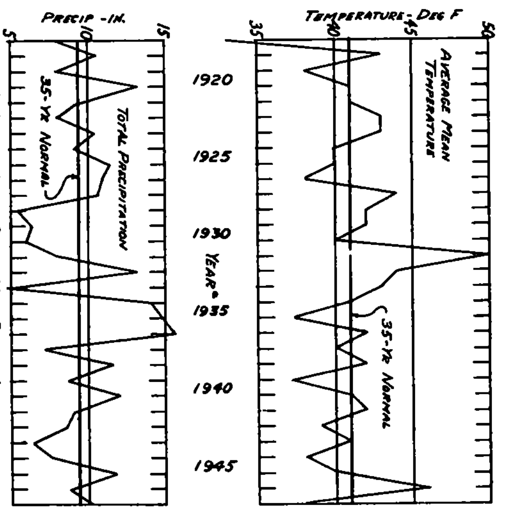

Figure 2. Climatic Data Charlottesville, Va. Data are for November and December of given year plus following January.

In total precipitation the winter of 1936-37 was the wettest with 15.5 in., however, the preceding winter (1935-36) was next with 15 in. These may be compared with a 35-yr. normal for the three months period of 9.3 in. Last winter the precipitation was above normal being 10.4 in.

The most severe spring break-up in recent years occurred in 1935-36. An examination of the chart reveals that an unusually low temperature (second lowest in 30 years) was combined with an abnormally high precipitation (second wettest winter). It may be pointed out that the past winter had the combination of precipitation and temperature most favorable to a severe break-up since 1935-36. The two items appear to go hand in hand and merely a cold winter does not necessarily result in a severe break-up.

Materials for highway construction and maintenance are found in various parts of the district. Sand and gravel in the Coastal Plain are currently being produced at several locations. Natural river sand can also be obtained at several locations where it occurs as an alluvium. Stone from several formations is used as road metal. Several of the granites have been quarried successfully. Quarries opened in the Wissahickon formation have furnished a variety of granitic rocks. A variety of aggregates has also been produced from the Triassic "Red Beds". Several quarries have been operating for a number of years in the conglomerate (trap phase) and in basaltic dykes. The latter are extremely tough with resulting wear on crushers. The Manassas sandstone of Triassic Age also has been used as road metal. Also, greenstone has been a source for road aggregates.

As may be expected, the soils of the area are a reflection of the geology. On the basis of parent materials the district has been divided into five general soil areas as shown in Figure 3. The Coastal Plain soils vary widely from sand and pebbles to interbedded sand and clay. Along the shore line some unconsolidated peat may be found in marshy land. On the Piedmont Plateau the granitic rocks (whether metamorphosed or not) weather into clays to sandy clays. The soil type is a function of the amount of quartz present in the parent material - the more quartz present, the less plastic the soil. As a rule, the amount of quartz in the granitoids varies inversely as the amount of clay forming minerals such as the feldspars, micas and members of the hornblende family. The weathering process which transforms them is not a simple one but it can be stated that through physical and chemical actions they are changed into elastic and (or) plastic soils. Wherever the quartz and other siliceous elements are predominant, the resulting soils are essentially granular and non-plastic (sandy to silty).

The Triassic basin is very properly described as the "Red Bed" area since the soils weather into deep red-brown clay. Though the top horizon is rather shallow near the Potomac, it becomes deeper further south. The soil is one of the most unfavorable for highway work and, as is often the case, it is also among the most favorable in the State from an agricultural point of view. The diabase dykes often weather into a sand before breaking down

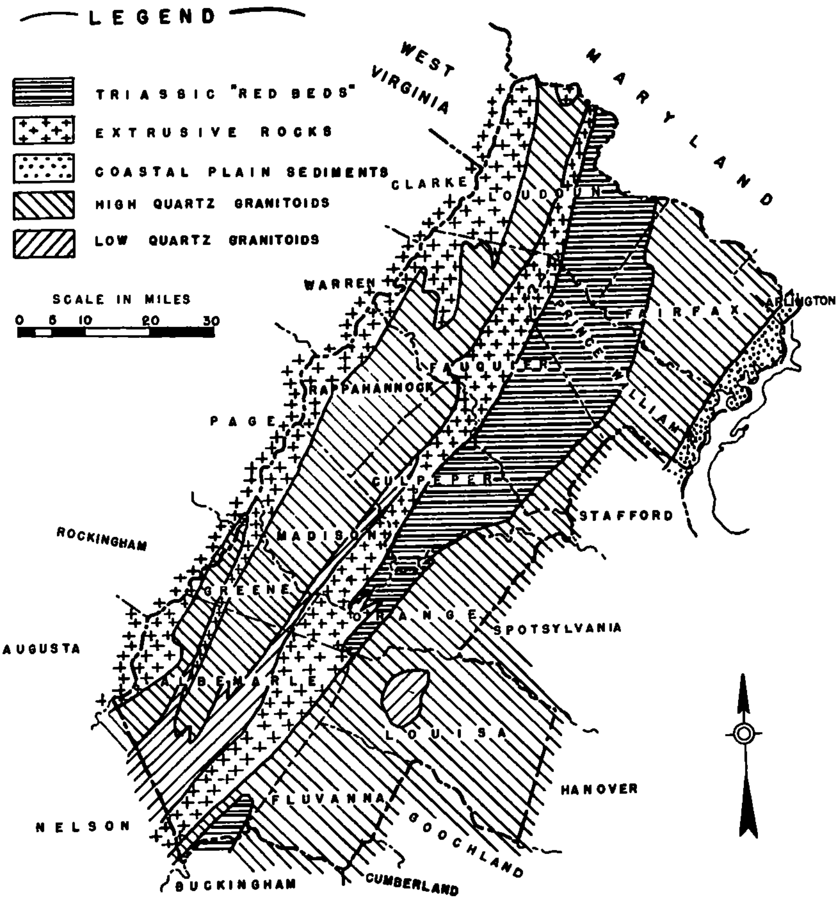

Figure 3. Culpeper District - General Soil Areas.

into a very heavy clay (agriculturally known as the Iredell), which has justly been nicknamed "black jack soil". The Triassic sand as well as the clay is most unfavorable for highway work. The volcanic rocks which are mostly greenstone also break down into clay. As a whole, it may be stated that from a highway standpoint, soils encountered in the Culpeper District present problems because of poor internal drainage and variable bearing power.

SURVEY PROCEDURE

Immediately after the assignment of the state-wide road condition survey to the Research Section, the first step was the preparation of a working plan. Meetings were held with the field forces who were to conduct the survey. Specific instructions were issued to the District Materials Engineers, the District Soil Engineers, Testing Division staff members and field men assigned to the survey. The District Materials Engineer was placed in charge of the field survey and held responsible for its conduct in each district. Four or five field parties, consisting of a driver and a recorder, were organized in each district. Thus, for conducting such an extensive survey in a relatively short time the services of about 100 men were required.

The field parties worked in close cooperation with the District Engineers and Resident Engineers and checked with the latter at least once a day to report extensive or severe failures so that they could be corrected and thus expedite traffic. The driver of the two-man party was usually a road patrolman who was intimately familiar with local conditions. The recorder prepared the log, rated the roads, took pictures to illustrate typical conditions and kept all notes.

Prior to beginning the survey each party was supplied a list of road sections by counties which had been prepared on the I.B.M. machine by the Auditing Division. These tabulations contained the following information. (1) county code, (2) route number, (3) description (from, to), (4) surface type (by code), (5) base type (by code), (6) road width and (7) length. In addition, each party was furnished the following supplies and equipment· field note book (one per county), code of counties, code for base and surface types, county maps, state maps, list of Resident Engineers and counties in each Residency, colored pencils, pick, shovel, scale, camera, supply of film, working plan and set of instructions.

The survey was made by driving at slow speed over each section of road, noting and recording conditions in a field note book. The same general procedure was followed in each of the eight districts. For each road section the following information was recorded in the county field book: (1) survey party, (2) date inspected, (3) weather conditions, (4) log of section locating type and extent of failures, and other information pertinent to performance such as: topography, predominating soil type, position of grade line with respect to ground surface and water table, drainage conditions, unusual traffic conditions, etc.

Immediately after logging and inspecting a given section, an estimate was made of the degree of distress and a rating was given on the basis of the arbitrary evaluations listed in Table 1.

As soon as the rating had been estimated, the road was colored on the county map according to the color code given in the Table. If only surface failures were encountered, short lines perpendicular to the road were colored on the map. When the failures were found in both surface and base, a solid color was used. Thus, uniform rating and color schemes were used in all districts. It is realized that it is not always easy to distinguish between "winter damage" and that resulting from delayed or deferred maintenance. The ratings reflect conditions existing at the time of the spring break-up, regardless of their cause.

About 600 pictures were taken to record typical road conditions and to illustrate both good and poor performance. At the completion of the survey each District Materials Engineer prepared a report and submitted the data for each district. Included in the report was a colored map of each county showing the rating of each road section. The reports contained tabulations showing the number of miles of primary and secondary roads in each rating by counties. Secondary roads were further divided into those with hard surfaces and the ones without. Some of the reports supplied information concerning the different types of construction and some contained recommendations for improvements in design, construction and maintenance procedures.

In order to secure further information concerning the effect of such variables as soil area, base and surface types, an analysis of the survey results in one typical district (Culpeper) was attempted.

CULPEPER DISTRICT SURVEY RESULTS

Primary Roads - Results of the condition survey of primary roads have been summarized according to ratings for each of the 13 counties in Table 2. The summary includes not only the total mileage but also the percentage in each rating. These data are also presented graphically in Figure 4. It will be observed that about 505 mi. or 43 percent of the primary roads were giving good performance (ratings 1 and 2). In contrast, only about 348 mi. or 29.5 percent were considered as giving poor performance (rating 4 and 5). More than 50 percent of the primary roads in two counties, Albemarle and Fairfax, were rated good. Incidentally, these two counties are two of the heaviest populated counties in the District and the primary

TABLE 1

Hard Surface Roads

Rating	Description	Color Code
1. Excellent Performance	Roads showing no break-up and in perfect condition.	Blue
2. Good performance	Roads showing only a slight amount of distress such as an occassional alligator crack or some surface raveling.	Green
3. Slight Distress	Those roads with less than 5% of the total area showing base and surface movement.	Brown
4. Secondary Distress	Those roads showing 5 to 20 % of the total area with movement in the base and surface.	Orange
5. Primary Failure	Those roads with over 20% of the surface showing base and surface movement.	Red
	Non-Hard Surface Roads	
1. Excellent	Remained smooth with no break-ups.	Blue
2. Good	No break-up. Slick in places but no ruts deeper than 1 in. Traffic moving in high gear on entire section.	Green
3. Fair	Not over 5% badly rutted so as to force traffic to change gears.	Brown
4. Poor	From 5% to 20% badly rutted. Traffic may get stuck in places.	Orange
5. Very Poor	Over 20% of surface badly rutted. Very difficult for traffic to pass without getting stuck.	Red

roads in them carry more traffic than in most of the other counties. On the other hand, more than 50 percent of the primary roads in Fauquier, Fluvanna, Greene and Loudoun counties were rated as showing considerable distress (ratings 4 and 5).

A map of the primary highways was prepared to show the condition rating of each road section (Figure 5). It will be noted that considerable mileage of Rts. 7, 15, 17, 28 and 233 were rated as primary failure (rating 5). Typical performance pictures are illustrated in Figures 6-9 inclusive.

Secondary Roads - Results of the survey on secondary roads have been summarized in a similar manner for the twelve counties included in this system (Table 3). The secondary roads have been further divided into two groups: hard and non-hard surfaces. Of the 4958.47 mi. of secondary roads, only 857.48 mi. or 17.3 percent have a hard surface. While both classes of secondary roads were included in the survey, only those secondary roads with hard surfaces are included in the analysis. Conditions of the unsurfaced secondary roads change so rapidly that their

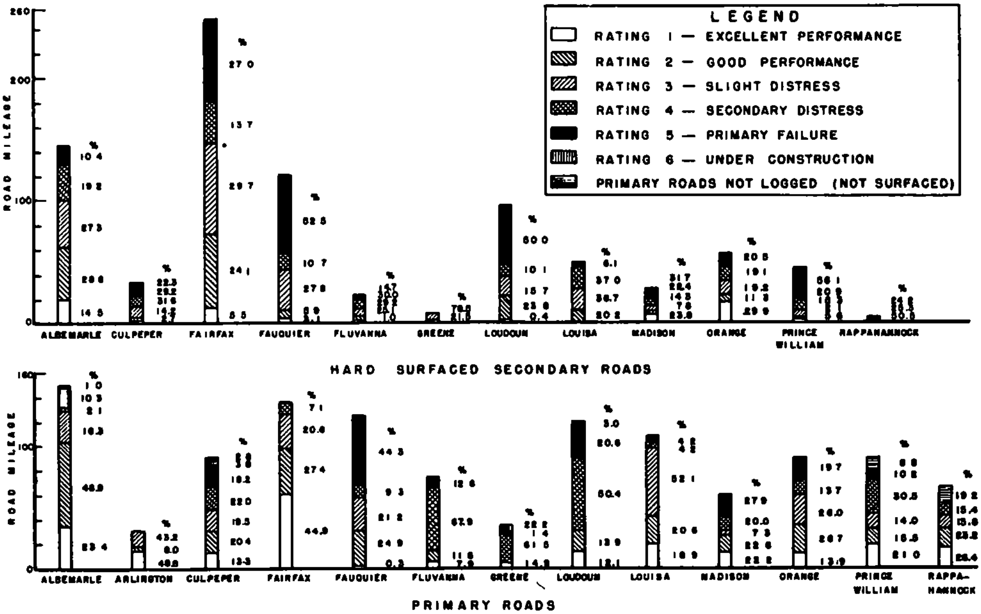

Figure 4. Road Mileage and Condition Ratings by Counties.

rating is extremely difficult. Such a road may be impassible one day and rated as a primary failure; however, after blading and drying by sun and wind its condition may have improved so much as to class it as fair. Failures of hard surface roads are not so easily repaired and tell-tale patches indicate past conditions.

Considering the hard surface secondary roads, 232.35 mi. (27 percent) were rated as giving good performance (ratings 1 and 2) as compared to 399.23 mi. (47 percent) that were showing considerable distress (ratings 4 and 5). In only one county (Rappahannock) were more than 50 percent of the hard surface secondary roads rated as showing good performance (ratings 1 and 2). In contrast, more than 50 percent of the hard surface secondary roads were in poor condition (ratings 4 and 5) in five counties - Prince William, Fauquier, Loudoun, Madison and Culpeper. County maps were prepared showing the rating of each secondary road, however, because of their bulk they are not included in the report. They are quite useful to the Resident and District Engineers for planning maintenance and construction schedules. Conditions typical on secondary roads at the time of the survey are illustrated by Figures 10, 11 and 12.

DISCUSSION OF RESULTS

In order to simplify the analysis of the data, ratings 1 and 2 were combined and classified as good performance. Also, ratings 4 and 5 were combined and reclassified as poor performance. The data for the primary roads was then rearranged and summarized according to soil area by base and surface types (Table 4). It was thought that such a summary might be more revealing concerning the factors upon which road performance is dependent. It is recognized that there are many variations of soil due to topography and other factors within the five general soil areas; however, the areas are based upon predominating parent materials. Information on base and surface types were secured from the road inventories as prepared by the Division of Traffic and Planning. Data on hard surface secondary roads was tabulated in a similar manner and is summarized in Table 5.

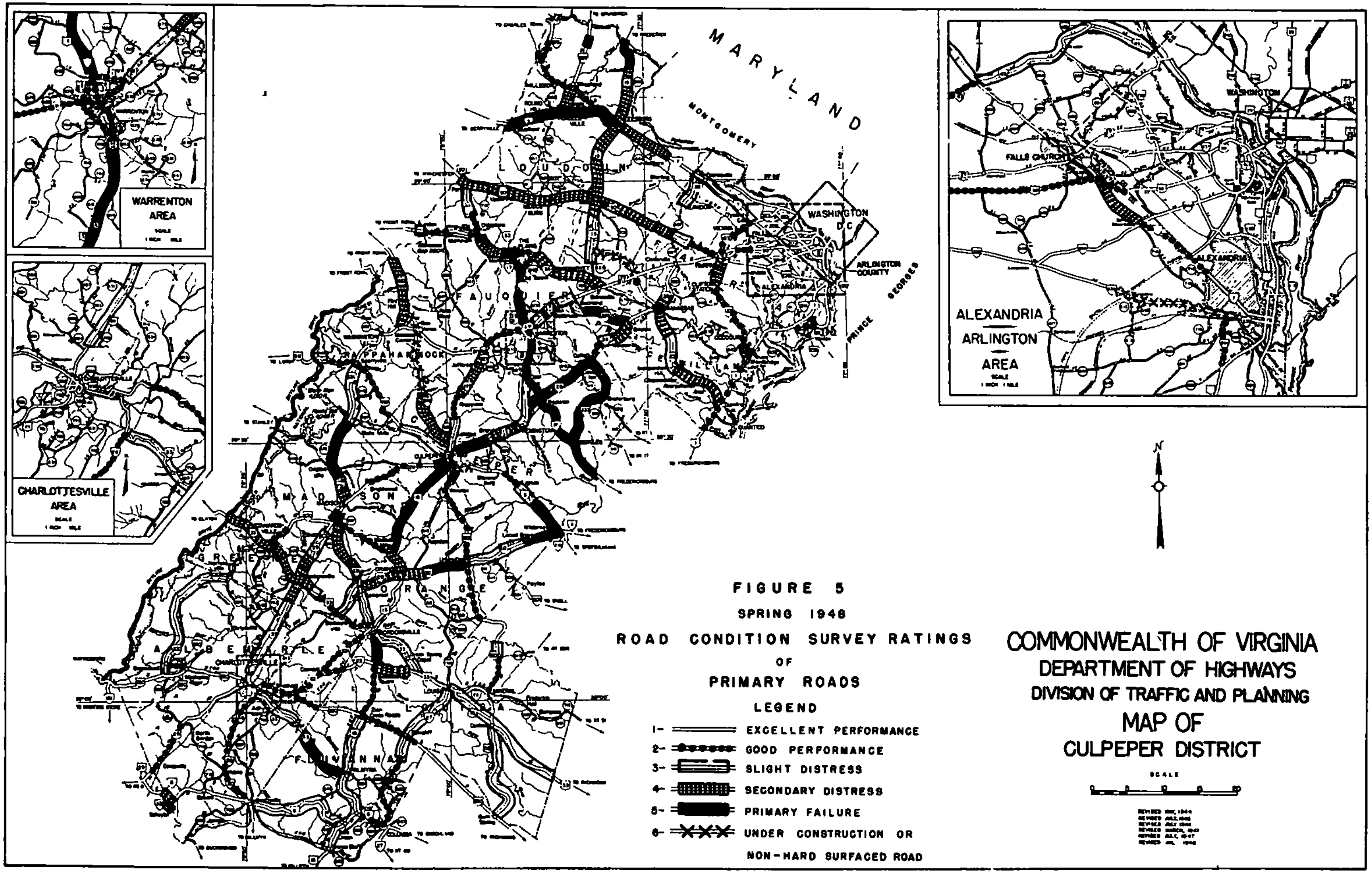

FIGURE 5
SPRING 1948
ROAD CONDITION SURVEY RATINGS OF PRIMARY ROADS

TABLE 2

MILEAGE AND PERCENTAGE BY CONDITION RATINGS

Primary Roads - Culpeper District

County		Condition Ratings 1	2	3	4	5	6[a]	7[b]	Total
Albemarle	mi.	34 95	70.21	24.30	3.12	None	15.49	1.50	149.57
	%	23.4	46 9	16 3	2.1	None	10.3	1.0	100.0
Arlington	mi.	15.30	2.50	13.56	None	None	None	None	31.36
	%	48 8	8.0	43.2	None	None	None	None	100.0
Culpeper	mi.	12.08	18 35	17.71	20.15	16.55	3.50	2.57	91 11
	%	13 3	20.4	19.5	22.0	18.2	3.8	2.8	100.0
Fairfax	mi.	60.78	37.18	27.85	9.62	None	None	None	135.43
	%	44.9	27.4	20 6	7.1	None	None	None	100.0
Fauquier	mi.	0.42	31.00	26 38	11.66	55.15	None	None	124.61
	%	0.3	24.9	21 2	9.3	44.3	None	None	100.0
Fluvanna	mi.	5.93	8.70	51.14	None	9.50	None	None	75.27
	%	7.9	11.6	67.9	None	12.6	None	None	100.0
Greene	mi.	None	5.40	None	22.39	0.50	8.10	None	36.39
	%	None	14.9	None	61.5	1.4	22.2	None	100.0
Loudoun	mi.	14.46	16.55	None	60.18	24.60	3.60	None	119 39
	%	12 1	13.9	None	50.4	20.6	3.0	None	100.0
Louisa	mi.	20.39	22.19	56.01	4.54	4.59	None	None	107.72
	%	18.9	20.6	52.1	4 2	4.2	None	None	100 0
Madison	mi.	13.30	13.55	4.40	12.00	16 80	None	None	60.05
	%	22.2	22.6	7.3	20.0	27.9	None	None	100 0
Orange	mi.	12.44	23 93	23.34	12.23	17.63	None	None	89.57
	%	13.9	26.7	26.0	13.7	19.7	None	None	100.0
Prince William	mi.	18.70	13.80	12.40	27.16	9.03	None	7.81	88.90
	%	21.0	15.5	14.0	30.5	10.2	None	8.8	100.0
Rappahannock	mi.	17.43	15.28	10.39	10 19	None	None	12.70	65.99
	%	26.4	23 2	15.8	15 4	None	None	19.2	100.0
Totals	mi.	226 18	278.84	267.48	193.24	154.35	30.69	24.58	1175.36
	%	19.3	23.8	22.7	16.4	13.1	2.6	2.1	100.0

[a]Under construction
[b]Non-hard surface

Primary Road Bases - Primary road bases were of five types with macadam and natural soil, gravel and stone predominating. These two represent 81.1 percent of primary road bases in the district. Slightly more than 100 mi. of concrete bases and 82.6 mi. of stabilized selected material bases were available for study. Also, one project of soil-cement, 6.8 mi. in length was included. Considering all base types, 44.6 percent were rated good, 22.2 percent fair, and the remaining 33.2 percent classified as poor. The concrete, soil-cement and macadam bases rated above the average for the district. Performance of the stabilized selected materials bases was better than that for the natural soil, gravel, stone, etc. bases.

Figure 13 illustrates variations in performance of the two predominating base types according to soil area. The mileage in percent for each of the three performance ratings is plotted according to the five general soil areas. The upper graphs represent the natural soil, gravel, stone, etc. base while macadam bases are shown below. It will be seen that in all soil areas macadam bases were superior in performance. Both base types performed best when located in the Coastal Plain

TABLE 3
ROAD CONDITION RATINGS BY CLASSIFICATION
Secondary Roads - Culpeper District

County		Hard Surface 1	Hard Surface 2	Hard Surface 3	Hard Surface 4	Hard Surface 5	Hard Surface Sub-total	Non-Hard Surface 1	Non-Hard Surface 2	Non-Hard Surface 3	Non-Hard Surface 4	Non-Hard Surface 5	Non-Hard Surface Sub-total	Under Const. 6[a]	Total Hard Surface, Non-Hard Surface and Under Const. 1	2	3	4	5	6[a]	Total
Albemarle	mi.	21.00	41.41	39.49	27.74	15.15	144.79	46.55	205.76	150.75	81.34	29.66	514.06	None	67.55	247.17	190.24	109.08	44.81	None	658.85
	%	3.2	6.3	6.0	4.2	2.3	22.0	7.1	31.2	22.8	12.4	4.5	78.0	None	10.3	37.5	28.8	16.6	6.8	None	100.0
Culpeper	mi.	0.90	4.69	10.47	9.68	7.35	33.09	None	57.66	171.94	45.60	76.56	351.76	3.80	0.90	62.35	182.41	55.28	83.91	3.80	388.65
	%	0.2	1.2	2.7	2.5	1.9	8.5	None	14.8	44.3	11.7	19.7	90.5	1.0	0.2	16.0	47.0	14.2	21.6	1.0	100.0
Fairfax	mi.	13 62	59 80	73.80	34.04	66.99	248.25	0.85	20 99	74.97	100.48	170.28	367.57	7.64	14.47	80.79	148.77	134.52	237.27	7.64	623.46
	%	2.2	9.6	11.8	5.5	10.7	39.8	0.1	3.4	12.1	16 1	27.3	59.0	1.2	2.3	13.0	23.9	21.6	38.0	1.2	100.0
Fauquier	mi.	3.70	7 14	33.58	12.90	63.62	120.94	4.90	7.10	227.20	158.42	178 82	576.44	None	8.60	14.24	260.78	171.32	242.44	None	697.38
	%	0.5	1.0	4.8	1.9	9.1	17.3	0.7	1.0	32.6	22.8	25.6	82.7	None	1.2	2.0	37.4	24.7	34.7	None	100.0
Fluvanna	mi.	0.20	5.55	6.45	6.65	3 26	22.11	5.70	75.40	62.49	56.20	22.75	222.54	None	5.90	80.95	68.94	62.85	26.01	None	244.65
	%	0.1	2.3	2.6	2 7	1.3	9.0	2 3	30.8	25.6	23.0	9 3	91.0	None	2.4	33.1	28.2	25.7	10.6	None	100.0
Greene	mi.	2.00	None	7.30	None	None	9.30	None	47.19	52.43	11.92	38.21	149.75	None	2.00	47.19	59.73	11.92	38.21	None	159.05
	%	1.3	None	4.6	None	None	5.9	None	29.6	33.0	7.5	24.0	94.1	None	1.3	29.6	37.6	7.5	24.0	None	100.0
Loudoun	mi.	0.35	22.82	15.00	9.78	47.83	95.78	None	88.85	269.60	59.60	126.54	544.59	None	0.35	111.67	284.60	69.38	174.37	None	640.37
	%	0.1	3.6	2.3	1.5	7.5	15.0	None	13.9	42.1	9.3	19.7	85.0	None	0.1	17.5	44.4	10.8	27.2	None	100.0
Louisa	mi.	None	10.00	18.12	18 25	3.00	49.37	None	56.44	114.86	104.40	70.75	346.45	7.30	None	66.44	132.98	122.65	73.75	7.30	403.12
	%	None	2.5	4.5	4.5	0.7	12.2	None	14.0	28.5	25.9	17.6	86.0	1.8	None	16.5	33.0	30.4	18.3	1.8	100.0
Madison	mi.	6.85	2.26	4.10	6.44	9.07	28.72	10.40	53 58	38.61	69.96	98.15	270.70	None	17.25	55.84	42.71	76.40	107.22	None	299.42
	%	2.3	0.8	1.4	2.1	3.0	9.6	3.5	17.9	12.9	23.3	32.8	90.4	None	5.8	18.7	14.3	25.4	35.8	None	100.0
Orange	mi.	16.95	6.44	10.88	10 84	11.86	56.97	6.50	28.49	56.27	58.24	115.10	264.60	None	23.45	34.93	67.15	69.08	126.96	None	321.57
	%	5.2	2.0	3.4	3.4	3.7	17.7	2.0	8.9	17.5	18 1	35.8	82.3	None	7.2	10.9	20.9	21.5	39.5	None	100.0
Prince William	mi.	2.47	0.90	6.70	9.16	24.58	43.81	None	22.92	139.27	79.72	40.01	281.92	None	2.47	23.82	145.97	88.88	64.59	None	325.73[b]
	%	0.8	0.3	2.1	2.8	7.5	13 5	None	7.0	42.7	24.5	12.3	86.5	None	0.8	7.3	44.9	27.3	19.8	None	100.0
Rappa-hannock	mi.	2.20	1.10	None	None	1.05	4.35	None	2.50	35.05	63.05	91.27	191.87	None	2.20	3.60	35.05	63.05	92.32	None	196.22[c]
	%	1.1	0.6	None	None	0.5	2.2	None	1.3	17.9	32.1	46.5	97 8	None	1.1	1.9	17.9	32.1	47.0	None	100.0
Totals	mi.	70.24	162.11	225.89	145.48	253.76	857.48	74.90	666.88	1393.44	888.93	1058.10	4082.25	18.74	145.14	828.99	1619.33	1034.41	1311.86	18.74	4958.47
	%	1.4	3.3	4.5	2.9	5.2	17.3	1.5	13.5	28.1	17.9	21.3	82.5	0.4	2.9	16.8	32.6	20.8	26.5	0.4	100.0

[a]Under construction
[b]Does not include 53.46 miles of secondary road which received no maintenance
[c]Does not include 5.90 miles not shown on map

TABLE 4

PRIMARY ROAD MILEAGE ACCORDING TO PERFORMANCE BY SOIL AREAS, BASE AND SURFACE TYPES

Culpeper District

	Soil Area		Coastal Plain Sediments			High Quartz Granitoids			Low Quartz Granitoids			Extrusive Rocks			Triassic "Red Beds"			Totals for Dist.			
	Performance		Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Totals
Base Type	1[a] Natural Soil, Gravel, etc.	mi.	5.0	0	2.2	43.6	102.7	62.3	0	11.1	2.4	1.3	13.3	10.8	5.9	5.5	72.9	55.8	132.6	150.6	339.0
		%	69.5	0	30.5	20.9	50.6	29.5	0	82.3	17.7	5.1	52.4	42.5	6.9	6.5	86.6	16.5	39.1	44.4	31.8
	2. Stabilized Selected Material	mi.	0	0	0	24.0	17.6	18.8	0	0	0	0	0	8.8	4.9	2.4	6.1	28.9	20.0	33.7	82.6
		%	0	0	0	39.8	29.2	31.0	0	0	0	0	0	100.0	36.6	17.9	45.5	35.0	24.3	40.7	7.8
	3. Soil-Cement	mi.	0	0	0	6.8	0	0	0	0	0	0	0	0	0	0	0	6.8	0	0	6.8
		%	0	0	0	100.0	0	0	0	0	0	0	0	0	0	0	0	100.0	0	0	0.6
	7. Macadam	mi.	2.0	0	0	112.7	24.7	31.1	44.8	0	6.5	84.9	28.8	57.3	37.4	30.2	75.7	281.8	83.7	170.6	536.1
		%	100.0	0	0	66.8	14.7	18.5	87.4	0	12.6	49.6	16.9	33.5	26.1	21.1	52.8	52.4	15.6	32.0	50.3
	8. Concrete	mi.	28.2	0	0	46.5	0	0	1.8	0	0	10.1	0	0	16.2	0	0	102.8	0	0	102.8
		%	100.0	0	0	100.0	0	0	100.0	0	0	100.0	0	0	100.0	0	0	100.0	0	0	9.5
	Totals	mi.	35.2	0	2.2	233.6	145.0	112.2	46.6	11.1	8.9	96.3	42.1	76.9	64.4	38.1	154.7	476.1	236.3	354.9	1067.3
		%	94.1	0	5.9	47.5	29.6	22.9	69.9	16.7	13.4	44.7	19.6	35.7	25.0	14.8	60.2	44.6	22.2	33.2	100.0
Surface Type	5[a] Surface Treatment	mi.	7.0	0	2.2	157.7	145.0	107.2	7.7	11.1	8.9	52.0	40.2	48.7	48.2	38.1	152.3	272.6	234.4	319.3	826.3
		%	76.1	0	23.9	38.5	35.4	26.1	27.8	40.1	32.1	36.9	28.6	34.5	20.2	16.0	63.8	33.0	28.4	38.6	77.4
	6. Plant Mix	mi.	28.2	0	0	67.9	0	5.0	38.9	0	0	34.2	1.9	28.2	13.2	0	2.4	182.4	1.9	35.6	219.9
		%	100.0	0	0	93.1	0	6.9	100.0	0	0	53.2	3.0	43.8	84.6	0	15.4	82.9	0.9	16.2	20.6
	8. Concrete	mi.	0	0	0	8.0	0	0	0	0	0	10.1	0	0	3.0	0	0	21.1	0	0	21.1
		%	0	0	0	100.0	0	0	0	0	0	100.0	0	0	100.0	0	0	100.0	0	0	2.0
	Totals	mi.	35.2	0	2.2	233.6	145.0	112.2	46.6	11.1	8.9	96.3	42.1	76.9	64.4	38.1	154.7	476.1	236.3	354.9	1067.3
		%	94.1	0	5.9	47.5	29.6	22.9	69.9	16.7	13.4	44.7	19.6	35.7	25.0	14.8	60.2	44.6	22.2	33.2	100.0
	Totals by Soil Areas	mi.	37.4 mi. (3.5%)			490.8 mi. (46.0%)			66.6 mi. (6.2%)			215.3 mi. (20.2%)			257.2 mi. (24.1%)			1067.3 mi. (100.0%)			

[a]Traffic and Planning Division Code Numbers

Figure 6. Failure of A Primary Highway (Surface Treated Water-Bound Macadam) - Rt. 29, Culpeper County - This portion of the road is located in Triassic "Red Bed" soil area. Approximately 60 percent of the primary roads in this soil area were rated as poor at the time of the spring break-up.

Figure 7. Failure of a Surface Treated Selected Soil Base Road in the Triassic "Red Bed" Soil Area - Rt. 15, Culpeper County - Note typical topography and drainage for this soil area.

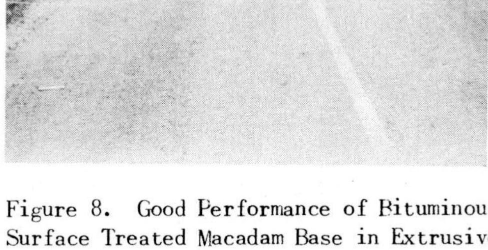

Figure 8. Good Performance of Bituminous Surface Treated Macadam Base in Extrusive Rocks Soil Area - Rt. 15, Orange County.

Sediments soil area. Poorest performance was obtained in the Triassic "Red Bed" soil area.

Hard Surface Secondary Road Bases - According to the road inventories, hard surface secondary road bases include 689.2 mi. (80 percent) stabilized selected materials, 160.9 mi. (18.9 percent) macadam

Figure 9. Alligator Cracking and Disintegration of Surface Treated Selected Soil Base Road in High-Quartz Granitoid Soil Area - Rt. 3, Orange County. Because of their high silt content these soils are subject to frost action.

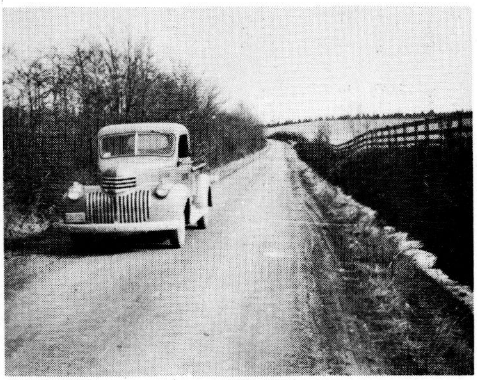

Figure 10. Failure of a Surface Treated Mechanically Stabilized Base Secondary Road in the Triassic "Red Bed" Soil Area Rt. 669, Culpeper County.

Figure 11. Excellent Performance of a Surface Treated Macadam Base Secondary Road, Rt. 627, Orange County.

and only 3.3 miles of concrete bases. In performance the concrete bases rated best followed by the macadam and the stabilized selected materials. Performance of the stabilized selected material and macadam bases according to soil area are illustrated graphically in Figure 14. Again, base performance is variable depending upon the soil area in which it is located. Considering only the stabilized selected material base (which constituted more than 80 percent of the bases), performance was

TABLE 5

HARD SURFACE SECONDARY ROAD MILEAGE ACCORDING TO PERFORMANCE BY SOIL AREAS, BASE AND SURFACE TYPES

Culpeper District

	Soil Area		Coastal Plain Sediments			High Quartz Granitoids			Low Quartz Granitoids			Extrusive Rocks			Triassic "Red Beds"			Totals for Dist			
	Performance		Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor	Totals
Base Type	2[a] Stabilized Selected Materials	mi	32.2	28.3	24.9	50.0	97.8	214.8	16.3	16.9	14.6	20.8	29.9	35.0	28.3	14.4	65.0	147.6	187.3	354.3	689.2
		%	37.7	33.2	29.1	13.8	27.0	59.2	34.1	35.4	30.5	24.3	34.9	40.8	26.3	13.4	60.3	21.4	27.2	51.4	80.7
	7. Macadam	mi	0	0	0	31.5	11.7	6.4	37.4	5.9	3.3	7.0	9.6	17.7	5.7	9.6	15.1	81.6	36.8	42.5	160.9
		%	0	0	0	63.5	23.6	12.9	80.2	12.7	7.1	20.4	28.0	51.6	18.8	31.5	49.7	50.7	22.9	26.4	18.9
	8 Concrete	mi	1.8	0	0	0.2	0.7	0	0	0	0	0.6	0	0	0	0	0	2.6	0.7	0	3.3
		%	100.0	0	0	22.2	77.8	0	0	0	0	100.0	0	0	0	0	0	78.8	21.2	0	0.4
	Totals	mi.	34.0	28.3	24.9	81.7	110.2	221.2	53.7	22.8	17.9	28.4	39.5	52.7	34.0	24.0	80.1	231.8	224.8	396.8	853.4
		%	39.0	32.4	28.6	19.8	26.7	53.5	56.9	24.1	19.0	23.5	32.8	43.7	24.6	17.4	58.0	27.2	26.4	46.4	100.0
Surface Type	4[a] Light Surface Treatment	mi	32.2	28.3	24.9	81.5	109.5	221.2	51.5	22.8	17.9	25.3	37.0	52.7	34.0	24.0	79.8	224.5	221.6	396.5	842.6
		%	37.7	33.2	29.1	19.8	26.6	53.6	55.9	24.8	19.3	22.0	32.2	45.8	24.7	17.4	57.9	26.6	26.3	47.1	98.7
	5. Heavy Surface Treatment	mi	0	0	0	0	0	0	2.2	0	0	2.5	2.5	0	0	0	0.3	4.7	2.5	0.3	7.5
		%	0	0	0	0	0	0	100.0	0	0	50.0	50.0	0	0	0	100.0	62.7	33.3	4.0	0.9
	8 Concrete	mi.	1.8	0	0	0.2	0.7	0	0	0	0	0.6	0	0	0	0	0	2.6	0.7	0	3.3
		%	100.0	0	0	22.2	77.8	0	0	0	0	100.0	0	0	0	0	0	78.8	21.2	0	0.4
	Totals	mi.	34.0	28.3	24.9	81.7	110.2	221.2	53.7	22.8	17.9	28.4	39.5	52.7	34.0	24.0	80.1	231.8	224.8	396.8	853.4
		%	39.0	32.4	28.6	19.8	26.7	53.5	56.9	24.1	19.0	23.5	32.8	43.7	24.6	17.4	58.0	27.2	26.4	46.4	100.0
	Totals by Soil Areas		87.2 mi. (10.2%)			413.1 mi (48.4%)			94.4 mi. (11.1%)			120.6 mi (14.1%)			138.1 mi (16.2%)			853.4 mi. (100.0%)			

[a]Traffic and Planning Division Code Numbers

Figure 12. Distress of a Surface Treated Mechanically Stabilized Base Secondary Road - Rt. 609, Culpeper County.

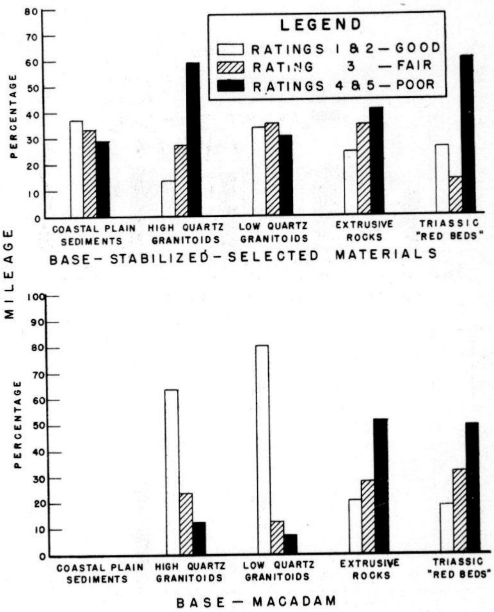

FIGURE 14

SECONDARY ROAD PERFORMANCE BY BASE TYPES

Figure 14. Secondary Road Performance by Base Types.

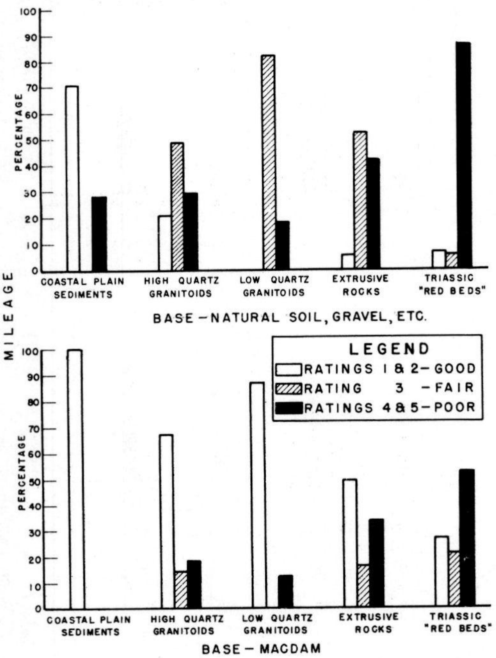

Figure 13. Primary Road Performance by Base Types.

best in the Coastal Plain and poorest in the Triassic soil area.

Primary Road Surfaces - Primary road surfaces are classified into three types by the road inventory, namely surface treatment (77.4 percent), bituminous plant mix (20.6 percent), and portland cement concrete (2.0 percent). All concrete surfaces were rated good in performance. Bituminous plant mix surfaces were generally good and 82.7 percent of them were so rated. Only those that were 8 or 10 yr. old and in need of a seal coat rated fair or poor. About one-third of the bituminous surface treatments rated good, while 38.6 percent were classified as poor. Figure 15 illustrates performance of plant mix and surface treatment.

Secondary Road Surfaces - While the surfaces of secondary roads were of four types, light surface treatments predominate since 98.7 percent were of this category. Performance in the five general soil areas is illustrated by Figure 16.

To further illustrate the correlation of road performance with soil areas Figure 17 has been prepared. The top portion shows that primary road performance is best in the Coastal Plain Sediments and poorest in the Triassic "Red Bed"

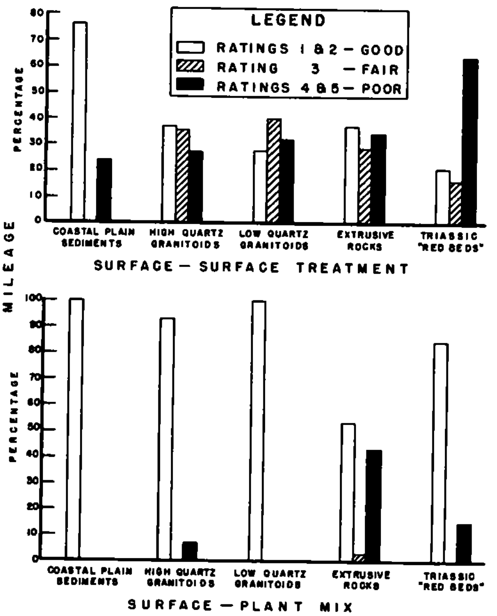

Figure 15. Primary Road Performance by Surface Types.

soils area. In general, this is further emphasized by the lower graphs.

SUMMARY OF RESULTS

Based upon the performance data obtained at the time of the spring break-up on more than 1000 mi. of primary and nearly 5000 mi. of secondary roads in Culpeper District, the following results have been summarized under appropriate headings.

Primary Roads

1. The break-up of this past spring of primary roads in the Culpeper District was more severe than that for the entire state. A comparison of primary highways in each performance rating is as follows:

Performance Rating	Culpeper District %	Entire State %
Good	43	58
Fair	23	23
Poor	29	18
Under Const.	5	1

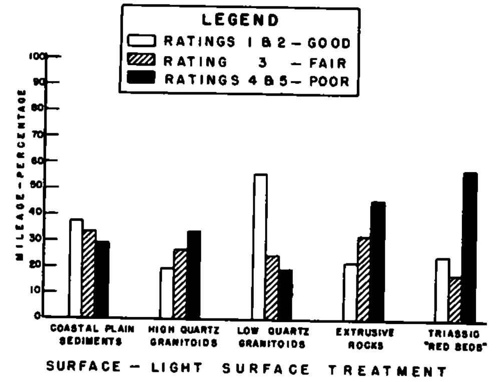

Figure 16. Secondary Road Performance by Surface Types.

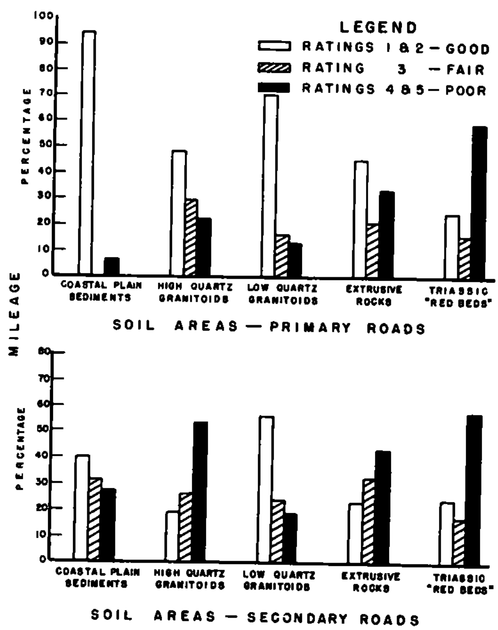

Figure 17. Primary and Secondary Road Performance by Soil Areas.

2. Primary road performance was variable, depending upon the design (base and surface type) and the soil area in which the road was located.

3. The order of rating the performance of bases from best to the poorest was as follows: (1) portland cement concrete, (2) soil-cement, (3) macadam, (4) stabilized selected materials and (5) natural soils, gravel, stone, etc. It

should be pointed out, however, that only one soil-cement project, 6.8 mi. in length was available for this comparison.

4. Considering the bituminous surfaces which comprise 98 percent of the primary road surfaces in the district, the performance of bituminous plant mix was far superior to that of ordinary surface treatment.

5. The study emphasized the importance of the general soil area as a major variable in road performance. Considering all the primary roads, the least amount of distress was found in the Coastal Plain Sediments and the most was evident in the Triassic "Red Bed" soil area.

Secondary Roads

6. Survey results also show that the spring break-up was more severe on secondary roads in the Culpeper District than for the entire secondary system in the state. This statement is based upon the following comparison:

SECONDARY ROAD PERFORMANCE

Performance Rating	Culpeper District	Entire State (8 Districts)
	%	%
Good	19	33
Fair	33	29
Poor	47	37
Under Const.	1	1

It should be emphasized that a greater percentage of Triassic "Red Bed" soil area is located in the Culpeper District than in the entire state and this probably is largely responsible for the differential performance.

7. The hard surface secondary roads performed slightly better than the non-hard surface ones as shown below:

Performance Rating	Hard Sur. %	Non-Hard Sur. %
Good	27	18
Fair	26	35
Poor	47	47

8. As in the case of primary highways, secondary road performance also varied with the design (base and surface type) as well as the soil area.

9. The order of rating secondary road bases from best to poorest in the Culpeper District was as follows: concrete, macadam and stabilized selected material. More than 80 percent of hard surface secondary road bases in the district were of the latter type.

10. While insufficient data were available for conclusive comparisons, it was indicated that heavy surface treatments resulted in better performance than light surface treatments.

General

11. The survey revealed that the higher the class or type of pavement the better the performance.

12. A study of weather records revealed that pavement break-ups are most severe for those winters with low temperatures preceded by subgrade and base saturation (high precipitation). The past winter was second only to that of 1935-36 as to climatic conditions favorable for a severe break-up.

13. The field studies throughout the state emphasized the importance of adequate provisions for drainage if good road performance is to be secured. In cases of flat topography it was indicated that improved performance can be secured by the use of a high level profile.

14. One of the most important results of the survey was the correlation of road performance with soil area. This suggests that design and construction practices should be varied with this important item.

CONCLUSIONS

In conclusion, road condition surveys, if made at a time of a severe spring break-up, are a practical means of securing information on the extent of distress and can be used successfully in evaluating factors affecting performance. Data from such surveys can also be used in conjunction with maintenance or reconstruction programs and for formulating policies regarding design and construction practices.

ACKNOWLEDGEMENTS

The writers wish to express their appreciation to the large number of individuals who have assisted from time to time in various phases of the studies. Especial thanks are due Mr. C. S. Mullen, Chief Engineer, for his foresight in initiating the study and for his interest in the data secured. Mr. A. W. Furgiuele, Culpeper District Materials Engineer, is to be commended for his painstaking and efficient manner of securing and reporting the field data. Practically all of the personnel of the Research Section have contributed in one way or another to the report.

PAVEMENTS AND INFLUENCES AFFECTING OR DETERMINING THEIR PERFORMANCE

A. T. BLECK, *Construction Engineer*
State Highway Commission of Wisconsin

SYNOPSIS

The extremely variable condition of pavements of various ages and design, together with the early failure of many new pavements, induced a study to ascertain, if possible, the causes therefor. A survey was begun in the fall of 1942 of about 3,000 miles of pavement throughout the state, in which it was endeavored to associate specific pavement condition to the circumstances of its exposure.

Agricultural soil maps were used as a base, and the soils as shown by them were correlated to the geology of their formation and deposition, and the character and proximity of bedrock and other subsurface conformations.

Certain fundamental relationships appeared to be manifest. Longitudinal cracking occurs in soils of certain characteristics and not in others. "D" cracking is associated only where the pavements, regardless of type of aggregate, were founded in soils of the calcereous glacial drift, and not in others. Faulting occurs at joints with or without load transfer dowels and is a function of the soils. Rhythmic waviness and blowups of rigid pavements are relatable to the soil-moisture conditions of the subgrade soils rather than to temperature stresses within the slab.

The study of pumping and the characteristics of the subsequent failure of slabs is indicative of pavement action and stresses other than commonly assumed, and a discussion of this is presented.

Aggregate base courses constructed to high densities and supporting impervious bituminous pavements have been found to suffer a complete loss of density and were waterlogged in their lower zones, with the underlying pervious material being only slightly damp

The extraordinary destruction of pavements during the last decade is due to a greater incidence of heavy loads coming coincident with a reversal of the meteorological cycle from the general drouth conditions of the preceding decade and a half.

Due to the presence of an impervious pavement and the energy of moving loads transmitted through the pavements, the moisture conditions in the subgrade soils are not comparable to those in soils freely exposed to the atmosphere.

Observations of the position of moisture in the soils and other phenomena of moisture movement, together with a study of literature on the subject, including so-called frost action, lead to the development of an explanation of these phenomena more compatible with observed manifestations than the popular theory of capillarity.

The subject is concluded with a discussion on sub-base courses, with particular reference, based on experience, to the operation and functioning of freely draining permeable sand or sand-gravel lift or ballast courses

The ensuing discussion on highway pavements sets forth the investigation and study that was made to determine, if possible, the causes of the failures of pavements, so that with the knowledge of the factors generating failures as well as those which determine good performance, effective steps may be taken to produce designs which would be immune to serious failure, thereby rendering improved highway service to the public at lower maintenance and upkeep costs, even though, if necessary, some additional cost might be incurred in the initial construction.

The geological conformations of Wisconsin are distinctive, if not unique, whereby it was possible to associate some variables in soils and pavement performance to the geological influences, where other methods of association seemingly failed to be conclusive.

The climatic or meteorological conditions in Wisconsin are severe, affording an opportunity to evaluate their influence without the necessity of fine differentiations, or refined measurements.

In the design and construction of pavements a conscientious effort was extended through the years to follow the current developments, both with respect to soils work and detail of pavement design. Regardless of this effort, the pavements under service yielded variable results, which it seems were quite incompatible with those expected from the design premises. These inconsistencies gave rise to the endeavor to ascertain, if possible, the fundamental reasons therefor.

While some of the inherent detail desirable for more precision has not as yet been entirely explored, due to lack of time and available personnel, it is believed that an understanding of some of the major contributory causes of specific pavement performance has been attained and a base established upon which future designs can be effected with more definite assurance of yielding satisfactory results over an extended period of time, and upon which more objective study can be made.

Some of the basic principles expressed are not new or novel, others have not been as closely or as specifically associated with pavement performance, and still others present perhaps an entirely new concept.

The synthesis of these several elements, and their application or adaptation toward the evaluation of pavement performance and the extension thereof to design purposes, is believed to represent a departure from previously held concepts for these purposes.

The Modern Era of Rural Pavement Design and Construction can be conceived to have had its inception with the passage of the first Federal Aid Highway Act, about three decades ago. During the intervening period many miles of rural pavements have been built. At the present time these pavements, of all ages, present a wide range of condition and state of necessity for repair or reconstruction. Some pavements have been rebuilt because of geometric obsolescence, others because of structural failure. Some of the oldest ones are still functioning in a satisfactory manner with a minimum of annual maintenance, while some, which have been in service for only about ten years, and built in accordance with the latest principles of design, are giving evidence of a necessity for substantial strengthening if their remaining service value is to be salvaged.

Wisconsin is perhaps not unique in this respect, because of the general interest displayed on a nationwide basis to find ways and means of a solution of the problems generated by the deficiencies in pavement performance encountered on a widespread scale.

It can be stated that in order to develop methods that will prevent unsatisfactory conditions, the causes therefor must be ascertained, for if the fundamental reasons for these conditions are not known any remedies that may be applied will prove to be temporary palliative measures only, through which the basic deficiencies will assert themselves sooner or later in one manner or another.

In the effort to find the causes for

deficient performance a vast amount of study and research has been done by various organizations, and their methods, and sometimes their findings and conclusions and bases for design, are reported in the literature.

These include, among others, investigations of the soils, both in the laboratory and in the field, studies of the constituent components of pavements, and the construction of full-sized test sections of highways containing variables in the design of the pavement and built on what are presumed to be subgrades in which the soils appear to have quite comparable properties. Some test sections are observed under actual traffic conditions, while others have been studied under specific tests.

There are under service, throughout the country, thousands of miles of pavement under various conditions of exposure to traffic and the elements. They represent all age groups, features of design, and rest on subgrades representative of soils of all types and constitution, which can be studied, and the circumstances causing their specific performance evaluated.

In actual experience with these pavements it has long been a matter of rather common knowledge that pavements laid on natural sand or gravel subgrades generally yielded quite satisfactory performance, even though this is not always true in all cases, while on the other hand there are also sufficient examples of pavements resting on subgrades of so-called poor soils that have yielded excellent performance.

This seemingly contradictory evidence gives rise to speculation as to the reasons therefor.

Differentials in the traffic characteristics could not alone be held responsible, because the differentials often take place on individual sections of the same highway, and in some cases the heavier travelled portions yield evidence of a better condition than some with lesser traffic volumes.

Differentials in the design features of the pavements themselves can also not be held accountable, because in the long range period and the widespread distribution of pavements with comparable design features quite a varied series of pavements employing all of the customary modes of design can be found in most locations, and some of the pavements designed to the more modern concepts of design theory suffered as badly, and in some cases worse, than some pavements constructed to what are conceived as rather archaic methods of design.

The factor of construction materials and methods does not appear to be the one essentially accountable, because of a distribution similar to that of the design features, as well as that through the years, tests made both on prepared specimens and on cores drilled from the completed work consistently yielded results indicative of a high quality product, if inherent strength can be accepted as a measure of quality.

The foregoing attempts to depict a generalized view of the existent situation. The confused aspects prompted an endeavor to try to ascertain or isolate, if possible, the elements or features causing the variable conditions.

As an initial venture, a study or examination of pavement conditions was begun about six years ago, and in this it was endeavored to associate pavement performance with the soils upon which they lay, as classified by the Pedological Classification of Soils, and as depicted on the available soil maps. This survey covered about 3,000 miles of rigid pavement, and about 500 to 1,000 miles of flexible type pavements, located in the several geological provinces of the state.

In this first venture it was endeavored to keep a statistical record of the type and degree of failure, as well as of satisfactory performance as related to the various soils types and classifications. A subsequent attempt to catalogue these in order to try to effect a direct correlation resulted in confusion. Soils of the same series, for example, in one part of the state showed failure to marked degree, to a lesser degree in another part of the state, even though the highway carried considerably more heavy traffic, and virtually no failure in a third part of the state on a highway which carried comparable traffic to the one where the worst degree of failure occurred.

A stalemate seemed to have been reached when the probability of the geological influence suggested itself through the fact that in the northern part of the state the gravels are of crystalline rock formation, while in the southern part of the state they are largely dolomitic in character, with an intermediate zone wherein the two types are intermixed with each other.

In the hope that the geological features would offer some basis upon which an isolation of some of the factors causing the variable pavement performance could be made, a review of the geology of the state was undertaken. Since this appeared to offer possibilities, and to be another step in the solution of these problems, a brief outline of the geology of Wisconsin is presented.

Wisconsin is underlain by the crystalline rocks of Pre-Cambrian age, and in roughly the northern half of the state, in a large shield-shaped area, these form the immediate bedrock formations, which are close to the surface in many places.

Around this shield-shaped area, in a roughly cresential form, toward the south, east, and west the later sedimentary deposits have been placed in sequence, beginning with the Cambrian sandstones and shales, and following with the Ordovician limestones and sandstones, the Silurian shales and limestone, and in a very small area the Devonian shale. These lie upon each other much as shingles on a roof, except that the butt ends of the shingles protrude upward, and quite distinctively form the immediate bedrock formations in the different portions of the state.

Since the land-forming processes in Wisconsin ceased early in its geological life, its surface had been subjected to weathering and erosion many ages before even some of the immediately contiguous territory emerged from the seas.

However, due to glacial action, much of the mature topography had become obliterated, ancient drainage courses were blocked, and new ones were developed, and youthful soil profiles were established in many places.

About three fourths of the area of the state has been subject to the ice invasions of the several glacial periods, ranging from the Kansas and Nebraskan, through the Illinoian and the several Wisconsin stages, and while most of the areas of the older glaciation have been overridden by that of the Wisconsin stages, there are areas where the older glacial deposits are still exposed. The westerly and southwesterly one fourth of the area of the state was never influenced by glaciation, except that a considerable area in the central part of the state was submerged under the waters of a now extinct glacial lake. This unglaciated area is known as the Driftless Area.

In the further investigation of pavement performance, not only was the bedrock geology of the state taken into account, but consideration was also extended to the geology of the soils, and the geological processes of their formation, deposition, modification, and development.

In the matter of glacial soils, Thwaites*(1)*[1] and others point out that the general glacial movement was like the flattening of a drop of very viscuous liquid, and covered distances of a few miles only, rather than one of concepts which hold movements of masses of soils over great distances by the ice.

Since the soils are the residua formed by the decomposition of rock, and since there were ages prior to the ice invasions, during which the surface of the earth was subject to the weathering influences causing such decomposition, accompanied also by erosion, the soils in a given region, even in the glaciated areas, are the products of the bedrock of such area. Glacial action modified them to a degree by working them over and intermixing with them the products of glacial movement, such as rock flour, rock fragments, and

[1]Italicized figures in parentheses refer to the list of references at the end of the paper.

similar material. Glacial effluent deposited sands, gravels, silts, clays, and lacustrine deposits of similar materials, some of which were, while others were not, covered by later glacial materials.

Windblown soils during glacial periods and post-glacial times deposited materials upon the ice sheets, the glacial drift, and upon the residual soils in the Driftless Areas. These form the surficial soils found today in many areas.

Calling to mind through experiences with excavations in the materials, some of the depositional forms of glacial soils and soils deposited by glacial effluent provided the background for the further evaluation of other soil materials.

The large masses of ground moraines, deposited in place by the glaciers without subsequent movement due to glacial action, consist largely of unassorted till. This material may be silt, clay, sands, pebbles, cobbles, and boulders, often intermixed with each other. However, the dominating fractions show a strong influence of the bedrock formations of the proximate area.

Occasionally, strata of sand or gravel or strata of clay or silt are sandwiched between layers of the till, indicative of an outwash or other fluvial deposit of an earlier stage of glaciation or from a recessional stage of the same period, which was later overridden by a re-advance of the ice sheet. Such layers are quite generally water-bearing or water-holding strata.

Terminal moraines, or recessional moraines often are apt to be of a rather gravelly nature, with, however, a poor sorting of the several materials. These were piled up at the ends of glacial ice where it stood at points of its farthest advance and contain, therefore, more of the coarser material, and were also subjected to a great degree to the washing action of the melt water from the stagnating glaciers.

Valleys existing in these moraines during glacial times are now often found to be filled with windblown silt.

Materials carried by the melt water formed the glacial outwash and glacial lake deposits. An examination of these deposits shows good to excellent sorting of materials with respect to size, although most of them are rather highly laminated into strata of different sized materials. As can be seen from Figure 1, the stratafication of outwash materials is quite readily discernible, ranging or consisting of strata of sand and gravel, or strata of coarser and finer sands to strata of very fine sand and silt.

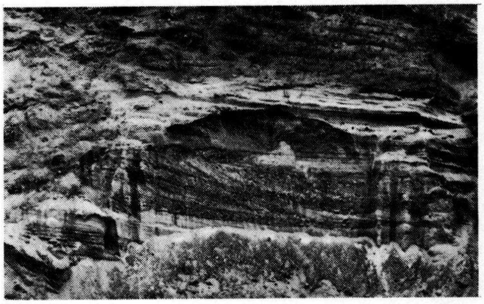

Figure 1.

It has been repeatedly observed that strata of fine sand or silt occurring in these deposits, sandwiched between layers of coarser materials, and whether they have a manifest connection to an exterior source of moisture or not, invariably contain substantial quantities of moisture, up to the saturation point.

Stratified beach deposits of fine sand and silt in now extinct glacial lakes yielded similar evidence of a comparable moisture relationship, the silt being the wetter.

Varved clays consisting of rather thin alternating layers of fine silt and clay are found in lacustrine deposits.

Among some of the more recent deposits composed of overwash or alluvial soils from higher lying areas, and subsequently deposited, a high degree of stratification and moisture differentiation has been observed.

The deposition of water-borne materials into strata takes place in accordance with Stoke's Law. The coarser materials drop out in the more turbulent or violent medium, while the finer ones are carried in suspension much longer or are carried farther. Since the amount of water, even from glacial effluent, was not constant, the velocities varied, thereby causing alternating stages of variable turbulence or velocity, creating the stratification in these deposits.

Comparable differences exist in the soils deposited in bodies of waters such as lakes. During stormy periods coarse material is deposited, while the converse is true under quiescent conditions. Deep water depositions are most apt to be stratifications of fine material, while along the beaches strata of the heavier materials are found.

Similar to water-borne and deposited materials, wind-borne materials were deposited in accordance with the principles of Stoke's Law. However, since the particle size is much smaller and the volumes capable of being borne through the air at any given time are much less than in water-borne materials, the stratification is apt to be quite insensible, but it can be postulated that it exists.

The element of stratification in the soils column may appear to have been stressed considerably, and this has been done because stratification is one of the most dominant features determining the position of underground moisture sources, since it acts as an inhibitor to the free internal drainage of the soils column. Whether or not they are manifest at all times, the finer strata, even though they overlie strata of coarser material, are potential zones of, or approaching saturation, the influence of which on the subgrade materials is dependent upon its relative position to the grade line.

In ordinary methods of soils sampling this stratification is often destroyed, thereby yielding a sample not truly depicting the characteristics of the soils, or it may not be recognized, wherefore tests on them may yield results incompatible with the true conditions.

Figure 2 shows a stratified fluvial deposit, the stratification of which might be difficult to detect by ordinary methods of soil sampling, especially boring.

In the Driftless Area, the soils underlying the wind-borne deposits where these exist, and where not, the surface soils themselves are the residual material of the bedrock, either of the formation upon which they now lie, or of the previous overlying formation now completely weathered, or a combination thereof. These range from the residual clays of the limestone formations, through the range of silt loams, loams and sandy loams of the shale formations, to the pure residual sands of the sandrock formations.

Figure 2.

Many of these silts are of a rather uniform grain size, affording percolation, but at a slow rate, and observations have been made, where these overlie the residual clay even on ridge locations, of a saturated condition of the silt prevailing at various depths below the surface. In other cases, where these silts were of considerable depth, they were saturated at the lower levels even though they overlay a coarser material.

Another study or survey of pavement performance was subsequently undertaken on the basis of the Pedological Classification of the soils, but correlating these concurrently with the geological background of the soils, bedrock, and related features.

Being more cognizant of the soil moisture relationships, it was also endeavored wherever possible to evaluate these potentials through clues furnished by the characteristic vegetation, culture, and similar features.

This investigation or survey, taking these features into concurrent account with the soils classification as shown by the Pedological Soil Maps, established some generalized though significant differentials in pavement performance.

Longitudinal cracking in rigid pavements, having a center joint, is almost entirely absent in the soils of the Driftless Area, except where the influence of the soils deposited in the now extinct glacial lake is exerted. In flexible type pavements in this area the turtle back cracking usually begins and is more severe in the center of the pavement even though, as in the case of the formerly used feather edge type of base course, the base is thicker at the center than at the edges. In the glaciated area the converse appears to be the rule. In the glaciated area longitudinal cracking in pavements with a longitudinal center joint appears to be most dominant in the red clay glacial till; in soils of certain regions of the Older Drift, the calcereous drift in the region of the Niagara limestones and Maquoketa shales, it diminishes as the influence of the St. Peter sandstone affects the calcereous drift; and it is present to some degree in the soils in the regions of the igneous bedrock. Longitudinal cracking is entirely absent in many of the glacial outwash deposits.

The pumping of rigid pavements is almost entirely unknown in the soils of the Driftless Area, although in a few cases in this region fine sands have been extruded through cracks and joints. Pumping is also quite foreign to the soils of the igneous glacial drift.

"D" cracking, sometimes attributed to the dolomitic characteristics of the aggregates, has been found only on soils of the calcereous glacial drift, including the red clay till, even though pavements of dolomitic aggregates were built in the other areas.

Blow-ups of rigid pavements without expansion joints have been found to associate themselves with soils conditions.

In the central part of the state, in the region of the Older (Illinoian) Drift, there is a large area in which the soils are classified in the Colby (Spencer) Series. The soil consists of a windborne silt overlying in many places the glacial till derived from the tranitic rock which it overlies. The area in general is quite level to gently rolling.

Near the southern terminus of this area the soil mantel over the bedrock is rather thin, perhaps four to six feet in thickness, and the area is also close to the contact between the granitic rocks and the lowest member of the Cambrian formations, the Mt. Simon sandstone, which is quite porous and is an aquifer.

Severe conditions exist in this area in many years in the early or middle part of the spring season, one of which is shown in Figure 3, which is a view of a concrete pavement covered in places with gravel, disintegrated granite, cinders, or whatever material of the nature is available to assist traffic in getting over the places where the pavement has failed due to the almost fluid condition of the subgrade soils which can be seen extruding and flowing from along the sides of the pavement.

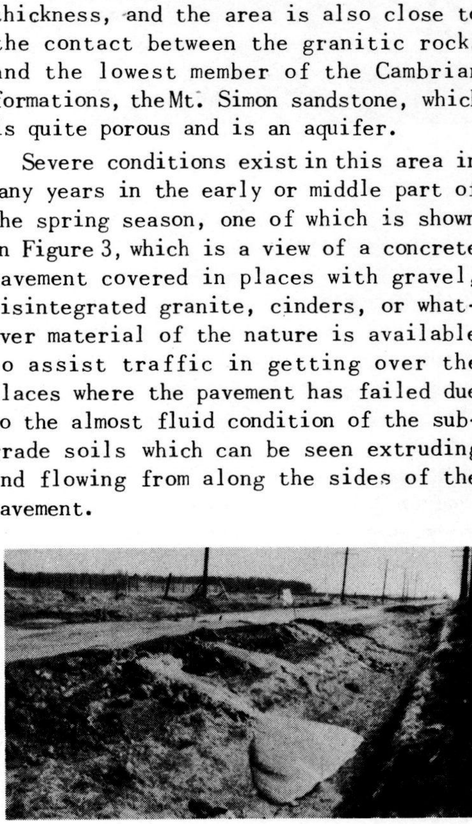

Figure 3.

Somewhat farther to the north in this same area, where the influence of the contact zone may not be as great and where the soil mantle is thicker, but of the same classification, the predominant type of failure consists generally only of almost continuous longitudinal cracking as shown in Figure 4.

In other parts of this area, but in soils of the same classification, where they overlie preglacial ridges in the granitic bedrock, the reaction of these soils is very moderate.

In the region of the calcereous glacial

Figure 4.

drift the influence of its characteristics on the surficial soils is shown in the accompanying photographs.

Figures 5, 6, and 7 show views of pavements laid on soils of comparable characteristics, namely, a loessial soil overlying glacial drift and developed under conditions of prairie cover. The soils are classified as the Carrington Silt Loams (Parr, U.S. Bureau of Soils); however, the underlying drift has characteristics of different natures.

The pavements each have expansion joints placed at 50-foot intervals and longitudinal joints, except the one shown in Figure 7, which also does not have the thickened edge.

The pavement in Figure 5 has developed no transverse cracking except in very occasional slabs and no longitudinal cracking or other distress. In this case it is laid on the thin drift of the Illionian stage in southern Wisconsin, overlying the St. Peter sandstone. The terrain is quite rolling and the relative topographic elevation of the site of the roadway fairly well elevated. The highway carries about 1,200 vehicles per day.

In Figure 6 the pavement has developed about two transverse cracks in each 50-foot slab and some faulting at the cracks and joints, besides some distress at the interior corners and a slight amount of longitudinal cracking, although this is of a very occasional and transient nature.

In this case the underlying drift is of the Wisconsin stage, also in southern Wisconsin, and is derived in part from the Lower Magnesian limestone and in part from the St. Peter sandstone. The topography is somewhat rolling. This highway carries about 2,000 vehicles per day, of which about six to eight percent are heavy trucks engaged in interurban hauling.

In Figure 7 the view rather speaks for itself. The underlying drift is derived largely from the Niagara limestone during the Wisconsin stage of glaciation. The traffic on this road is around 1,500

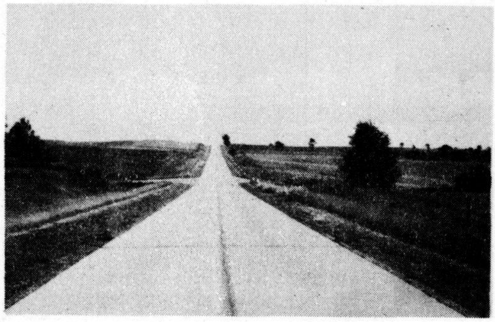

Figure 5.

vehicles per day.

These general surveys were begun in the fall of 1942 and carried on through a large part of 1943. Subsequent to that time further investigations were made at irregular intervals, to check upon some specific matter, or to study not only the presence of failure, but also its incidence, its characteristics, and the manner of its occurrence.

Other contributory factors were given consideration. The commonly held concept of the capillary movement of moisture in soils for a long time, even before this investigation, could not be associated with pavement performance, because pavements laid on soils of so-called high capillary properties never gave any trouble in the summer or fall of the year, even though a manifest water table was well within the purported height of capillary rise for the soil. A study of available literature on soil-moisture movements was undertaken, and from this, and the observation of the position of moisture in the soils during the processes of making excavations, and the observation of other natural phenomena relating to moisture movements, a synthesis has been developed by which moisture movements in detrimental quantities can be more reasonably explained and understood, and which quite apparently is borne out through the satisfactory functioning over a period of time, up to and over ten years, of a substantial mileage of pavement of both the rigid and flexible types in the state, under the severest conditions of exposure to moisture movements in and through the soils, but which were built on freely draining permeable ballast courses.

These studies, and the observations of the differentials in the spring break-up in different years, associated with certain differentials in the meteorological conditions over the past twelve years, indicate another strong long-time influence on pavement performance, not so much as affecting the surface structure directly, but indirectly through the volumes of moisture in the soils and its proximity to the subgrade zone.

From the combination of all of these contributing elements some conclusions have been drawn; some from direct observation and diagnosis of conditions, and others from inference based on the same factors.

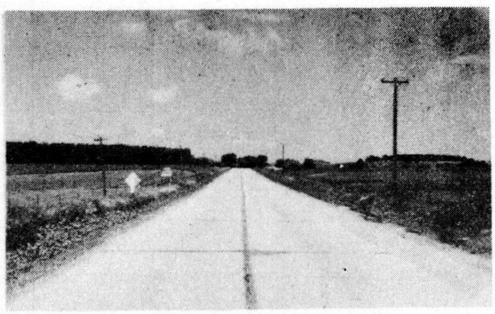

Figure 6.

The sampling area was large, so that the idiosyncracies due to exotic local conditions were more or less recognizable and do not have too great a bearing on the final thesis.

These features and other evidences of pavement failure and the probable causes and mechanics thereof will be further described subsequently in greater detail, but in order to form a further background some of the other influences will be treated with prior thereto.

METEOROLOGICAL INFLUENCES

For ordinary highway engineering purposes the influence of the meteorological conditions is conceived of under the

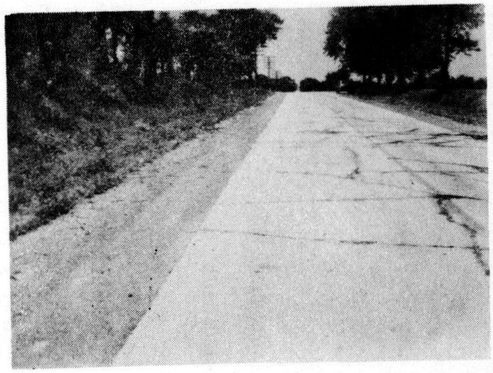

Figure 7.

rather broad aspects of general climate. Thus it is considered that in certain areas humid conditions will prevail, while in others they may be considered as arid or semiarid. Similarly, areas are spoken of as being subject to frost conditions, while others are rarely, if ever, exposed to freezing temperatures, and so on, for the other phenomena associated with climate in general.

While these broad distinctions do prevail, it is conceived that the differences generated by them are those of degree, rather than that they are fundamental. Within these broad climatic zones there are differentials created by the immediate weather or meteorological conditions which contribute substantially to the performance evidenced by pavements in a given year or period of years, because they have a bearing on the amount of moisture contained by the soils, which in turn is a factor in the disposition of further moisture made available to them, or the quantity held by them, to come under the influence of the forces causing movements of it to the subgrade zones in deleterious quantities.

The forces causing these movements are also generated largely by the meteorological conditions, namely, the temperatures.

These conditions in Wisconsin are severe and are conceived to be such that the manifestations thereof become evident without the necessity of taking refined measurements.

The mean annual average precipitation is of the order of thirty inches, ranging over a 56-yr. period from a minimum of 21.41 to 41.64 in. The mean annual average temperature ranges from 39 F. in the northern part to 44 F. in the southern part of the state. Extreme ranges in temperature of from 54 below zero to 110 above have been recorded.

The state itself lies on the divide between two of the great continental watersheds, the Great Lakes-St. Lawrence River and the Mississippi River-Gulf of Mexico drainage basins. The many tributary watersheds to these, which often appear encroach upon each other's territory because of their apparently level nature at their sources, generally assure of a copious amount of moisture in the subsurface reservoirs to provide the moisture which many times comes under the influence of the forces causing its movement.

It has been observed through the years that the so-called annual spring break-up does not occur with equal intensity every year. During the past twelve-year period there were three years in which the break-up was exceedingly severe, so much so that in certain areas of the state it became necessary to close roads surfaced with concrete pavements, not so much for the purpose of saving the pavements, but because they became impassable to traffic.

The subgrade soils became almost fluid, and are often extruded along the side of the pavement, as shown in Figure 3. Trucks often broke through the pavement slabs and became mired in the subgrade soils. Figure 8 shows another view of a similar condition, where sections of the broken slab had been partly pulled out from the roadway, some of the soft soil had been removed, and the entire area covered with surfacing material to keep traffic going.

In other cases, ramps had to be built to the heaved areas so that traffic could get over them. The view in Figure 8 portrays to some extent the height attained by some of these heaves.

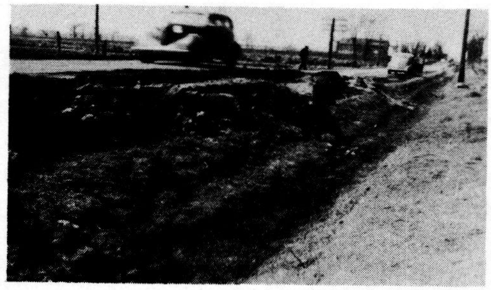

Figure 8.

The extent to which this occurred varied from year to year.

The meteorological conditions during the three periods first mentioned were quite parallel. These occurred in the winter and the following spring seasons of 1935-1936, 1940-1941, and 1942-1943. The snowfall began rather early in the winter and, with successive storms, considerable snow had accumulated. The winters were marked with almost constant subfreezing temperatures, yet by late winter nearly all of the snow had disappeared without any appreciable surface runoff.

The blanket of snow on the shoulders of the road had prevented deep penetration of frost, or acting as an insulator, permitted the thawing of the soil and the infiltration into it of the melt water of the gradually disappearing snow. It is particularly recalled that by Washington's Birthday in 1943 the snow had almost entirely disappeared, without any surface runoff, and upon its disappearance the tulips were found to be sprouting. The dissipation of the snow in this manner provided a copious supply of moisture in the shoulder zones, from whence it could readily be conducted into the subgrade zones under the pavements themselves, where it froze, with the consequential severe damage under traffic upon thawing.

In contrast to this, the break-up in the spring of 1947 was relatively light. Considerable snow had fallen early in the winter, and was on the ground when a January thaw and rain brought about its rapid dissipation, causing floods in a number of places in the state.

Cold weather following this froze the soil to a considerable depth, whereupon subsequent snowfall, even in substantial quantities, coming later in the winter, occasioned no extensive damage, because most of this was dissipated through surface runoff.

The rather obvious correlation of the severity of these conditions to the immediate meteorological influences gave rise to a consideration of the probability of the overall influence of the meteorological conditions to pavement performance.

The extensive and severe drought conditions of the early part of the 1930 decade are still within general recollection.

Lloyd L. Harrold, Assistant Engineer, U. S. Geological Survey, *(2)* reports that in the 17-year period, 1920 to 1936, there were 13 years in 31 humid states, affecting from 5 to 62 percent of the area, and 9 years in 5 semiarid states, affecting from 10 to 100 percent of the area, in which the precipitation was sufficiently deficient to class the same as drought conditions.

It is quite distinctly remembered that through those years frequent comment on the bid prices for contract work was made to the effect that the prices offered by given bidders reflected the experiences of the proceeding dry years, and that some of them would be caught without having sufficiently taken into account the contingency of wet weather conditions, should they prevail in the ensuing season.

The full effect of this was, however, veiled because the drought period ended during the depression of the fore part of the 1930 decade, and the transition into the wet cycle was coincident with economic recovery, which influenced prices, and the ensuing war inflation obliterated the possibility of observing any trends in the price structures due to changes in the meteorological cycles.

Economic cycles have been associated to the meteorological cycles, because of diminished income of the agriculture industry, causing abandonment of farms and migrations of population.

Sidney Makepeace Wood *(3)*, in connection with work on the fluctuations of the Great Lakes and shore line erosion, has shown an association of meteorological conditions to solar radiation and planetary cycles.

The latter part of the 1930 decade and the forepart of the 1940 decade indicate a departure from the dry conditions to a more manifest wet cycle.

While there appears to be some controversy regarding the cyclical recurrence of given weather conditions, there is some agreement with respect to the periodicity of the Bruckner *(4)* cycle. This generally sets forth that warmer and drier periods alternate with periods cooler and wetter in an oscillatory cycle of about 35 years from maximum to maximum.

The graphs on Chart No. 1 developed from the precipitation records of three Wisconsin U. S. Meteorological Stations have been constructed in the manner indicated for the Bruckner cycle. The values shown for the individual years are the ten-year averages of the five years preceding and the five years succeeding the particular year.

Due to the manner in which these values are determined, the significance of the chart lies in the analysis of the trends of the graphs rather than in the individual values.

Lines sloping upward from left to right indicate a general increase in the trend of precipitation, even though there may be individual years within the period when deficiencies have occurred, and conversely a downward slope from left to right represents periods of generally accumulating deficiencies of precipitation.

The Milwaukee graph indicates a general accumulation from 1897 to 1916, with deficiencies generating quite rapidly from 1916 to 1932, after which accumulations take place again. The Madison graph follows this pattern very closely, except that the deficiencies extend somewhat beyond 1932, and with a lower rate of recovery succeeding the low point.

The Eau Claire pattern follows the first two in the earlier years, after which there is a sharp drop indicating a severe droughty period with a subsequent slight

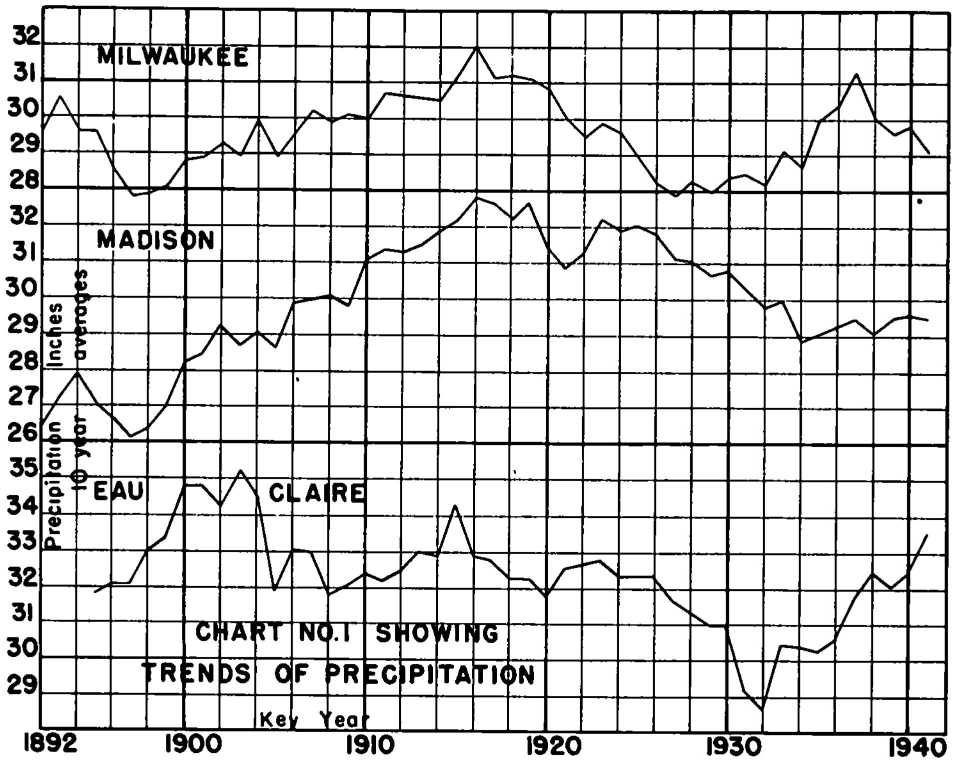

Chart No. 1.

recovery and, except for a small peak, shows a rather uniform period of precipitation at about the mean level for the station until 1926, which is again followed by more deficiencies, with a rapid recovery following 1932.

The graphs for several other stations follow similar trends.

The significance of these cyclical occurrences developed on this basis is that it is indicative of the fluctuations of moisture in the natural storage reservoirs, as manifested both in the open reservoirs, such as lakes, and in the underground storage in the soils.

The consideration of cyclical recurrences of meteorological conditions, even though there may be some controversy regarding the same, is worthy of note, because meteorological conditions control and influence the quantity of moisture in the soil and its subsequent movements.

During drought periods of some extent the underground reservoirs of moisture are depleted, accompanied by a general dehydration of the soil in the surficial zones. It has been observed, in excavations made on highway construction during the height of the drought period in the early part of the 1930 decade, that the soil had been dehydrated to depths of 15 or 20 ft., as evidenced by the blocky nature of the excavated material.

Additional moisture, becoming available to the surficial zones under these circumstances, will rapidly be absorbed and dissipated, through percolation, into the body of the soil, leaving but little in the subgrade zone to cause deleterious reactions in the pavement.

On the other hand, during periods of ample moisture, the soil throughout its body, as well as in the surficial zones, is at its moisture-holding capacity, so further absorption and percolation is retarded or inhibited, with consequential reactions in the pavements.

The period from the middle twenties to the latter thirties was marked by rather extensive investigation and development of fundamental highway or pavement design features. Since this was largely during the periods of deficiencies of moisture in the soils, these developments were influenced by these conditions; and the conclusions drawn thereon therefore, no doubt, reflect these conditions to a greater or lesser degree.

The problems evidenced of late by the pumping and faulting of pavements, and their general decreciation, are generated to a large degree by the generally changed meteorological conditions, with the consequent greater quantities of moisture in the soils rendering them more susceptible to deformation under load, rather than to increase in the number of heavy loads alone. This should be indicative that some fundamental changes in design precepts would be in order.

MOISTURE IN THE SOILS AND ITS MOVEMENTS

In the considerations involved in the performance of pavements, and the factors to be resolved in formulating precepts for their design, there are some that can be determined with reasonable accuracy because their properties are more or less fixed. Illustrative of these, in the element of the soils, are determinations of grain size and distribution, structure, certain soil-moisture relations, strength or bearing values as related to, and under the conditions of, the test, and similar properties.

The quantity of moisture that will be present, and the manner in which it occurs under conditions of actual field exposure, is a variable, not only with respect to location, but also with respect to different periods at the same location.

It has been pointed out that the meteorological conditions have a bearing or influence upon the quantity and position of moisture in the soils. Other studies and observations of position of moisture in the soils, and of pavement performance, indicate that the subsurface conformations are a rather controlling element, associated with the potential of the presence of moisture in the subgrade zones, to cause harmful effects upon the pavements built on them.

While it may be possible to make determinations in the field of the quantities of moisture present in the soils at given locations, such determinations by and large would reflect only the immediate values under the prevailing meteorological conditions. Due to the variability of all of the elements involved, and conditions generated by the meteorological influences and subsurface conformations, it appears doubtful, at the present time, as to whether the potential presence of moisture in detrimental quantities in the subgrade zone can be arrived at more definitely or exactly than through inferential methods based on the synthesis of all of the elements and forces involved, and contributory to the presence and movement of moisture in the soils.

The movement of moisture through the soils can and does take place in all three dimensions, namely, downward, upward, and laterally, the direction of the movement being controlled by the forces activating the movement.

The upward movement of moisture has commonly been associated with a force termed "capillarity," and is described as being quite parallel to the capillary rise of fluids in tubes having bores of small diameter. References to it set forth that rises of moisture to considerable heights, but at slow rates, take place in very fine grained soils such as clays; to lesser heights, but at more rapid rates, through silts and very fine sands of intermediate grain sizes, and quite negligible both as to rate and height in materials falling in the range of coarser grain sizes, such as sand.

Demonstrations of this can be effected by the exposure of the bottom of a dehydrated column of soil to a source of moisture, and perhaps measurable quantities and heights of rise can be ascertained.

However, except for a narrow fringe at the contact between the soil and the source of moisture, the quantities of moisture raised through the soils, under the influence of this presumed force, are insufficient to cause harmful results in the subgrade zones.

Observations of this have been made during the course of excavations, in borings, and in the conditions of road surfaces during the warmer seasons of the year, where these lie upon soils of high capillary properties, and have a manifest water table but a small distance below the surface of the roadway.

On an occasion, borings which were made through a clay soil to an aquifer located about six feet below the surface indicated a rather stiff and firm, though damp, condition to a distance of about six or nine inches above the water-bearing stratum, within which small distance the soils were saturated. The water in this stratum was under head, since, upon penetrating to it, the water rose in the boring to within a short distance of the surface. Borings made a year apart at the same locations yielded almost identical results. The quantity of moisture in this column was at about 22 percent, which is approximately the "Optimum" moisture content for this soil.

Studies made for agricultural purposes of the so-called capillary rise, to determine its effect upon plant life, generally indicate that it is insufficient to prevent the wilting of plants, which is borne out by the more rapid wilting of the short-rooted plants.

Bouyoucous *(5)*, summarizing certain tests on the capillary rise of moisture in soils under field conditions, sets forth that the results yielded by the method used showed that, when the water table did not rise to wet the soil by very close proximity, the actual capillary rise was very slow, very small in total amount of moisture moved, and extremely limited in height of movement.

On the other hand, it is known through observations and experience that detrimental upward movements of moisture do take place, and observations have been made where these have occurred, not only through soils possessing so-called highly capillary properties, but also through noncapillary materials.

Several projects had been constructed by placing layers of porous sand upon the earth grade. These ranged between the projects from 12 to 18 inches in thickness. In order to "stabilize" the upper surface of these lifts, a layer of a silty or silty clay soil several inches in thickness was placed upon them, upon which 6-inch base courses of stabilized crushed stone or crushed gravel aggregate were built, which in turn supported a 1½ to 2-inch dense graded bituminous surface course. It was but a matter of a couple years before the mats had all but completely failed, and the base course material had become contaminated to a large degree with the underlying soil material, indicating its change of state from a firm to a very plastic or partially fluid state.

Other cases have been seen, upon investigation, where the lower portions of stabilized gravel or crushed stone base courses placed directly upon porous lift courses, and protected from surface infiltration by dense bituminous mats, contained free moisture to the extent that water flowed out of them when the sides of the test hole were disturbed, while the underlying sand was only slightly damp. In one case the underlying lift course was crushed limestone, maximum size three inches, with insufficient fine material to fill the large voids between the rock.

These demonstrations have been observed in widely separated portions of the state, but only toward the early middle part of the spring of the year. While, in areas of the latitudes of Wisconsin, these conditions may be loosely associated with frost action, it must be considered that, in order to have frost action, moisture must be present because dry soils, like other dry materials, do not freeze, wherefore frost action is but the supplemental result of a previous accumulation of moisture. These accumulations will take place in the subgrade zone, because the escape of this moisture into the atmosphere is inhibited by the presence of an impervious surface, such as a pavement.

The term "capillarity," or better, "capillary capacity" in the field of Agricultural Science, is used to denote the quantity of moisture that can be retained by a soil, against the force of gravity, under conditions otherwise permitting of free internal drainage. This denotes a state or condition, rather than a force, and the further use of that term in this discussion shall be interpreted in that light.

This can be considered to be a condition of equilibrium between the forces acting on the moisture, which tend to cause or stay its movement, and no further movement will take place unless the equilibrium is disturbed, or other forces act upon it to cause futher movement.

That films of moisture surround soil particles is a quite generally accepted concept. There is also general acceptance with regard to some of the properties of this film moisture. For example, the films of moisture surrounding the soil particle are built up of layers, such as the layers of an onion, but blending rather insensibly with each other. The moisture on the outer layers of the film is quite readily removed, and freezes at -4 C. That which is in immediate contact with the soil particle cannot be removed by a force many times that of the force of gravity, and will not freeze at temperatures above -78 C. It is also known that fine particles of soil will hold relatively thicker films of moisture, and with greater tenacity, than do the coarser particles of soils. It has also been found that, on particles of the same size, the thickness of the film varies with the chemical composition of the soils.

These phenomena indicate that there is some force within or surrounding the soil particle which causes the moisture to adhere to it. Whether this force is caused by electrical energy or by molecular attraction, or is derived from other sources, is not too material or critical in these considerations; however, some of the attendant phenomena are not entirely clarified by the general concept alone.

Edlefsen and Anderson *(6)* adopt the idea of an adsorptive field of force surrounding the soil particle which holds the moisture against the soil particles under pressure, setting forth that such idea is not radical, since practically all explanations of adsorptive phenomena use some such concept. Such field of force acts in a manner analogous to the force of gravity of the earth. In accordance with Newton's inverse-square law, the force of gravity of the earth varies inversely, as the square of the distance from its center. Similarly, the adsorptive field of force surrounding a soil particle is effective in some inverse ratio. While this has not been measured, it has been postulated by various investigators that this ratio varies from an inverse square to an inverse fifth of the distance from the center.

Such variation could perhaps be accounted for by the difference in chemical composition of various soils.

However, following up on this concept, it can readily be seen that the film moisture in immediate contact with a small particle of soil being under extreme pressure cannot be removed by a force many times that of gravity, or frozen at temperatures above -78 C., while, progressing outwardly through the film, the pressure holding it diminishes to the point where it is almost free water, and freezes at temperatures only slightly below the freezing point.

Similarly, the pressure per unit on a film surrounding a large particle of soil is considerably less than on that surrounding a small particle, which accounts at least in part for the lower moisture-holding capacities of the coarser grained materials, and the greater ease with which such moisture can be removed from the coarser soils.

For the sake of simplicity, this force will be designated as the surface energy of the soil particles.

At the condition of soil-moisture equilibrium or capillary capacity, the surface energy of the soil particles is satisfied, so that the particles are surrounded with films of moisture and hold, suspended between them, pendular moisture, held there by the surface tension of the water. Since the span or space between two small particles is less than the span between two larger particles, the force of surface tension is not extended over as great a distance, hence a relatively greater volume of water is held by this force in fine grained soils then there is in soils of greater grain size.

At the condition of equilibrium, therefore, the only force tending to cause further downward movement is the force of gravity, and this is conceived to be insufficient to overcome the combined effect of the surface energy and the forces of surface tension, hence there is a static condition, and to disturb this some other force must be applied if movement is to be initiated.

This may be occasioned by a hydrostatic pressure exerted by additional moisture applied at the surface. In certain soils, such as the silty clays of rather densely graded constitution, or clays of a platy structure, additional moisture may become available to their surface, as through cracks or other fissures in pavements, at a faster rate than at which the hydrostatic head developed can force the moisture through the equilibrium, wherefore the added moisture competes for space in the surficial zones of the subgrade soils, swelling these, so to say, with the consequent formation of the slurry under the action of traffic, with consequent pumping and faulting.

A demonstration of the percolation rate through soils was made by Nelson and Muckenhirn *(7)* in conducting field tests on the rate of intake of water or field percolation rates on four Wisconsin soils, which shows a rate of intake ranging from one to three and one-half inches of water during the first hours, and then drops off very rapidly, so that at the end of about six hours this ranges from about one-fourth inch to one inch. The rate of one inch was sustained in the soil having an initial rate of three and one-half inches, up to about 22 hours, after which it continued at a steady rate of about one-half inch per hour. The soil with the initial rate of one inch per hour dropped to about one-fourth inch at the end of six hours, and after ten hours continued at a rather steady rate of four hundredths of an inch per hour. The two other soils reacted in a parallel manner in a range between these.

On the other hand, should the soil become dehydrated due to a general depletion of the moisture in the underground storage reservoirs and other causes, as occasioned by protracted dry meteorological periods or deficiencies of precipitation, often accompanied by fissuring in the soils, there will be an absence of pendular moisture, and perhaps of film moisture, accompanied by unsatisfied surface energy of soil particles extending to greater or lesser depths, dependent upon conditions. Additional moisture becoming available is rapidly absorbed and dispersed vertically by the combined effect of the force of gravity and unsatisfied surface energy of underlying particles, and to some extent laterally by the latter forces, which continues as additional moisture becomes available, until the condition of equilibrium has been re-established.

Under the dehydrated conditions and reactions, insufficient moisture will be retained in the subgrade zone, so as to eventually result in pumping, displacement of soil to cause faulting, or to allow a deformation of the soil, with consequent distress in the pavement.

There is, however, another set of forces, extraneous to the soils themselves, which have a major influence on the movements of moisture in the soils. They will assist in downward percolation, cause rises of moisture from subsurface sources in detrimental quantities, and under certain conditions cause lateral movements in, or in the proximity of, the subgrade zones. These movements are induced by and follow the direction of the flow of conduction currents of heat in the soils.

These forces or movements perhaps remained unrecognized because of a concept that movements generated by these forces were solely associated with frost action.

However, the application of the Studies of Stephen Taber *(8)*, in the matter of moisture movements in the soils under conditions conducive to freezing, is indicative that these forces and movements are also active to a greater or lesser degree, dependent upon temperature differentials, even though the process is not carried through to the ultimate frozen condition.

When there is a differential in temperature between two substances there will be a flow of heat from the warmer to the cooler medium, never the reverse. When, therefore, the atmospheric temperature is lower than that of the earth, heat is radiated away from its surface. This loss of heat at the surface will cause conduction currents of heat to rise through the soils in the effort to maintain a condition of temperature equilibrium throughout its mass. According to Taber, these currents carry with them moisture in the form of fine films or filaments under high tension, but that this movement is not due to capillarity, because there is no free surface or meniscus.

That this movement is not due to certain rather prevalent concepts of capillarity is borne out by previously cited observations, made where moisture rises took place from underlying sources through a rather heavy layer (18 in.) of coarse sand, into a silty clay layer, about four to six inches in thickness, placed upon this sand layer, to provide what was then thought to be a stable layer upon which to construct a gravel base course.

Other evidence of this can be observed in the freezing taking place in sands which apparently were quite dry before freezing, and which yield a very wet condition upon thawing.

Whether this moisture rises in the form of films or filaments, or as vapor, or as molecular moisture, is not too critical. The concept of this movement as being one of moisture in the molecular state yields perhaps the most satisfactory explanation of many of the phenomena associated with it.

The velocities of the molecules making up the mass of a substance are not constant, nor are they uniform. The large proportion of them vibrate at what can be termed an average rate. On either side of this there are proportions whose rate is very slow, and others whose motion is very rapid. The rates of movement are also dependent upon conditions of temperature and pressure.

In a mass of water or moisture, the rapidly moving molecules are more sensitive to a change in conditions than the slower moving ones, and will leave the parent body in a vapor or molecular phase upon a change in conditions. As the differential conditions become more pronounced, the greater will be the number of molecules affected thereby.

Visible manifestations of this movement may be seen in the ground fog occurring in scattered areas during a cool evening following a warm day, or in the relatively thin layer of recondensed vapor overlying the surfaces of open water in the wintertime when there has been a sharp drop in temperature, and similarly by the drying of wet pavements accompanying a rapid, sharp drop in temperature in the wintertime; while with a slight or gradual drop an icy condition on the surface will occur.

This movement is further demonstrated in the mechanical refrigeration units by the dehydration taking place in certain articles of food stored in them. Leafy vegetables, such as lettuce, celery, cabbage, etc., will wilt when stored in these without being placed in a receptacle containing water; meat, cheese, etc., especially when sliced, will dry out and become unpalatable; and during the process, accumulations of ice continue to grow on the cooling element of the refrigerator, even though there is no physical connection between it and the food substances.

When this movement takes place in soils underlying a pavement, the moisture cannot escape because of the imperviousness of the pavement, and thus will continue to accumulate in the immediate subgrade soils. The volume that will accumulate in this manner is dependent upon the quantity of moisture available, the constitution or stratification of the soil, or both, and the degree and duration of the temperature differential.

Accumulations of moisture whose movement was induced by temperature differentials have been observed at the interface between the top of base courses and the bottom of bituminous surfaces. Holes dug through dense bituminous surfaces to the dense graded gravel, or crushed stone base underlying them, in the fall of the year gave evidence of the existence of an almost saturated thin layer of the aggregates of the top of the base course. The observations made in examining the mat itself showed no indications that this moisture had penetrated through the surface from surficial sources, because the material in the mat was dense, hard, and dry.

The moisture having access to the subgrade zones and which, as indicated, may even penetrate through dense graded base courses, may be from several sources.

A soil at or near its capillary capacity will have loosely held moisture in the outside rims of the films surrounding the soil particles, besides containing a certain amount of free pendular moisture. Being at or near the surface of the subgrade, it may migrate upward to affect the subgrade quite readily, even with temperature differentials of lesser degree.

Free water, either as ground water or in water bearing strata within the limits of the influence of the forces of heat conduction, will provide prolific sources of moisture.

Accumulations of snow piled upon the shoulders, and which are dissipated without appreciable runoff, as previously referred to, form an insulating blanket on the shoulders, so that any frost in the shoulders will be thawed out from below, opening the soil so that the melt water from the snow can penetrate into it, and since there is a great temperature differential between the pavement, on account of its being kept free of snow, and the shoulder zone, which causes a flow of heat from the shoulder to the pavement, conducting with it moisture from the melt water. In the densely graded soils, such as silty clays and clays, the movement laterally is not conceived to be very rapid, so it freezes before reaching the center of the pavement. Upon thawing there will be a considerable moisture differential across the width of half the pavement, causing considerable variation in the bearing value of the soil, with consequent longitudinal cracking under load.

In the porous silts, however, this movement is more rapid, and whem coming from both sides of the pavement will cause concentrations along the center, causing center breaks in rigid pavements, and the prevalent turtleback cracking along the center of bituminous pavements, even though the base underlying these is considerably thicker in the center than along the edges. (Old type feather edge base construction). This type of failure is quite common in the Driftless Area, while in the glaciated regions the failures of bituminous surfaces more often start at the edges and progress toward the center.

Many failures in pavements have been ascribed to a cause commonly referred to as frost action. This can be considered to be somewhat of a vague description covering a multiplicity of conditions. Freezing can be conceived to be a negative result, consequent to the loss of heat, rather than as a positive force.

In order to have freezing in the soils, moisture must be present, because dry soils cannot freeze. Moisture will accumulate in the soils, however, due to the forces previously described, and when sufficient loss of heat has been sustained, this moisture will change from the liquid to the solid state. In some soils this will create no readily manifested disturbance, because most of the expansion involved in the change from the liquid water to the solid ice is taken up in the interstitial pore space. In other soils the freezing will generate segregated ice crystals or lenses, as described by Taber *(8)*. These occupy space and create pressure, resulting in a raise of the surface of the pavement. Such raise may be a matter of several inches and be quite uniform, with a gradual transition between areas of varying lift, so that it is not perceptible except at bridge abutments, railroad crossings, or other features which are not similarly affected, or it may affect only areas of limited dimensions, but with appreciable rise resulting in bumps of greater or lesser severity in the pavements. Such areas of differential heave may be due to either a change in the constitution of the soil, or a difference in the moisture available to soils of the same characteristics. Other than the riding discomfort, and at times the actually dangerous condition created by the differential heaves, and the consequent pavement cracking or breakage due to the sharpness of such raises, the lift occasioned by such freezing of itself does no particular damage as long as the ground is frozen, because of the almost unlimited bearing inherent to the frozen soil.

The thawing of the accumulated frozen water in the soils is the element which creates the conditions highly detrimental to the pavements and roadway surfaces. The large volumes of water created by the melting ice are impounded between the pavement and the underlying, still frozen, zone.

This moisture cannot escape vertically or laterally when it is thus impounded in most soils, and dependent upon the quantity, will cause failure ranging from slight cracking to complete destruction of sections of pavements, because of the almost fluid consistency of the soil generated by this water.

Figure 9 shows a view where a spot of this condition has broken through to the surface. When these occur in isolated spots they are often referred to as frost boils; however, in many cases it is not alone that isolated spots are thus affected.

Such conditions in Wisconsin generally prevail for a number of weeks, until the underlying frozen strata have thawed completely, permitting of downward percolation of the excess water, assisted by a reversal in the flow of temperatures usually beginning at those times.

In the foregoing it has been endeavored to develop and outline some of the principles and phenomena of moisture movements in the soils, and the influence of the meteorological conditions, both upon the quantity of moisture present in its sub-

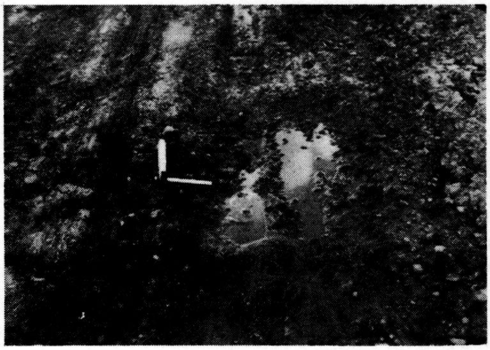

Figure 9.

sequent movements. The application of these principles to the study of pavement performance, together with the consideration of the characteristics of the soils and of the effect of the energy of moving loads imparted to the soils through the pavements, yields what appears to be a rational and logical basis upon which a diagnosis of the performance of pavements can be built, and which has been found to be applicable from place to place, so that it should yield a background upon which future design premises can be established.

In the following it will be endeavored to associate pavement performance with these features, in accordance with the observations and studies made thereof, in conjunction with the correlation of soils to their geological background and environment.

PAVEMENT PERFORMANCE

The pavements built in the State of Wisconsin in 1920 and earlier were of the thickened center type, usually 6 or 7 in. thick at the edges, and 8 in. thick in the center, without a longitudinal center joint, and had 1/4 in. expansion joints at intervals of 50 ft. During 1921 and 1922 the same general type of pavement was built, except that the expansion joints were omitted; the omission of which was continued through 1924. However, the year of 1923 saw the introduction of the longitudinal center joint, formed by means of full depth deformed metal plates with ½ in. square steel tie bars spaced four feet on centers. In the subsequent discussion, whenever longitudinal cracking is referred to, it shall always connote such cracking in pavements having the longitudinal center joint. This year also saw the beginning of the use of the thickened edge section, which has been used continually since then up to, but not including 1946.

In about 1925 the first venture with dowels across the transverse expansion joints was made. The concept of load transfer was unknown, and four ½ in. square rods about 4 ft. long were placed across each joint to aid in preventing one slab from rising over its neighbor, by sliding on the joint material, which had been observed in a number of cases. This design was carried through 1925. In 1926 and 1927 a comparable design was used, except that expansion joints were eliminated on pavement placed when the atmospheric temperature was generally above 60 F.

A large mileage of pavements was built with the foregoing types of design, these years being among the most active in the history of pavement construction in the state.

In 1928 expansion joints were again installed, ½ in. at approximately 50-ft. intervals. This year also saw the inception of the use of dowels across the transverse joints on the principles expressed as load transfer. These were usually 3/4 in. round steel rods, painted and greased to break bond, with a socket on one end to permit of movement when the pavement expanded. These dowels were placed at 15-in. intervals, which was subsequently reduced to 12 in.

During this and the ensuing years up to 1946 expansion joints were used. There was a heterogeneous variety of spacing of these, with and without transverse contraction joints. These ranged from ½ in. joints every 30 to 50 ft. to 3/4-in. and 1-in. joints at spacings ranging from 90 to 120 ft.

When these spacings were over 30 feet, the slabs were divided by means of contraction jo s to yield slab lengths not in excess of 25 or 30 ft. The expansion joint materials used were of the felt asphalt premolded type, cork, rubber, and asphalt impregnated fiber. On a few jobs redwood boards were used, and there were also a few experimental installations of the all metal air cell, water seal type. Contraction joints usually were of the impressed type, the groove thus formed being filled with asphalt joint filling material, heated and poured into the space.

The slab cross section was of the thickened edge type, 9 in. at the edges and 6½ in. at the center, except for work on the heavier travelled roads, where these dimensions were 10 and 8 in. respectively. The transition from the thickened edge through the several years ranged between 2 and 4 ft.

The aggregates consisted largely of gravels and sand, with a few jobs using coarse aggregate produced from ledge rock, usually limestone.

In the northern part of the state the coarse aggregates were largely gravels formed out of the crystalline or igneous rock, such as granites, rhyolites, porphyry, basalt, and similar substances, the fine aggregate being predominantly quartz mixed with some of the igneous material.

In the southern part of the state the coarse aggregates were gravels formed out of the Niagara, Galena-Trenton or Lower Magnesian limestone formations, with sands having large percentages of quartz, supplemented by the dolomitic material, while, in areas in the central part of the state, there were mixtures of these two basic types.

These aggregates generally are free from foreign or deleterious substances, yielding concrete having flexural strengths of better than 750 psi., and compressive strengths well over 3,000 psi. in 28 days, for the ordinary paving mixtures, with 1.25 barrels of cement per cu. yd., which was the cement factor used for quite a number of years, but which was later raised to 1.35 for more or less administrative reasons.

The service record of all of the pavements shows that there is substantially no failure or disintegration attributable solely to the factor of the aggregates or the cements that were used. There is actually only a portion of one job, a few miles in length, where the disintegration of the concrete could be attributed to characteristics of the aggregate, and this, as closely as can be ascertained, was due to a contamination of the aggregate with residual material from the iron formation, and which could not be removed by washing.

Innumerable cores were taken from these pavements through the years, and tested at ages of from six to nine months, consistently showed compressive strengths in the range of from 4,000 to 6,000 psi.

The foregoing briefly and generally outlines the principle features of pavement design and its constituent materials. It is not considered necessary to go into greater detail in these, because these elements, with the exception of the expansion joints, have no great bearing on the performance and condition of the pavements.

It will be essayed in the following to outline some of the common failures in pavements, together with the probable causes of failure, based on observations of their incidence and other manifestations of their characteristics.

Transverse Cracking - This is no doubt the most common type of failure. Transverse cracking is a function of slab length, thickness, load, and soil conditions. Temperature stresses, or stresses induced by expansion or contraction of the slab have but an insignificant effect upon the occurrence of transverse cracks. This is evidenced by the occurrence of cracks in pavements built without transverse joints, located in areas where the soils upon which they lie are not adversely affected by moisture, and in some instances where the loads are not severe even though the soils are adversely affected by moisture.

Observations of crack intervals ranging from 50 to 60 ft. up to as high as 180 ft. have been made on pavements now about 20 yr. of age built without transverse joints.

Some of this cracking had occurred, no doubt, and such observations had been made, shortly after the pavement had been placed, due to initial shrinkage in the concrete while set was taking place. In some cases this occurred at about 50-ft. intervals, in others at around 75 ft., and up to the distances cited before.

Other observations on pavements in service almost 20 yr. where transverse joints had been placed at intervals of from 50 to 75 ft., show that many of such slabs suffered no intermediate cracking.

On the other hand, there are innumerable places, even on substantially reinforced pavements (58-lb. mesh - No. 4 wires at 6-in. centers in both directions) where intermediate cracking has taken place in most of the slabs, on pavements with transverse joint spacing in the range of from 25 to 50 ft. On pavements with 50-ft. spacings substantial numbers of slabs have at least two transverse cracks.

On four-lane, divided highways the right-hand lanes, which carry the greatest volume of the slower-moving heavy vehicles, suffer far more from transverse cracking than do the left-hand lanes, which are used more generally only for overtaking passing maneuvers. This has been observed on a pavement of this nature, 10 in. thick at the edges, with an 8-in. center thickness, reinforced as above described, with joint spacings of 30 ft., where in less than 10 yr. time about 80 percent of the slabs in the right hand half had transverse cracks, with but relatively few of these cracks extending through to the contiguous half of the slab on the left hand side. Quite parallel observations were made on single pavements, of otherwise comparable design and age, on which the heavy traffic was largely unidirectional.

Other observations which indicate that transverse cracking is largely a soil-load function are the intermittent sections on the same road where cracking and no cracking occurs, and many times these sections conform very closely to changes in soil types, as shown on the soil maps, or other underlying conditions.

The soil conditions in the subgrade underlying the slab are not uniform or constant throughout the year, or in successive years. Under some of these conditions the soil will permit of deflection in the slab under load, of sufficient magnitude to permit of rupture. It has been observed that many times this cracking is preceded by a permanent rhythmic waviness in the pavement before the cracks even show up, indicating that some deformation has taken place.

Careful examination of many slabs disclosed cracking on the surface, in what could be termed the incipient stage. A fine hairline crack, barely perceptible, will start either at the edge of the pavement and extend part ways to the center, or it may start at the center on one side of the road and extend part way to the edge. Eventually this will extend across the half or full width of the pavement.

The examination of cracks which appear to have opened up or become wide, in a great many instances, is merely the surficial spalling at the crack edges due to their impinging upon each other during deflection under load.

In other cases, especially on pavements with expansion joints, the transverse cracks actually open up. Similarly, transverse contraction joints in pavements with expansion joints will open up. This is due to the fact that when expansion takes place the entire section between expansion joints acts as a unit, and will extrude the expansion joint material; even wood extrudes to a degree. When contraction takes place, each slab, formed by a joint or crack, acts as an individual entity, causing the crack or joint to open up, providing a space for the prolific infiltration of moisture and detrital matter.

Substantially all of the expansion joints examined in the survey had closed to the degree so that the opening remaining even in subzero weather, was less than the thickness of a 10-cent piece, where it had previously been from ½ to 1 in. in width.

Another type of transverse cracking is due to a shrinkage in the soils upon which the pavements lie. Observations of this have been made both on rigid and flexible type pavements in the winter time, when the soils are frozen. Cracks or fissures appear in the shoulders and shoulder slopes on both sides of the roadway, and are coincident with cracks or fissures in the pavement. These have been observed to occur at intervals spaced as closely as about fifty feet.

The pavement or base course, or both, frozen solidly to the subbase or soil, form an integral mass. Longitudinal fissures, between the edge of the pavement and the shoulders have been observed to be at times concurrent with the transverse fissuring.

Under certain exposures, when cold temperatures set in, there will be a dehydration of the soil, due to the escape of moisture whose movement was induced by the forces of heat conduction, and, if this moisture is not replaced from subsurface sources or by precipitation, a shrinkage of the soil will take place, causing a rupture of the soil at the critical locations.

Longitudinal Cracking -Under longitudinal cracking, as shown in Figure 4, it is to be understood that this discussion concerns itself only with cracking of this type that occurs on pavements with a longitudinal center joint, and the term slab is considered to be one half the width of a two-lane pavement, or one traffic lane.

This type of cracking is a function related to the soil-moisture-load relationships, rather than to temperature or restrained warping stresses within the slab itself, as has been set forth on occasion. This is manifested by the incidence of its occurrence, both by its position within the slab itself, by the soils upon which it occurs, and by certain observations made. Within the slab its position may vary from a line along the end of the tie bars extending between contiguous slabs to a location within about 2½ to 3 ft. from the edge.

With respect to soil types, it is very common on soils of the Colby (also known as Spencer) series, which are located generally in the region of the Older (Illinoian) Drift, consisting of a loessial covering over drift formed of the granitic rocks, and also on soils of the calcereous glacial drift. On the latter it is quite common when the drift had its origin from the Niagara limestone or Maquoketa shales, and appears to a much lesser degree when the drift is a mixture of the lower Magnesian limestone and St. Peter sandstones. These soils are classed largely as the Miami and Carrington (Parr) series. It is very common also on the Red Clay glacial drift, which is a reworked lacustrine deposit. Longitudinal cracking is not quite as predominant on the counterpart of the Miami soils in the region of the crystalline rocks, namely the Kennan series.

It is seldom found on the soils of the Driftless Area, or on many of the glacial outwash deposits.

Its presence in urban areas where full width construction, consisting of traffic and parking lanes with curb and gutter, is quite negligible, even on many of the soils where this is predominantly present under rural exposures.

The cause of this cracking is a differential moisture content in the soils between the edge and the center of the pavement, developing a differential in bearing power. The soils in which it occurs have greater resistance to moisture movement, because of either their finer grain size, structure, or gradation, and are largely silts, silty clays, and clays. The soils in which it is absent have a greater porosity due to larger or more uniform grain size, even though some of them are predominantly silts, so that a uniformity of bearing is maintained, or when they possess sufficient internal friction, such as sands, so that differentials in moisture are not significant.

Cracking of this kind is induced by a differential in the moisture content in the soils underlying the slab, and may occur accompanying the freezing of these soils, as a supplemental consequence of freezing, or at times when freezing is not a contributing element.

It has been observed, during periods when the ground is frozen, that the edges of the pavement are raised so that in cases they are higher than the elevation of the centerline, indicative of a higher moisture content, with consequent greater expansion or lift of the soil along the edges than at the center. When this moisture content diminishes at a rather uniform rate from the edge to the center, the lift or rise of the pavement is gradual, and while the edges may be substantially higher no breakage in the slab takes place. On the other hand, when there is a rather abrupt change throughout the width of the slab, the edges will rise, as will a portion of the slab, and the balance will not conform to the rise, with consequent longitudinal breakage along the line of change of moisture content.

While it is not always the case, the source of the moisture is often from the slow dissipation of snow on the shoulders of the road, melting without appreciable surface runoff. This moisture has access to the subgrade, where it freezes, the penetration laterally being contingent on the constitution of the soils.

Upon thawing, the areas having the most moisture will sustain the greatest losses in bearing, with consequent breakage under load. The first observation of the cause of this breakage was in 1936 on a pavement laid upon the red clay glacial drift about five years previous to that time, and which had suffered no distress of this kind until then. The winter of 1935-1936 was one of severe snowfall, and the location of the cracks throughout this project could quite readily be associated with places where excessive drifting of snow had very likely occurred.

In urban exposures, snow is piled on impervious surfaces, or even on the pervious grass plots between the curb and the walk. Even though some percolation of melt water will take place, which will be induced under the pavement, its penetration will rarely exceed the width of the parking lanes, which are not subjected to the repetition of heavy loads as are the travelled lanes, wherefore even if some loss in bearing is sustained the consequent effects are not too serious.

Mesh reinforcement of the type described arrests the appearance of longitudinal cracks to some degree, or at least until the members become over-stressed and fail. Since the Wisconsin practice has been to use mesh having equivalent members in both directions, and was placed about two inches below the surface of the pavement, the factors of area of steel, location and arrest of cracking, are indicative of the cantilever beam type of failure, resulting in longitudinal cracking.

Occasionally cracks of this type have been observed to start at a transverse crack or joint and extend only part ways along the slab. In these cases, such incipient cracks are more visible than similar beginnings of transverse cracks.

In a great many cases after these cracks have formed there is also a further tilting of the outside part of the slab, to a greater or lesser degree, causing a fault along the crack; that side of the crack on the outside part of the slab rising above the portion of the slab toward the center of the pavement.

Longitudinal cracking in many of the soils is a forerunner of the further almost complete destruction of the slab, caused by the subsequent increased amount of transverse cracking in the outer portion of the slab, probably because of its diminished width.

Longitudinal cracking has rarely been observed on old 9 or 10-ft. pavements subsequently widened by a strip on either side, although in a few cases the widening strips have suffered severe transverse breakage. The full freedom of the widening strips permits them to conform to the bearing of the soil, the hinging taking place along the joint rather than that it becomes necessary for the pavement to produce a joint for itself by cracking of this type.

Pumping and Faulting - These two types of pavement failure can be considered together, since they are often concurrent or complimentary to each other. In either event they have a common cause, namely, the combination of moisture in the soil and the progressive deflection of the slab under moving loads, and whether the ultimate result is pumping or faulting depends to a large degree upon the quantity of moisture in the soil.

Pumping is conceived to be the expulsion of a fluid slurry of soil and water through a crack or joint under the influence of a moving load. While pumping predominantly takes place on the densely structured or graded soils, such as silty clays and clays, it has been observed to some extent on some sands, but rarely, if ever, in the silts of the Driftless Area, where the grain size of the soils is predominantly in the coarse silt fraction.

Faulting can be described as the condition observed where one face of a joint or crack is higher than the other. It is almost universally the case that the side of the crack or joint on the near side toward approaching traffic is higher or above the far side. Wherefore, on opposite sides of the road, the high and low sides appear reversed. Faulting occurs irregardless of whether the joint contains load transfer devices or dowels or not. In fact, instances have been observed where all of the dowels through the joint (3/4-in. round steel rods spaces 12 inches on centers) had broken at the joint.

Faulted cracks and joints are visibly quite obvious, as well as evident by the thump occurring when riding over them. The fact that the near side of the fault is high can be observed when riding over the road, where it appears that the left hand side of the road is cracked to a greater degree than the right side, because the view is against the high side of the fault, while on the right hand side it is over the high side of the fault.

This is portrayed in the view shown by Figure 10. Figure 11 shows a close-up of a faulted joint.

Pumping and faulting, as before stated, are the consequence of a soil-moisture-load combination. It has been observed, in following heavily loaded trucks, during a rainstorm that free water pumps or squirts quite freely out of the cracks and joints as the wheels of the vehicle pass over them. This, however, does no serious damage, since no soil is carried with the expelled water, and on some roads where this was observed no supplementary pumping of soil and water was evidenced later on, nor were any consequential effects in evidence, which would indicate that no movement of soil took place. The reason for this is very likely that after the actual rainfall ceased the remaining water in the subgrade was exhausted from the subgrade zone, through downward percolation, leaving an insufficient quantity to form a slurry, or even to render the soil sufficiently plastic to be displaced under load.

On the other hand, on soils where downward percolation is inhibited due to the texture of the soil, or because its natural

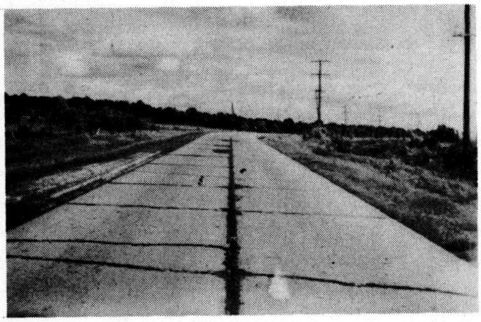

Figure 10.

moisture-holding capacity is satisfied, the excess moisture remaining under the slab is not drained off, and may penetrate only the very surficial portion of the subgrade zone, which, under agitation and vibration induced by traffic, will create a thin layer of a slurry of soil and water, quite fluid in consistency.

In the colder climates such slurry may also be the result of accretions of moisture from subsurface sources in the subgrade at its interface with the pavement, where it freezes. Upon slight thawing an excess of water will be in these soils, creating the same condition.

As heavy traffic passes over the pavement the pavement is deflected, which deflection is carried forward under the load, forcing ahead of it portions of this slurry under considerable pressure. When a crack or joint is reached, this slurry is ejected forcibly upward and out of the fissure, usually before the wheel passes over the opening, with a secondary surge

Figure 11.

into the opening after the wheel has passed over it, part of the slurry from the forward slab having been passed into the void formed by the ejection of the slurry from the back slab. This is in part due to the partial vacuum, which perhaps forms upon ejection and recovery of the slab, and in part from the pressure on the slurry under the forward slab.

Figure 12.

The views on Figures 12 and 13 were taken of a pavement where severe pumping was occurring. This pavement was built in 1937, with joints at thirty-foot centers, and was laid on the red clay glacial till soil. A trench was carefully excavated in the shoulder, taking care not to remove the soil immediately contiguous to the slab until the major portion of the trench was completed, upon which the soil next to the slab was carefully shaved off. A thin layer of slurry was found on top of the subgrade soil, below which the soil for another quarter inch or so was somewhat plastic. Under this the soil was firm. It can be seen in Figure 12 that the forward slab is apparently unsupported for a distance of 4 feet or

Figure 13.

more, and the back slab for about one half that distance. Upon the passage of the first heavy vehicle, the slurry and soft soil extruded sideways, with the results shown in Figure 13.

There will be times during these soil-moisture conbinations when there is insufficient moisture to actually cause a slurry, because the excess may have been pumped out, or the soil may have been able to absorb it to a degree, but there will be enough moisture in it to cause it to become quite plastic, in the sense that it will deform or move quite readily under load. Under these conditions, slight increments of the soil will be moved forward under the progressive deflection caused by a moving load, until a crack or joint is reached, where such forward movement is arrested, and the soil will either move up into the fissure, or build up under the end of the slab, gradually raising it by a wedge-like action, creating the rise on the end of the near slab, thereby generating the fault. Joints of felt asphalt construction sandwiched between thin sheet metal plates have been removed, and upon removal it was found that the felt had been extruded and lost, while the remaining space was partially filled with soil. When it is considered that the average pavement slab weighs only about 3/4 psi. of surficial area, it is readily conceivable that not too much force is necessary to raise its end.

A thickened edge pavement of 10-8-10 in. reinforced with 58-lb. mesh, and having joints at intervals of 30 ft., was built in 1936, and contiguous sections about a year or two earlier. This pavement, upon completion, was considered to be the smoothest built that year in the state, according to roughometer measurements. After about four years it developed extreme waviness, accompanied by transverse cracking. Subsequently this was followed by pumping and faulting. Being one pavement of a dual divided highway, the extreme distress was only on one lane, namely, the one used by the slower moving heavy vehicles. Faulting of as much as 3/4 to 1 in. is common, and permanent deflections at the centers of the slabs of as much as 0.15 to 0.2 ft. have been measured.

The pavement currently, being only about 10 yr. old, is in serious need of heavy resurfacing if its remaining value is to be salvaged.

Upon removal of a section of this pavement in both directions from an expansion joint to the cracks which had formed, it was found that all of the dowel bars, placed at 12-in. centers, were broken at the joint. The concrete under the dowel bars in the slab in the near side toward approaching traffic had been broken down into small pieces, and had become mixed up with the subgrade soil extruded upward into and between the broken pieces, to a greater degree than the breakdown of the lower part of the slab shown in Figure 14, which is a view of the face of the crack, again on the side of that slab nearest approaching traffic.

Figure 15 and 16 show the breakdown along the center joint under the left hand side of the pavement, as well as the disintegration in the triangular area at the intersection of the center joint and the transverse joint. These spaces were partially filled with soil.

Figures 17 and 18 show the condition along the center joint, on the left hand side of the pavement, but under the far slab. While there apparently is a lack of support, the pavement is not broken down.

As this area was being removed a thin layer of slurry of soil and water immediately underlaid the pavement, under which the soil was very firm, sufficiently so that when the slurry was removed the imprints of the feet of the workmen were not discernible.

All of the tie bars across the center joint at this location were also broken.

Taking the case of the breakdown of the

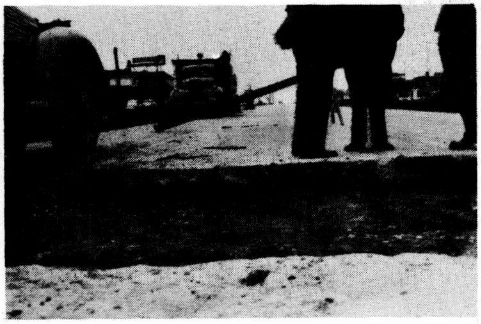

Figure 14.

lower part of the pavement under the dowels at the joint and at the crack, as shown in Figure 14, and along the center joint, as shown in Figures 15 and 16, when loads pass over the pavement there is a deflection downward under the load, with an accompanying rise in the pavement forward of the load, which, together with the build-up caused by the accretion of soil causing a fault, generates extreme shearing or vertical tensile stresses in the lower part of the pavement, generated by the resistance to movement offered by the dowels, tie bars, or aggregate interlock, with the consequent rupture and breakdown.

After the load passes over the crack or joint, the immediate deflection is downward; and, since there is no build-up of soil under the pavement, the pavement is

Figure 15.

not overstressed and remains intact.

These extreme conditions prevail on

Figure 16.

about 15 mi. of this highway where it passes through soils classified in the Carrington series, which consist of a loessial covering over the calcareous glacial drift, and whose development was under conditions of prairie cover, indicative in this state of a higher moisture exposure, than if the development had been under conditions of forest cover. The terrain is quite level, and the loessial

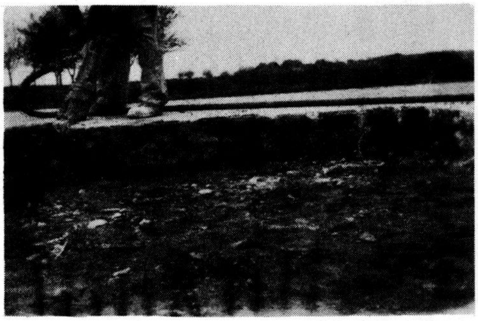

Figure 17.

cover rather deep, so that, in both cut and fill sections, the pavement rests primarily on these loessial soils.

Where this pavement passes onto the Miami soils, the counterpart of the Carringtons, but developed under forest cover, and into the more rolling terrain to the north, the condition is not nearly as serious, even though that pavement is now approaching 20 yr. of service, and the traffic volumes are fully 50 percent greater.

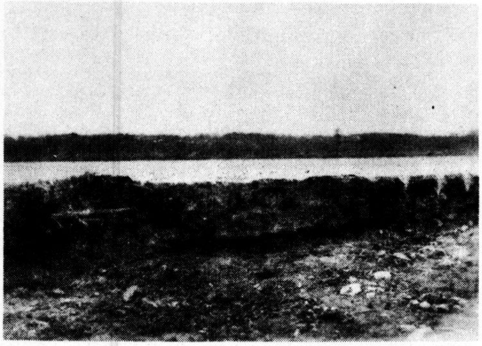

Figure 18.

This is quite definitely indicative that loads are not the sole elements contributing to pavement failure.

"D" Cracking - This type of failure begins with a slight fissuring at a corner, between the edge or center joint of a pavement and a crack or joint, and gradually develops along the crack or joint, and moves progressively away therefrom in either direction, as well as along the edge or center joint to form a pattern roughly in the shape of the letter "D". At the corners this fissuring is closely spaced, and generally on a 45 degree angle across the corner, while along the crack or joint it parallels the same, with the fissures also closely spaced. In both cases there is additional fissuring normal to the main lines.

In the initial stages these fissures appear to be filled with a substance obtained either from the solution of some of the constituents of the cement or of the aggregates. The view shown in Figure 19 is one of the succeeding stages where this substance has partially disappeared, and open fissuring has begun. Figure 20 depicts a very advanced stage where, through the action of moisture, freezing or traffic, or a combination of these, disintegration had set in, accompanied by a loosening and loss of aggregate. This had progressively worked into the slab and away from the crack or joint, leaving the pavement remaining intact in the shape of a wedge, the thinnest part being at the crack or joint.

Figure 21 is another view of an advanced stage of this type of failure, which has also attacked the lip curb along the edge of the pavement.

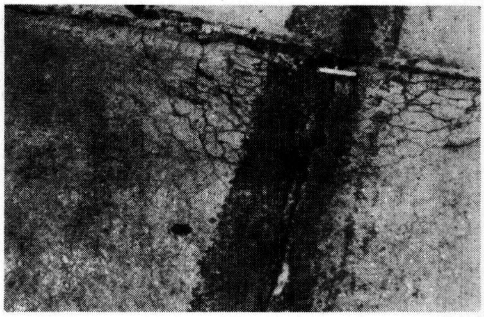

Figure 19.

Being of a progressive nature, this is a serious type of failure in pavements.

The causes of this type of failure have not as yet been fully evaluated, but through the association of the locations of its occurrence it can be set forth that it is a condition related to soil-moisture combinations and loads rather than the one associated with the materials used in the construction of the pavement. Whether or not Air Entraining Cement will minimize this type of failure cannot be forecast at this time, since this type of failure has not been observed in pavements less than ten or twelve years of age.

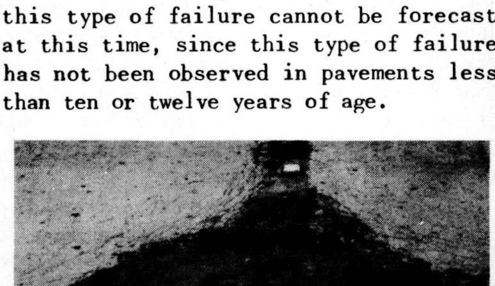

Figure 20.

At one time it was thought that this type of failure was inherent to pavements constructed with dolomitic aggregates. However, it was found that the characteristics of the foundation soils are the influences responsible for the failure rather than the type of aggregates used

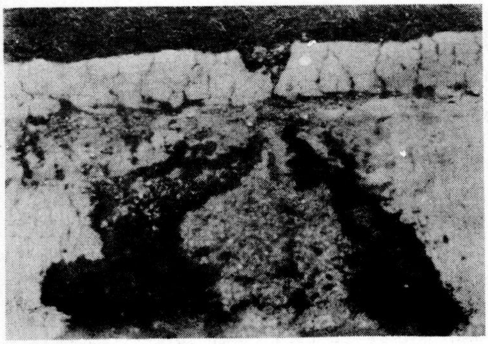

Figure 21.

in the construction, because it has been found that it occurs only in soils of the calcareous glacial drift or soils whose parentage was of a calcareous nature.

This type of failure is virtually absent in pavements in the Driftless Area and in those laid on soils of crystalline rock origin.

It has been observed that this type of failure is quite common on pavements laid on soils of the calcareous glacial drift but that when these pavements built of the same materials pass onto the soils of the Driftless Area or onto soils of the Cambrian Drift the failure ceases to exist.

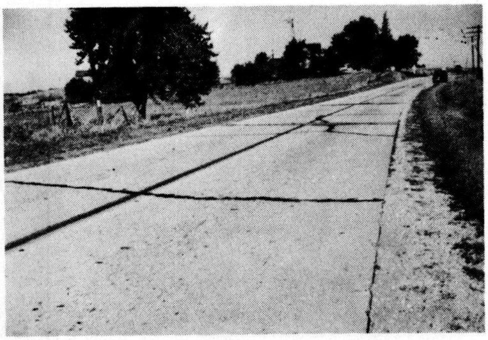

Figure 22.

The failure has been found in pavements built of igneous aggregates, granite, basalt, rhyolite, and similar substances, where they lay on the red clay glacial drift, which is of calcareous origin.

Another striking demonstration of the influence of the soil-moisture influence on this type of failure has been found on several contiguous sections of the same highway constructed in 1922 and 1923 with dolomitic aggregates from the same geological formation. The pavement was constructed without transverse joints of any kind, except construction joints at the end of a day's run.

The westerly four or five miles of this section are laid on soils identified as the Clyde Series, consisting of a colluvial soil formed in the lower lying areas from the overwash of the calcareous glacial drift.

Figure 22 shows a view of this section, where "D" cracking and consequent blowups have occurred at intervals of around 250 to 300 ft.

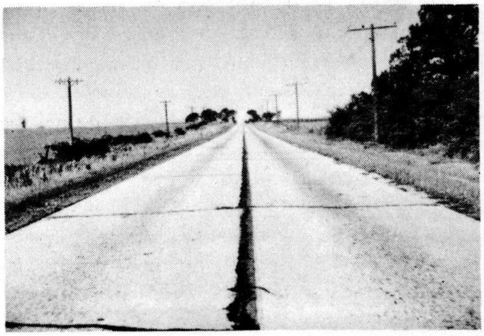

Figure 23.

The next section lies on soils classified as the Miami Series, consisting largely of the ground moraine of the calcareous glacial drift mantled to some extent with a windborne soil.

Substantially less failure of this kind has occurred in the 25 yr. of its service on this section.

The section contiguous to this to the east lies on glacial outwash soils, largely sand and gravel with but a thin windborne mantle, and which are classified in the Fox and Waukesha Series.

A typical view of this 25 yr. old section some 6 or 7 mi. in length, shown in Figure 23 shows that "D" cracking and blowups are virtually non-existent on it.

The three sections are so located that traffic on them is almost identical.

The views shown in Figures 19 and 20 were taken of pavements lying on the red clay lacustrine deposits in the now extinct glacial Lake Wisconsin, while that shown in Figure 21 is one of a pavement lying in the red clay glacial drift, all of which soils are of calcareous parentage.

Blowups - These are conceived to be the buckling, accompanied by a crushing of the concrete, due to compressive forces generated by the restraint of the expansion in pavements built without expansion joints. While this phenomenon has been observed in pavements without provision for expansion, it has also taken place to a lesser degree, in pavements having expansion joints placed at relatively close intervals, of the order of 50 ft. or thereabouts.

The early recollections of these blowups during the years when pavements had been built without such joints, are that they generally occurred when high temperatures were immediately preceded by precipitation.

Blowups have and do occur. The evidence secured by the investigation of the performances of pavements, however, establishes that the simple factor of restrained expansion is not the initial cause of blowups. A simple computation of the compressive stresses induced by restrained expansion develops that these seldom exceed about 1,200 psi., which leaves a factor of safety of around three to five, or more, if the increases in compressive strength due to age are taken into account. If other correction factors, such as subgrade friction, volumetric expansion, etc. are applied, some computations have been seen where this stress was reduced to about 600 psi.

Actual investigation shows that there is a substantial mileage of the pavements without expansion joints, built in the 1921-1924 and in the 1926-1927 periods, which are still entirely free from blowups. The evidence secured points quite conclusively that the soil-moisture conditions are primary factors in causing this phenomenon.

Figure 23 shows a typical view of a pavement built without expansion joints lying on glacial outwash materials providing adequate subdrainage, on which no blowups had occurred in about 25 yr. of service.

Figure 24 shows another pavement, in a different section of the state, which was built on the Plainfield sands (also glacial outwash) in 1927. The pavement has no expansion joints, and no blowups have occurred over distances of several miles where the location of the highway is close to the terminal moraine. Farther along on this same highway, the soil still being outwash sand, contains a small percentage of silt to classify it as a sandy loam, and is farther away from the terminal moraine; and, with a manifest higher ground water table, some impingement has occurred between the edges of slabs formed by cracks or joints, necessitating the cutting of transverse relief trenches through the pavement to preclude actual blowups.

On another job, in a still different section of the state, the pavement was built in 1924 on the Poygan soils, a soil similar to the Clydes, but derived from the red clay glacial till. No blowups have occurred on this pavement in over 20 yr. of service.

Blowups are also quite uncommon in pavements laid on the glacial drift derived from the crystalline rocks.

Innumerable further examples could be cited.

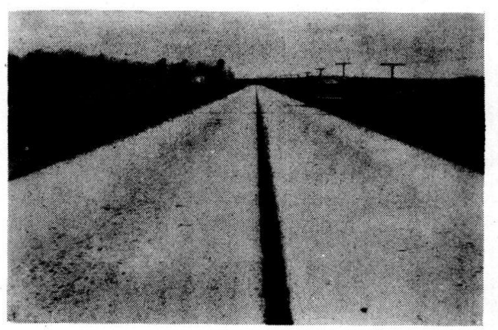

Figure 24.

Blowups can perhaps be divided into two classes. The first occurs at cracks, or at joints without other further previous disintegration of the pavement, the other is consequent to the disintegration caused by "D" cracking.

The former is due initially to some subgrade disturbance, such as a nonfully recovered slight frost heave, or a "swell" taking place in the subgrade soil, at the crack or joint, due to an infiltration of moisture, either of which would lift the locus out of the true axial plane, when, upon subsequent pressure due to the expansive effort upon a raise in temperature, the force is out of the axial plane, with consequent uplift and rupture.

On the latter type, the pavement having disintegrated to the wedge-shaped edge at the crack or joint, the expansive effort will force one or the other edges over the contiguous wedge, with the resultant rupture.

Figure 25.

An example of this is shown in Figure 25. On the far side of the view, the "blowup" had occurred on a previous occasion, on the near side, just shortly before the view was taken, while in the center it had not as yet occurred.

Rhythmic Waviness - This is a condition of rather uniformly spaced irregularities in the surface of rigid pavement, causing a periodic vertical swaying in the vehicles travelling over the pavement. This may either be permanent or transitory. The permanent type was touched upon in the previous discussion, as being due to a permanent deflection at the center of the slab, generally accompanied by faulting at the joints.

On the transitory type, it has been repeatedly observed that the condition is not always of equal intensity at all times, nor does it occur on all sections of the same highway at the same time. In all observations made, regardless of the time of year, it was found, with only one or another rare exception, that the pavement at the joint is high, while the center of the slab may perhaps be at its original elevation. In other words, the crests of the waves are at the joints, while the cusps are at the center of the slab.

The condition is generally prevalent only on pavements with joints, particularly those with dowelled joints. It has rarely, if ever, been observed on pavements built without joints, and no recollection of the condition is made of its occurrence on pavements with joints, but without dowels.

The condition has been observed on soils of all types.

One notable observation on a 15-mi. section of pavement is that it gets wavy nearly every fall, and remains that way until late the following spring. Other recurrent observations on many roads have been that, on the same day, on the same road, one passes from smooth to wavy sections and vice versa. The two conditions, the intermittent occurrence of this phenomena, and the fact that the pavement is always high at the joints, no matter what the temperatures are, should indicate that the differential lengthening or shortening between the top and bottom of the slab, due to differences in temperature at these locations, has but an insignificant bearing on the occurrence of this waviness.

This waviness is more apt to be caused by a differential in the moisture content in the subgrade soil, being higher at a joint, than at some distance from it. This may particularly be the case when the soil is at or near its moisture-holding capacity, when it will absorb the infiltered moisture, which causes some swelling in the soil, raising the slab at the joint. During cool or cold weather, the joints not having the same insulating properties as the slab, or because the slab away from the joint still retains heat previously absorbed, there will be greater movement of moisture from lower zones to the upper zones of the subgrade at the joints, increasing the moisture content of the soil in that location, with consequent swell of the soil and uplift of the pavement. When this moisture subsequently freezes, the waviness will remain until thawing sets in, and the moisture has receded to restore the normal condition.

The only cases of a reversal of this condition, namely, where the joints were low, were observed through a region where the pavement rested on a subgrade composed largely of fine sandy material, in a zone of a permanently high water table, so that considerable saturation of the sand was encountered. In this condition, some of the subgrade material was undoubtedly extruded through the joints by an action similar to pumping; and not being a plastic soil, was not replaced with material moved up to the joint under load deflections from regions nearer the center of the slabs.

Other Types of Failure - There are a substantial number of failures under certain soil conditions, where rectangular sections of one or the other side of the center joint break down and settle to a de-

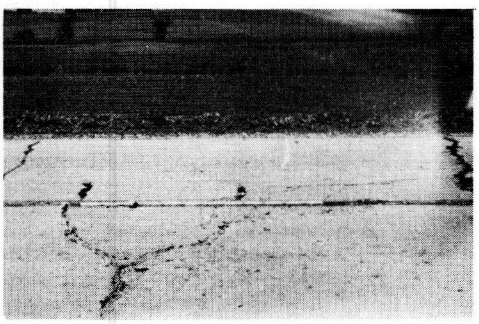

Figure 26.

gree. Some of these are shown in Figure 26 and in the background view in Figure 27. This would be indicative of a greater moisture content in the soils along the center line than along the edges, with consequent loss of bearing power. In the particular region of the view shown in Figure 27 the soil is a sandy loam, which overlies the Eau Claire shales as a very thin mantle, and in this region these shales are aquifers. In this case also, the grade line was not lifted sufficient-

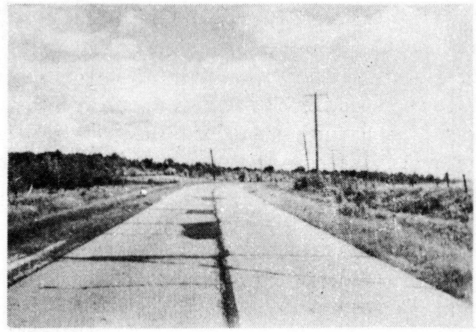

Figure 27.

ly high to permit full drainage of the sandy loam subgrade.

Similar types of failures have been observed on soils in the silt ranges.

While many interior corner breaks have been observed, generally of the nature described under Pumping and Faulting, exterior corner breaks, even in nonthickened edge pavements, are conspicuous by their absence, even under the most adverse soil conditions.

Frost Heave and Frost Lift - These phenomena are soil-moisture-temperature functions rather than pavement failures, and the mechanics thereof will be discussed at greater length under the caption of "Subbase Courses." However, some comment on their occurrence will be included at this time.

The region of their most severe occurrence, and consequent most detrimental effects on pavements, is in the flat phase of the Colby (Spencer) silt loam soils in the central part of the state. These soils consist of a windborne material deposited on the glacial drift of granitic rock origin in the region of the Older Drift, or where the windborne silt is deposited directly on the granitic bedrock. In this region there is also the contact between the granitic rocks and the water-bearing Mt. Simon sandstone or Eau Claire shales of the lower Cambrian formations, and in some cases the drift overlies these latter formations.

The surficial soils and the drift are rather thin, ranging from a few feet to

10 or 12 ft. in thickness over these rock formations. The igneous granites being highly impervious, and the lower Cambrian formations being aquifers, provides a condition where copious supplies of moisture are available to come within the forces generated by heat conduction, with their subsequent freezing.

The condition is extended somewhat to the north where the late Wisconsin Glacial Drift overrode the earlier drift, but in rather thin formations.

Severe conditions of differential frost heave are also found in some of the sandy loam outwash soils where they form but thin mantles over these lower Cambrian formations.

Differential heaves are also distributed through the glacial drift of the Wisconsin period in both the igneous and calcareous drift and in the red clay glacial till, as well as in many sand formations. These are due to differential stratification of the soil on one hand, and to differentials in available moisture on the other hand.

Differential frost heaves are quite uncommon in the Driftless Area, even though most of these soils are predominantly in the silt fraction. Appreciable uniform frost lift is also not noticeable in this area, because of the porosity of the soils allowing room for the expansion of the moisture upon freezing. However, the consequent effects upon thawing are quite parallel to those where freezing is accompanied by appreciable lift of the grade line.

Types of Failures Not Mentioned - There may also be other types of pavement failure which were not mentioned, because they are not prevalent in Wisconsin; however, it may be possible to also associate some of these to the conditions of their subsurface exposures, which, if possible, would provide at least another step toward the solution of the problems generated by them.

SOILS REACTIONS

The study of pavement performance related to the soils and associated influences is indicative of some reactions that take place which are not recognizable through tests of, and on, the soils.

These tests, in the laboratory or in the field, are useful tools for the determination of some of the characteristics and potential reactions of the soils. The limitations of such tests, however, are that they indicate or yield results representative only of their properties under the conditions of the test, and no assurance is obtained that these conditions will prevail under the particular circumstances of exposure to which they will be subjected under actual service.

The soils, considered generically through all the ranges of grain size, exhibit certain properties in their reaction to moisture. Sands are quite unstable when dry, but with given moisture contents are quite firm. In the coarser size ranges they are very permeable and are not affected seriously by their moisture content. Approaching the finer end of the gradation band their permeability diminishes, accompanied by an increase in their moisture-holding properties, and while they are more susceptible to swell or bulking under certain moisture contents, their load-bearing properties are not too seriously impaired when confined.

Soils in the silt family, when of uniform grain size, are possessed of permeability, although this is rather slow. Comparisons of pavement performance on soils in this category, when of rather uniform grain size, are indicative of yielding fair support for rigid pavements but are subject to ready deformation under the more concentrated loads imparted through flexible type pavements.

Soils in the clay range, because of their minute grain size and platy structure, causing overlapping of the pore spaces between other particles, offer substantial resistance to percolation.

Within these basic types are mixtures of the several classes influencing the basic types proportionately.

Within the mass of any volume of soil the space is filled with particles of soil, air, and water, in varying proportions.

In terms of the proportion of moisture, when a deficiency of moisture exists the pendular moisture between contiguous soil particles may be lacking, and the film moisture surrounding the soil particles diminished so that unsatisfied fields of force or unsatisfied surface energy exists surrounding the soil particles. In the effort to establish equilibrium the soil particles endeavor to attract the remaining film moisture from contiguous particles to themselves, thereby generating tensile forces within the soils body, which is cumulative between contiguous particles to the point where, in certain soils types, the cumulative tensile forces from opposite directions cause fissures to form at the critical locations. Such fissures may be both vertical and horizontal. After initial fissuring, the restraint offered by the forces acting in opposite directions is diminished, and the cumulative effect of the tension in the zones between fissures will tend to cause these to become wider and more open.

In soils, particularly those approaching the coarser grain sizes, where the forces of surface energy are not as dominant, the dehydration of the soils causes some shrinkage, but this is conceived to usually be dissipated vertically, tending to cause of condition approaching the maximum dry density of the soil.

Under the circumstances attendant to a deficiency in moisture under natural conditions, soils of the coarser grain sizes tend to become unstable, if not confined, while those in the finer ranges or graded soils become hard and not susceptible to deformation or displacement.

Under these conditions, also, they will readily absorb moisture, displacing the excess of contained air, and because of this will evidence no untoward reactions in pavements constructed on them as long as the soil moisture proportions remain below the so-called capillary capacity of the soil.

When the soils are at their natural moisture-holding capacities under conditions permitting of free internal drainage, or capillary capacity, an optimum (not necessarily the so-called Proctor optimum) condition is conceived to prevail. This can be considered the condition of equilibrium between the soil moisture and air distribution in the mass. The soil is firm and not readily susceptible to deformation under ordinary loadings, which is quite manifest during construction operations, when the construction traffic causes neither rutting or dusting in the surfaces of the earth runways being used.

When in this condition, however, the soils do not readily absorb or dispose of additional moisture becoming available to them, and excessive moisture contents in given zones will readily be established as additional moisture seeks entry into the body of the soils.

Under such conditions the excess moisture competes for space with the occupants of the mass. Air is further displaced, and the soil particles tend to be bouyed apart from the previous intimate contact with each other. Freezing tends toward further separation of the soil particles due to the expansive force of moisture when freezing. Upon thawing, a dispersion of the soil particles throughout the melt water can be conceived to be existent, yielding but very little, if any, supporting value.

The extreme conditions consequent upon freezing were observed in the condition of stabilized base courses, constructed under optimum moisture contents, out of crushed and graded gravels or crushed ledge rock, to densities of around 135-145 pcf.

Upon cutting a hole through the dense graded bituminous surfaces lying on these, late in April or early in May, the previously dense base course, which rang like concrete when hit with a hammer before the mat was placed, was found to be loose enough to permit easy removal of the base course material, and a pick could be driven into it to a depth of several inches with very little effort.

Moisture entering these base courses from subsurface sources under the influence of the forces of heat conduction, when low atmospheric temperatures prevail, cannot escape because of the impervious bituminous surface. This moisture freezes and, due to the expansive force of this moisture upon freezing, disrupts the dense structure of the base course, leaving it loose and relatively unconsolidated upon thawing. No particular harm accrued to the surface, however, even under repeated applications of heavy loads, except in one case where the base course contained a certain amount of clay, in which case some fissuring or turtle back cracking occurred in the relatively thin bituminous surface overlying it, as also in another case where the mat was less than one inch thick. The slight movement inherent in the base course in being reconsolidated under traffic, after the moisture had drained out, was sufficient to cause fissuring in this thin mat.

Further observations on base courses during this period of the year disclosed free water hanging in the lower portion of the base courses even though these overlay granular materials which were merely damp. In one case the underlying material was a coarse sand, in another a coarsely, crusher-run, crushed stone, with rather open voids.

Whether, under the conditions of excess moisture, the ultimate severity engendered by freezing took place or not, the dispersed condition of the soil under excess moisture conditions is one of degree rather than of essence.

Lacking the intimate contact between contiguous grains, and moisture in excess quantities being readily displaceable, the soils are rendered susceptible to deformation and displacement, the amount and character of which is contingent on the characteristics and constitution of the soils, and on the characteristics of the pavement itself.

Deformations or displacements of the soils under rigid type pavements are manifested by transverse and longitudinal cracking, pumping, faulting, displacement of slabs, and similar features. In flexible type pavements these are manifested by turtle back cracking, rutting with or without upward displacement of the areas contiguous to the ruts, and complete breaks through the pavement and base course.

It is not only the magnitude of the loads imposed upon the pavement, but also the frequency of the incidence of these loads, that is a potent factor in the ultimate performance of the pavement. The infrequent application of loads within legal limits, under many conditions of excess moisture in the soils, may reflect no particular harm in the pavement, where a frequent incidence of these loads will progressively lead to the destruction of the pavement.

This is manifested by the condition of pavements lying on soils conducive to detrimental reactions, but on which the incidence of heavy loads is quite infrequent, and by the difference in the condition of contiguous pavement lanes where the flow of heavy traffic is largely unidirectional.

Moving traffic or loads traversing the pavement impart energy to the pavement, which in part is transmitted through the pavement to the underlying soils. This causes a vibration or agitation in the soils, tending to bring the dispersed particles into contact with each other, with a consequent displacement of moisture. Since this moisture cannot be dispersed downward or laterally, its tendency will be to come to the surface, generating a still greater loss of sustaining power at the contact between the soil and the pavement or base course. The action, and the consequent result, can be described as being quite parallel to that of tapping a moist area of soil with the soles of the shoes, or tapping a moist soil pat with the fingers or palm of the hand, during which moisture is brought to the surface.

When loads causing this reaction are infrequently applied, the time interval between successive loads permits the moisture brought up to recede into the body of the soil, thereby maintaining some semblance of supporting value. On the other hand, the frequent incidence of loads of this magnitude will tend to keep this moisture at the contact between the pavement or base course and the subgrade, or to give rise to accretions of moisture in this zone, with the consequent diminishing bearing power of the soils. In this light, the factor often spoken of as fatigue in the pavement in reality is fatigue of the supporting medium, the results of which are reflected in the pavement itself.

Base courses of porous material with fine interstitial pore space in contact with subgrades under these conditions may tend to absorb portions of this moisture to alleviate the ultimate result. However, if the pore space is large, the slurry, when formed, may intrude into the base course materials, tending to separate the contact between the individual aggregate particles, thereby destroying their stability or load transmitting properties.

Passing mention had been previously made on the influence of stratification in the soils column. Stratification of any sort in the soils column acts as an inhibitor toward the free internal drainage of the soil, due to the differences in the force of the surface energy surrounding the soil particles.

Strata of finer grained materials, due to their higher moisture-holding potentials will hold moisture against the action of gravity, and prevent the percolation from them into underlying strata of coarser materials; or, because of their moisture-holding potential being satisfied, will inhibit percolation from overlying strata of coarser material.

Strata of coarser material, under topographic influences conducive thereto, will act as aquifers to keep the moisture-holding potentials of the finer materials satisfied.

Figure 28 shows another view of a rather highly stratified waterborne deposit, which would cause trouble to pavements laid on it, even though a sample of it secured from ordinary borings, when tested, would show the characteristics of a so-called A-2 nonplastic soil.

The ultimate effect upon the soils in the subgrade zone of stratification in the soils column depends upon the degree of stratification, its proximity to the subgrade zone, and the expected temperature differentials between the atmosphere and the earth, which control the magnitude of the forces generated by head conduction.

In like manner, the accumulations of moisture over impervious bedrock forma-

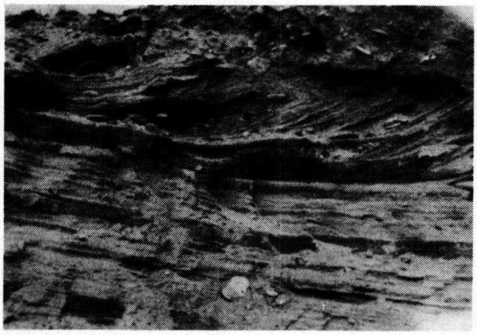

Figure 28.

tions, or water-bearing rock formations, are sensitive to the forces of heat conduction.

It has been stated by Taber *(8)* that, under severe conditions, the influence of these forces may extend to as much as 15 ft. below the surface.

The conditions accompanying severe frost reactions can be considered to be special problems involving provisions for insulation, disposal of relatively copious volumes of moisture, besides the factors of bearing; these will be dwelt upon separately in greater detail under the discussion on Subbase Courses.

The topographical conformation of the terrain may or may not have an influence on the reaction of the soils; however, of itself, topographic position of a given site is not conclusive with respect to the effect upon the soils.

Some of the worst conditions have been encountered on high ridge locations, with side slopes falling steeply away from the proximate highway location; while in the more gentle topography comparable soils gave less trouble.

On the other hand, the converse is also true.

Relatively level areas constituting rather flat divides between main drainage courses, with tributary streams apparently encroaching geographically upon each other's territory, are usually contributory to substantial volumes of moisture in the subsurface soils.

In general, however, it has been found that the conditions of surface drainage, without the concurrent consideration of subsurface conditions, are seldom significant of what can be expected under the surface.

PAVEMENTS

The problem of developing methods whereby thicknesses of pavements and other structural features may be determined has been the subject of an extensive amount of work and research. A number of methods have been developed which, or modifications of which, are in more or less common usage. Similarly, extensive work has been done, both in the field and in the laboratory, to ascertain means whereby the bearing power and other properties of soils could be evaluated. These features have been extensively reported and discussed in the literature.

For the design of flexible type pavements, 22 different formulae or methods are presented and discussed in the 1945 *Proceedings* of the Highway Research Board. A review of some of these discloses that, for the same values of load and soil bearing power, thicknesses varying several hundred percent from each other are developed.

For the design of rigid pavements, the so-called Corner Load Theory, the Westergaard Analysis, and adaptations of modifications of these are in more or less general use. These lay emphasis on the flexural stresses generated in pavements under wheel loads, or to combinations of wheel loads and temperature differentials within the slab, and at times though somewhat arbitrarily, attempt to introduce the factor of subgrade support.

Slab lengths, under these precepts of design, are proportioned on the basis of temperature criteria, and provision is made to take care of expansive movements in the pavements. Devices for the transfer of load across slab ends have been developed and used. The feature of aggregate-interlock is credited with load transfer ability across cracks or joints. The thickened edge cross section is credited with being a balanced design with respect to stresses at the corners, edges, and interior areas, whereby economies in the distribution of materials can be affected.

There is an extensive mileage of pavement under service which can be accepted as proof of the theories involved, and the assumptions made in the development of these design practices and formulae. There is also, however, an equal, if not greater, mileage which can be used as proof of an opposite contention.

Serious exterior corner breakage was found to be highly infrequent in pavements, even in those of non-thickened edge design. Failures at interior corners, in pavements of the thickened edge design, and provided with load transfer dowels is, on the other hand, quite common in many pavements. Transverse cracking has frequently been observed to start at the center of a pavement and progress to the edge, as well as starting at the edge and working inward.

The habitual lack of longitudinal cracking in certain areas, and its prevalence in others, even in pavements with full depth longitudinal joints, indicates that the causes of this type of failure must be sought for elsewhere than in temperature stresses.

Pavements in urban areas with curb and gutter and parking lanes are generally found to be in substantially better condition than rural pavements, notwithstanding that city pavements often carry more and heavier loads than do their rural neighbors.

If the over-all factors contributing to pavement performance are taken into account on the basis of the conditions under which typical performance is manifested, and the mechanics of failure are analyzed in the evidence of the characteristics of the failure and the conditions under which it occurs, the problems resolve themselves primarily into considerations of loads as related to the soil-moisture relationships of the supporting medium, and the distribution of these loads over the supporting medium, without excessive deflection or deformation in medium. If, therefore, the manifestations of pavement performance are associated to the conditions of their subsurface exposure, it can be established that reactions often attributed to other causes either do not occur or are quite insignificant in their extent.

The application of loads to the supporting medium of rigid type pavements can be conceived to be required to be done in a manner that will not cause appreciable concentrations of load on the supporting medium or, as can be stated in another manner, must be distributed as uniformly as possible over sufficient area of the supporting medium to develop a reaction throughout, which will be equivalent to the load imposed.

Concentrations of stress will be imposed upon the supporting medium if the pavement is allowed to deflect under load. With deflections under load, movements or displacement of the underlying medium will be encouraged or engendered. These may consist of a vertical compression of the soil, or a longitudinal displacement inherent to the wavelike progress of the deflection under the moving load, or both.

While it may not be possible to absolutely inhibit all deflection, the more the deflection is minimized the less concentration of load will be made effective upon the soil, with a consequent lesser deformation in it.

Since a rigid pavement is supported over its entire area, and since the pavement tends to minimize the deformation of the soil through the restraint it imposes on the upward vertical pressure exerted by the soil in the process of its deformation, it can be conceived to be a statically indeterminate structure whose functioning does not lend itself to ready mathematical analysis.

Also, due to the inherent characteristics of a rigid pavement, it is perhaps more proper to consider the loadings on a lane or axle load basis, rather than on an individual wheel load basis, although either way would not be too critical, however, for the purposes of illustration of some of the foregoing, lane or axle loadings will be used.

If an axle load is considered to be 30,000 lb., consisting of the legal loading of 20,000 lb. plus a 50 percent increase for impact, shock of sudden application of load, frequency of application, and occasional overloads, spread over a lane width of ten feet, supported by a soil conceived of yielding a bearing value or resistance to deformation of five pounds per square inch, an area of 6,000 sq. in. or about 40 sq. ft. would be necessary to sustain this load, or a length of lane of four feet. On the other hand, if this soil will only sustain a load of two pounds per square inch, an area of 100 sq. ft. or 10 linear ft. of lane length will be required to support such load.

From this comparison it can be seen that as the supporting power of the soil increases lesser areas of support are required, which perhaps accounts for, at least in part, the relatively secondary consideration extended to the bearing power of the soil in most of the design theories or methods in vogue for the determinations of thicknesses and general slab design for rigid pavements, which in turn very likely have been developed and investigated under conditions lacking the extreme loss of bearing value encountered in actual service, even though this lack of bearing may appear only in rather thin layers in the surficial zones of the subgrade soils.

Due to the complexities involved, a direct approach toward a satisfactory determination of thicknesses of rigid pavements may be difficult of solution, and while the following demonstration may appear to be rather elementary, it is presented to further illustrate some of the principles involved.

If the deflection of a 7-in. slab is calculated for a span of four feet and a 30,000-lb. load, by the simple deflection formula, a value of about 0.005 in. is developed. For a span of ten feet, for a comparable deflection, a thickness of 16 in. would be necessary. The deflection of a slab 12 in. thick for a span of 10 ft. is about 0.015 in., as computed by this method.

Whether the deflection ought not exceed the first value or whether 0.015 in. can be tolerated is a matter than has not as yet been resolved. However, the experience gained through the service of old pavements which were later resurfaced with three to five inches of either Portland cement concrete or bituminous concrete to produce a gross thickness of around 12 in. is indicative that quite satisfactory results are attainable with such thickness. This might be indicative that the restraint against deformation of the soil engendered by the 12-in. slab would be sufficient to minimize the deflection to less than the 0.015 determined by the simple computation.

Whether soils conditions such as produce results requiring greater thicknesses of pavement can be ascertained by test methods or whether they can only be evaluated through experience with the performance of pavements on soils of the same constitution, mode of deposition, geological influences and similar influences in the area, is a matter that requires further study.

The incidence of failures, however, such as longitudinal cracking, interior corner breaks, failure along the center joint, and the general absence of exterior breaks, would indicate that, where these failures did not occur, the extra edge thickness was wasted material, but where they did occur the central portions of the slab were too thin and should at least be more equal to the edge thicknesses, indicating a slab design of uniform thickness throughout.

The foregoing has concerned itself largely with the thickness of the rigid pavements. The prevalence of transverse cracks at intervals of about 15 ft., even when the original slab length was only of the order of 30 ft., whether reinforced or not, is indicative that slab length is a structural feature related to deflections and loads, rather than one of temperature stresses. The premise is therefore made that deflection of the slab must be arrested by a break in its length, which, if it is not, will of its own accord provide such break.

Whether the comparison is directly applicable or not, the relative stiffness of a slab can be illustrated by the length over radius of gyration (l/r) formula used in column design.

This follows in a linear ratio in direct proportion to the length of slab. Thus, for an element of a slab 12 in. wide, and 7 in. thick, and 15 ft. long, the l/r ratio is 90, for one 30 ft. long it is 180; and for 50 ft., 300.

In order to more fully illustrate the principles involved, and the relative reaction of a paving slab, the pavement can be conceived to be a board thrown across a mud puddle.

If this is a relatively long board, the weight of a man stepping on one end of it will tend to slightly sink the end of the board into the mud and raise it off the ground in the center. Hopping on the end of it will whip it up and down in the center, splashing the mud in both directions laterally from it. As the man advances toward the center, the ends tend to rise and the board under the man sinks quite deeply into the mud as he reaches the center, and may break if it is thin enough, and the man is of sufficient weight. If this board is cut in half and the two pieces used, the sinking is not nearly as great at the ends or center, nor is it nearly as possible to whip the board up and down, and the danger of its breaking is minimized.

Since rigid pavements are composed of an elastic material, the situation and reaction is no doubt quite analogous.

The study of the performance of pavements shows, as previously indicated, that there is a substantial amount of transverse cracking in slabs only 30 ft. long, and whether reinforced or not, even on good subgrades composed of materials such as sand or gravel. In order to preclude the irregularity of such cracks with supplemental spalling, these cracks may be predetermined by the installation of planes of weakness at intervals of 15 or 20 ft.

Provisions for expansive movement of the pavement, except when necessary to insulate other vertical structures protruding through the pavement, is not only unnecessary, but actually detrimental to the pavement. As has been pointed out previously, the forces created by restrained expansion are of a relatively low order, well within the ultimate strength of the material, and these joints do close up after a period of service, permitting other cracks and joints to open up.

These openings provide places for the infiltrations of water onto the subgrade, thereby providing, in many cases, the moisture necessary to cause pumping, faulting, and other failure.

The prevention of blowups is a function related to the subgrade materials, rather than one of control within the slab itself by the interposition of spaces to permit of longitudinal movement of the individual slabs formed thereby.

If the pavement is of adequate thickness, as related to the subgrade support, dowel bars or other load transfer devices are quite unnecessary; if it is of insufficient thickness they are quite valueless, and may at times even be detrimental, causing severe failure in the pavement, as described previously under faulting and pumping. Other means of providing so-called load transfer, such as piers under the joints or thickening the edge under the joint, may only be the cause of transfer of the distress away from the joint to the point contiguous to the support.

Longitudinally the slab should be limited to widths of an order of around ten feet to perhaps a maximum of 15 ft. With the variations in the conditions of the soil, even on good subgrades, some variation in support can be expected. A preformed crack or longitudinal joint will permit of some movement of the slab to conform itself to its support, where, if this were not provided, the conformation would be accomplished with a resultant meandering, ragged crack.

Tie bars across such joint do not serve to transfer load to the companion slab, as can be seen from the substantial amount of failure along the center longitudinal joints. In many cases they act to restrain a slight movement of the slab necessary to permit itself to conform to the subgrade condition, and thereby be a cause for longitudinal cracking in the slab. There is evidence to indicate that faulting along the longitudinal joint will take place when tie bars are omitted, and when the heavy loads are largely unidirectional. There is also evidence that this has occurred in pavements provided with tie bars operating under the same conditions. In this case, tie bars were actually found to be broken, and the concrete under them disintegrated. In other cases, where the loading was uniform on both lanes, and the pavement was built on a good subgrade, no evidence of faulting was found even though tie bars were not used.

Observations have also been made of the failure of tongue and groove joints used without tie bars under dissimilar conditions of loading of contiguous lanes. This was in the nature of a crack parallel to the joint and three or four inches distant from it, and with further service the concrete between the crack and joint will spall out.

As has been pointed out, rigid pavements are not particularly sensitive to subgrade soil reactions except when these give bearing values in the low ranges. On the other hand, flexible type pavements are considered to be more directly influenced by the bearing values of the soils because of the difference in the manner in which the loads are transmitted to the subgrade through flexible pavements and their base courses. Flexible pavements are considered to consist of a bituminous surface course resting on a base of graded aggregate, such as gravel or crushed stone, or similar materials.

While a rigid pavement can be considered analogous to a board or plank distributing the load, a flexible pavement can be considered in the light of a series of blocks, each of which must individually carry the load without assistance from the contiguous blocks. In this light, therefore, it may be more proper to consider the loadings on the basis of individual wheel loads, rather than on the basis of axle loads, as in the case of rigid pavements.

There have been, as previously stated, quite a number of methods or formulae proposed through which determinations of thickness might be made. These yield quite variable results, and from a study of the literature it is seen that there is not too much agreement between those who have endeavored to use the same method in different places. A given method may be charged with yielding both overdesign and underdesign.

The fact that a given method may produce either overdesign or underdesign is quite understandable, and perhaps quite logical. Soils, when tested in the laboratory or in the field, may yield quite comparable results. However, in the light of the influences of the meteorological and subsurface conditions in different localities, the soils under service conditions in the one location may not have sustained the loss in bearing value indicated under the conditions of the test, while in another location, even though the soils had measurably comparable characteristics, their actual reaction was considerably more severe than the test might indicate.

It has also been pointed out that the presence of a base course may modify the condition of the subgrade soils, if it is somewhat pervious, to permit of the absorption of moisture extruded from the subgrade soils, due to displacement of it under load, thereby diminishing the loss of bearing of the soil.

The observations made on base courses of high density would indicate that such density is not necessary, or perhaps not even desirable, if the material is otherwise stable or resistant to displacement under traffic.

The lower portion of the base course may preferably have a greater proportion of fine material in it to prevent large voids in contact with the subgrade. Large voids in contact with the subgrade will not hold moisture in suspension, but permit it to pool on the subgrade contact causing the softening of the grade, with consequent intrusion of the soils into the base course material. The finer material will act to retain excess moisture until the soil can naturally absorb it without undue softening, or permit it to bleed off laterally if the base course extends across the shoulders. Enclosing a base course along its sides with relatively impermeable soil will cause the base to act as a reservoir, permitting accumulations of water, thereby softening the grade.

The upper part of the base, if possible, may preferably be somewhat deficient in fines to permit of some void space within which the freezing moisture may expand without affecting the structure of the base course to an appreciable degree.

Flexible pavements similar to rigid pavements carry moving loads whose weight and the energy imparted by them are in part transmitted to the subgrade, and in part dissipated vertically and laterally by friction between the individual particles forming the base course. If such base course were made thick enough, this friction would dissipate the energy imparted by the moving loads to the degree that the pressure on the subgrade would approach a zero value as a minimum.

Such thickness is not necessary because the soils have some supporting power, the value of which as previously set forth, however, is not conclusively determinable. As a concrete example the following type case may be cited. On a certain highway a flexible type pavement was constructed, having a crushed stone base about six inches in thickness (compacted to a density of better than 140 pcf. when built) supporting a bituminous concrete wearing surface two inches thick. The subgrade soils are a loessial silt, yielding bearing values of a very low order when tested by customary procedures. On about one and one-half miles of this road the distress in this mat is of a minor order, while on the balance of the project the mat has suffered very severely. In the first case the soil overlies the highly fissured Galena Limestone as a thin mantle, on the balance of the project it is thicker and overlies residual clay formations or the lesser fissured Trenton Limestone formation.

There is also a representative mileage of flexible pavements in various sections of the state supported on combinations of permeable subbase or ballast courses, plus base courses ranging from about 12 to 24 in. in thickness, where an almost fluid condition of the soil existed, rendering the roads impassable nearly every spring, prior to their reconstruction. The number of failures after service up to about ten years is almost negligible.

In the light of the complexities of soils, moisture movements, and traffic loads involved in the considerations of pavement design for either the rigid or the flexible type, it may be difficult to translate the qualitative analysis of the functions of these features into methods whereby quantitative results are directly ascertainable.

It would appear that the application of any formulae or data developed by a specific test or tests must be correlated and tempered with judgement to the service experience of other pavements operating on soils of similar geological origin and environment in the locale, together with the considerations of traffic loads and the meteorological influences prevalent in the area.

If such correlation is made it may be possible to develop a stock of information on existing pavements which can be extended and made applicable in the design of new work.

SUB-BASE COURSES

A subbase or ballast course or lift course, as it is also sometimes referred to, performs several functions in the intermediate zone between the pavement or base course and the earth subgrade. The more generic term subbase will be used to refer to courses of this kind, regardless of the type of material out of which they are built, while the term ballast course or ballast will be reserved for freely or fully draining subbases built of permeable granular material, extending across the full width of the roadbed from ditch to ditch, or, if not built to this width, when they are provided with subsurface drainage conduits such as tile, perforated pipe, or similar facilities.

The interposition of a subbase built of comparatively less expensive materials, which are influenced by moisture to a lesser degree than the subgrade soils, permits of a reduction in the thickness of the pavements or base courses placed on them. Through their being influenced to a lesser degree by moisture they will be less susceptible to deformation or displacement under load. In the case of rigid pavements, this will accomplish a greater distribution of the load through thinner pavements, because of lesser deflection, than if these pavements were placed directly on the subgrade soils, and will thereby also inhibit pumping and faulting and other distress consequential to deflection. Subbases under base courses for flexible pavements will accomplish results in a parallel or comparable manner.

The selection of the type and characteristics of the subbase depends on the materials available and the conditions that must be accommodated. While in Wisconsin the only experience has been with freely draining ballast courses built of permeable materials, the observations and analysis of the functions of these ballast courses leads to the inference that where the combination of meteorological and subsurface conditions are not nearly as severe as in Wisconsin subbases of other types may prove quite effective.

Selected soil, not as susceptible to deformation under the influence of moisture, might be satisfactory where the exposure to moisture is of a low order, especially where the temperature differential between the earth and atmosphere is not too great or of too long duration, and where, if freezing takes place, the frost penetration is relatively small, so that the subsequent melt water can dissipate itself by percolation into the ground.

Under conditions of a similar nature, the waterproofing or stabilizing of the subgrade soils by the incorporation of materials adaptable for the purpose, such as Portland cement or bituminous materials, should give quite satisfactory results.

On the other hand, where the combination of long periods of exposure to low temperatures, with consequent deep penetration of such temperatures, together with subsurface conditions providing a source of moisture within the influence of the forces generated by heat conduction, is existent, the problem becomes one of control of ice formations and disposition of melt water.

Under such circumstances, the ballast course not only must serve to spread the loads, but also act as an insulator and a subsequent drainage facility.

As was previously described, when the atmospheric temperature is lower than that of the earth, there is radiation of heat from the surface, which heat is replaced by heat conducted towards the surface from below. These heat conduction currents activate moisture movements toward the surface, and, being confined by an impervious pavement surface, this moisture accumulates in the subgrade zone.

This moisture accumulation is directly proportional to the temperature differential, the period of duration of such temperature differentials and the amount of moisture available. Such moisture can be from that held in the interstitial pore space of the soils, from moisture-bearing or moisture-holding strata, or from lower lying accumulations, or from snow piled on the shoulders of the road, especially when the same disappears without appreciable runoff.

If the temperature becomes low enough the moisture in the subgrade soils will freeze, and in some soils freeze in the form of segregated ice lenses or crystals. If the descent of the frozen zone is slow, the accumulation of ice in the subgrade zone will be greater than if severe cold weather freezes the soils to substantial depths quite rapidly, because the time element is a factor in the amount of moisture rise. After the soils have frozen to a depth at which the further freezing process is retarded, ice crystal accumulations grow at this lower limit of frost penetration, and when this occurs, due to the high intermolecular attraction between ice and water, no more moisture rises to the surface.

The only source of moisture remaining then for the subgrade zone is the melt water from slowly melting snows on the shoulders or inslopes.

Upon thawing, the melt water contained in the soil cannot escape because of the impervious frozen zone beneath, whereupon an almost fluid condition of the soil will be generated, whereupon the pavement will break under load, become displaced, and the subgrade soils may at times be extruded along the edge of the pavement, as shown in Figure 29.

If a pervious ballast course of sand or similar material is interposed between the pavement or base course and the soils, the early accumulations of moisture rising with conduction currents of heat, and

Figure 29.

later those from the snow on the shoulders, will enter such ballast course and the base course, in which such moisture will freeze.

Due to the higher insulating properties of a porous material, the penetration of the freezing isotherm into the underlying soils will be generally delayed or postponed to some extent until severely cold weather sets in, causing a rapid freeze of the underlying soils, after which ice crystal formation at the lower limit of frost penetration can set in, thereby minimizing the amount of ice crystal formation in the soils in contact with the ballast course.

Evidence of the dual layers of ice crystal formation has been seen in explorations made on certain work during March of a certain year. There was a layer about four feet thick of glacial till material, consisting of sand and gravel, with somewhat over thirty percent of the material falling into the silt and clay fractions, overlying a layer of clayey silt about two feet thick, which in turn was lying on sand at the water table. Ice crystal formations to a depth of 14 inches were found in the material immediately below the concrete pavement. Below this the till material was frozen solid, to the layer of clayey silt, which again was filled with ice crystal formations. The moisture content of the intervening frozen material was about 7 percent, which was also its moisture content before freezing took place.

Further evidence of such action can be inferred from the entire absence, or greatly reduced size, of differential frost heaves on pavements lying on such ballast courses, in regions where, prior to such construction, the heaves were so great that ramps of gravel or cinders or whatever material there was on hand had to be built approaching these heaves so that traffic could get over them.

Some frost lift of a very uniform nature has been observed on pavements constructed on ballast courses of this kind, both by taking measurements of elevations and by observations at bridge abutments or railroad tracks which did not lift comparably. Whether this lift was from deep conditions or from ice crystal formations in the subgrade soils in contact with the ballast has not been explored, but the belief is that the latter is the case.

Upon thawing, except for a disrupting of the density of highly compacted base courses, no particular harm will result because of the inherent stability of granular materials, even in the presence of water, provided measures have been taken so that the melt water can drain away, either by full width construction or by the installation of pipe conduits for the purpose.

In the event some ice crystal formation had taken place under the ballast course in the subgrade soils, this will be accompanied by some lift of the pavement and its substructure. If the melt water can escape, and thawing has progressed through the ballast course and attacks the ice crystals in the subgrade soils, these melt and form water.

Under the condition generated by the ice and the excess moisture from the melting ice in the subgrade soils, the soil particles are dispersed in this water, thereby yielding readily to the pressure imposed by the overlying structure and traffic loads. As this yielding takes place gradually and uniformly with the formation of water from the melting ice, this water is displaced. Since it cannot escape downward into the impervious frozen underlying soils, it is forced upward into the ballast course, from whence it can bleed or drain off as it accumulates.

The displacement of this water into the ballast course, from which it can be drained away, prevents the extreme loss of bearing power of the soil which would result, were such ballast course not provided, or if a ballast course could not drain itself quite freely, but had to wait for dissipation of the melt water by downward percolation after the entire frozen zone had completely thawed, and downward percolation of moisture was again possible.

An experience of this kind was encountered on a highway where sand gravel ballast material had been trenched into the subgrade, during construction, to a depth of 18 inches, but to a width of only two feet wider on each side than the width of the pavement built over it. In less than five years time the mud from below was pumping out in the shoulder alongside the pavement, as shown in Figure 30. The stone drains placed intermittently through the shoulder, in the hope that they would permit of the drainage of this ballast course, proved quite valueless.

During periods of alternating freezing and thawing occurring during the late winter, differential frost heaves often grow or swell substanitally with rapidity in subgrades not protected by permeable ballast courses.

As thawing takes place in the surficial zones of the heave and contiguous area, moisture in the form of melt water becomes available to nourish the existent ice crystal formation.

This is free water, and the ice for-

Figure 30.

mation is a zone providing a temperature differential. This temperature differential, and the high molecular attraction of ice for water, conducts this moisture into the heaved area from places even longitudinally along the road. Since this is free water, it provides a rather copious quantity to nourish the existent ice crystals, causing them to grow, with consequent swelling or expansion of the heave itself.

In a permeable ballast course interposed between the pavement and the subgrade, even though this contains ice, there is not apt to be any segregated crystal formation. Thawing converts the ice into water which drains off. By the time thawing has progressed through the ballast course there is little danger of alternate freezing or thawing in the soil, and with the absorption of moisture into the ballast course the source of moisture to nourish an ice crystal formation, if any is existent, would be lacking.

Freely draining permeable ballast courses, extending from ditch to ditch, have been in service on a representative mileage in the state for periods ranging up to 11 years, and in one case 15 yr., under both rigid and flexible type pavements, in areas where the worst conditions in the state prevail, and to date the failures on these have been so few that, for all intents and purposes, it could be stated that practically no failures have occurred.

The view shown in Figure 31 is that of a U. S. highway in the northern part of the state which was surfaced with a bituminous mat on a relatively thin base, and at the time the view was taken was again about in a condition so that traffic could again get through, although not too comfortably. The soil is either a wind or water deposited silt, its exact origin not having been exactly determined, overlying the granitic bedrock as a mantle about five feet in thickness. As can be seen, the effect of frost lift, and consequent entrapment of melt water, was quite uniform throughout its extent. This condition is not unusual on this highway, both with respect to the frequency of its recurrence and the consequent effects, necessitating recurrent rehabilitation of its surface.

Figure 31.

While the view shown in Figure 32 was not taken on the same highway, the conditions prevailing before reconstruction were almost identical. The view shows the condition after its reconstruction, completed early during the war, consisting of a road-mixed bituminous mat on a six-inch dense graded gravel base, supported by an 18-in. freely draining sand ballast course. The area in which this highway is located is that shown in previous views where the fluid soil extruded from beneath rigid pavement.

Figure 33 is a view of a truck that became mired on a concrete pavement.

The additional surfacing material placed over the pavement was insufficient to prevent the truck from breaking through the pavement slab and getting stuck. This also is in the area where the extrusion of the subgrade soils along the sides of the pavement is common.

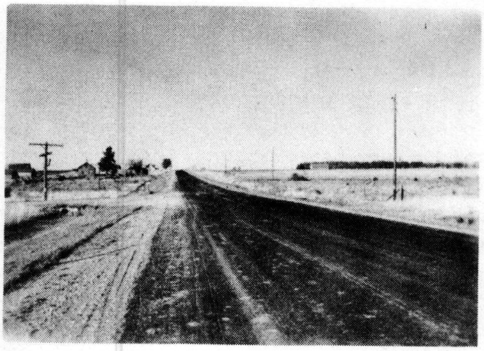

Figure 32.

About 20 mi. of this highway, where these were common occurrences, has now been rebuilt as three projects in successive years. The first project replaced the old pavement when it was only about 15 yr. old. In the reconstruction freely draining sand ballast courses were provided, the thicknesses being 12, 18, and 24 in. respectively on the individual projects.

Figure 34 is a view of the second of these projects, which has an 18-in. bal-

Figure 33.

last course, after about 10 yr. of service, during which it was subjected to the severe conditions previously mentioned of the winter and spring seasons of 1940-1941 and 1942-1943. The condition of the entire 20-mi. section is quite comparable to that shown in this view.

It can further be set forth that, with what actual investigation of these ballast courses had been made, no evidence to date has yet been found of an intrusion of the subgrade soils into the ballast course so that it would become fouled and impermeable. The relatively small pore space of the sand seems to effectively be able to permit the water to filter into it without any disturbance in the underlying soils, perhaps because the action is rather slow and gentle. On the other hand, it is known that crushed stone base courses, which had the fines removed from them before placing, have almost entirely disappeared. This is conceived to be because of the larger void space, and due

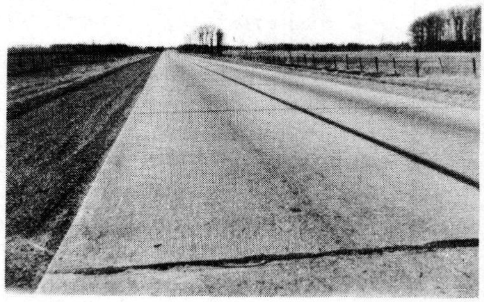

Figure 34.

to the shape of the particles a much lesser degree of bearing is obtained, so that concentrations of load permit the individual particles to be pushed into the soil, which then forces the stone particles out of contact with each other, thus successively diminishing the bearing and effectiveness of the base course.

There is no way in which the necessary and economical thickness of a subbase or ballast course can be measured or arrived at except through experience, an evaluation of the conditions of exposure, and an appreciation of the principles of moisture movements and freezing, and the disposition of moisture, regardless of whether freezing had occurred or not.

The generally overall satisfactory service performance rendered by ballast courses ranging from 9 to twenty-four inches in thickness can probably be considered as significant of some overdesign in those cases where thicknesses in the higher ranges were employed, and that therefore lesser depths would render comparable service.

On this background, ballast courses

ranging from 9 to 15 in. in thickness are now considered for general use for both rigid and flexible type pavements in most cases where the use of ballast courses are indicated by the soil-moisture-freezing and subsequent break-up reactions. In those areas where extreme differential frost heave conditions are manifested these can be increased to be in the range of from 12 to 15 or 18 in. The differentiation in thickness is also made, in part at least, on the expectable traffic to be carried by the particular highway.

In some soils, notably certain clay type soils of platy structure, where the movement of moisture through the soils, even under freezing conditions is apt to be minimized by the overlapping of the soil particles upon each other due to their shape, but which soils are very conducive to pumping, ballast courses of even lesser thicknesses, as from 6 to 8 in., should prove effective in inhibiting pumping.

The porous material overlying such soils will provide a storage space for the moisture entering through fissures in the pavement, from whence it can bleed out laterally without too much effect on the underlying soil, so that the slurry formation which otherwise occurs will not take place.

Since most porous sands will not offer much resistance to displacement when unconfined, it is necessary to effect some stabilization of the upper surfaces of such ballast courses. Since ballast courses are essentially used where detrimental moisture movements occur, stabilization with earth or earthy materials is not indicated, because these are apt to cause concentrations of moisture immediately below the pavement or base course, and by becoming somewhat unstable themselves in the presence of moisture are apt to vitiate to some degree the effect of the ballast course.

In Wisconsin, the practice now is to stabilize the upper surfaces of these ballast courses with several inches of crushed and graded gravel or crushed stone, the thickness ranging from 3 to 5 in., depending upon the traffic characteristics of the highway. This not only acts to provide a roadway surface for construction operations which, if it were not provided, would permit of severe displacement of the ballast material, the subsequent truing up of which just prior to placing the pavement will create differentials in density, which in turn will be reflected through the pavement by erratic cracking; but also provides a strengthening of the entire substructure, thereby permitting of generally thinner slab designs for rigid pavements than might otherwise be necessary. The presence of coarser material in the immediate subgrade will also tend to minimize the rhythmic waviness occurring in pavements laid on sand subgrades, especially when the sand tends toward the finer grain sizes, since the graded material will tend to absorb moisture differentials within itself with much less consequent bulking or fluffing than if the finer sands were immediately exposed to moisture differentials, and will prevent the extrusion of the finer sands when saturated with water and subjected to heavy traffic loads.

CONCLUSION

The study of pavements under service for many years in the different geological provinces of the state demonstrates that their performance is not contingent upon the abstract physical properties of the soils alone, but rather on the extent to which these physical properties are activated, and the manner in which they react under the several influences brought to bear upon them.

These influences vary widely, not only from place to place, but also at different times in the same place, wherefore soils of apparently the same constitution, and handled with the same care during construction, may yield different reactions, which can often lead to an interpretation of the reflected pavement performance as being caused by factors other than by disturbances in the supporting medium, if these conditions are not understood and recognized.

Rigid and flexible type pavements on gravel or crushed stone base courses are basically different in their functional relations to the supporting medium, generating stresses and reactions in the soils which are at quite a variance with each other.

Moving loads have a different influence on the pavements and the supporting medium than that created by static loadings.

The processes activated in the soils through the meteorological influences can be qualitatively evaluated through the factors of the constitution and geology of the soils and of the area.

Some of the elements contributing to the variable performance or behavior of the soils appear to have been isolated and have been brought out in the foregoing discussion.

Whether quantitative determinations on some of these elements can be made, or whether exact differentiations in degree of reaction can be correlated to measured differentials in some of these factors, is at this time still an unanswered question.

The soils are materials formed by natural processes and have not been manufactured in accordance with a set of specifications drawn to limiting ranges, neither has their deposition been made to a geometrical plan, and in their subsequent development they have been subjected to the vagaries of the meteorological conditions and the underlying other soil or rock formations.

In Wisconsin the weather conditions generating some of the potentials for severe disturbances in the soil are active for about six months of the year, but the conditions that are caused thereby, and which must be considered in design, prevail for only about six weeks in the spring of the year, and are not the same every year, which yields only a very limited time in which extensive analyses for quantitative determinations could be made.

It is therefore quite improbable that a set of hard and fast rules can be developed which would be applicable to any and all areas, except to develop a knowledge and familiarity with the individualities of the soils of the various origins, modes of deposition, and general enviroment, together with a study of their reactions under the influences of the locality.

Perhaps a system of classification can be devised in a manner parallel to the Pedological Classification, but which would give greater recognition to the geology of the soils and their environment; but, until such system is devised, the pedological classification correlated to the geology forms an excellent basis upon which differentials in soils reaction can be studied, and a familiarity with them established, and which can be made adaptable to design purposes.

Having established the geological history of the soils, and their geological environment, further supplemental soils investigations, if necessary, can be more objectively made. The tangible association to the conditions of exposure afforded thereby will form a more definite background upon which designs can be premised, and their subsequent performance studied and evaluated.

While soil maps based on the Pedological Classification are convenient and will facilitate such investigations, they are not absolutely essential. The lack of then will only mean that some additional work will need to be done in establishing both the geology of the soils, their constitution, and subsurface environment. This, together with the considerations of the meteorological influences of the area and an appreciation of moisture movements, will permit of an adaptation to areas where some of the background information provided by the soils maps has not previously been developed or established.

It is conceived, therefore, that the phase of highway engineering dealing with pavements and their foundations is an Art based on a knowledge of the conditions of the locale, an understanding of the phenomema of nature, and the recognition of the mechanics of the functional processes of pavements, rather than a strictly technical process which deals in the abstract with some of the several elements disassociated from each other.

In the foregoing it was perforce necessary to deal only with Wisconsin conditions, because of our unfamiliarity with conditions in other places. The combination of the several circumstances generates conditions in Wisconsin which are quite severe, and it is believed that through their severity some of the causes thereof became manifest, which made it possible to recognize them.

Through this recognition, judgement can be more effectively directed so as to provide designs which have so far given evidence of an assurance of longer lived pavements at diminished annual costs for upkeep and maintenance, which will tend to amortize the initial increased cost, and may even, in the long run, be conducive to paying actual dividends.

ACKNOWLEDGEMENTS

The investigation and study has been carried on intermittently through the past six years or so by the writer and John R. McGuire and John S. Piltz, Assistant Construction Engineers, and is still being followed through as time and opportunity permit. The writer and the two assistants named, in the investigation and study have drawn on their experience extending back over the past three decades of active participation in highway design and construction problems.

Valuable assistance was also extended by Dr. E. F. Bean, State Geologist, whose participation in some of the phases of the study aided in the resolution of some of the factors incident and contributing to the variable typical performance of pavements under otherwise seemingly comparable conditions.

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DISCUSSION

K. B. WOODS, *Purdue University* - Mr. Bleck is to be complimented for making a great many field observations and in associating these observations with the performance of highways in the State of Wisconsin. This method of approach has been used extensively by a number of highway departments and it is by this procedure that many of our present highway engineering practices have become established.

It has been said that the highway system of a state can constitute an excellent laboratory in which full-scale testing is employed. However, this method of analysis has certain limitations. In the first place, variables cannot be controlled and as a result, a half dozen or more influencing conditions must be evaluated without the benefit of controlled study where one variable is isolated and evaluated at a time as can be done in the laboratory. In the second place, the ability, experience, and background of the observer determines to a large extent the ultimate value of this method of analysis. In the first instance, individual variables can be, in effect, isolated and studied by obtaining a great mass of data and analyzing the information statistically. In the second instance, the responsibility placed upon the observer is great and the pooling of ideas of several observers covering one situation, as was done by Mr. Bleck, is at least a partial answer to this limitation.

In Mr. Bleck's presentation, it is unfortunate that he did not include rather extensive tabulations and charts of data with statistical analyses to verify the validity of his conclusions which apparently have been drawn from observations alone. Under the heading of "Pavement Performance," Mr. Bleck states that the service records of all the pavements "show that there is substantially no failure or disintegration attributable solely to the factor of the aggregates or the cements that were used." Later on he states that it is not necessary to go into greater detail in connection with these matters since they "have no great bearing on the performance and condition of the pavements." Since these statements are contradictory to conclusions which have been drawn by others[1] in connection with a similar type of survey, the data used in drawing these conclusions would have been an important addition to the paper. In addition, the Indiana work showed that a striking correlation existed between the type of coarse aggregate employed and the occurrence of blowups. Furthermore, it was established in this work that mapcracking (presumably Mr. Bleck's "D" cracking) was to be correlated generally with blowups.

Similarly, in Mr. Bleck's analysis of blowups he points out that this type of failure is to be correlated with the type of soil upon which the pavement is located. Although Mr. Bleck does not report detailed data on this point either, the result confirms the work of Sweet and Woods[2] who showed good correlation between mapcracking and blowups with the drainage characteristics of subgrade soils upon which the pavements, made from inferior aggregates, were constructed.

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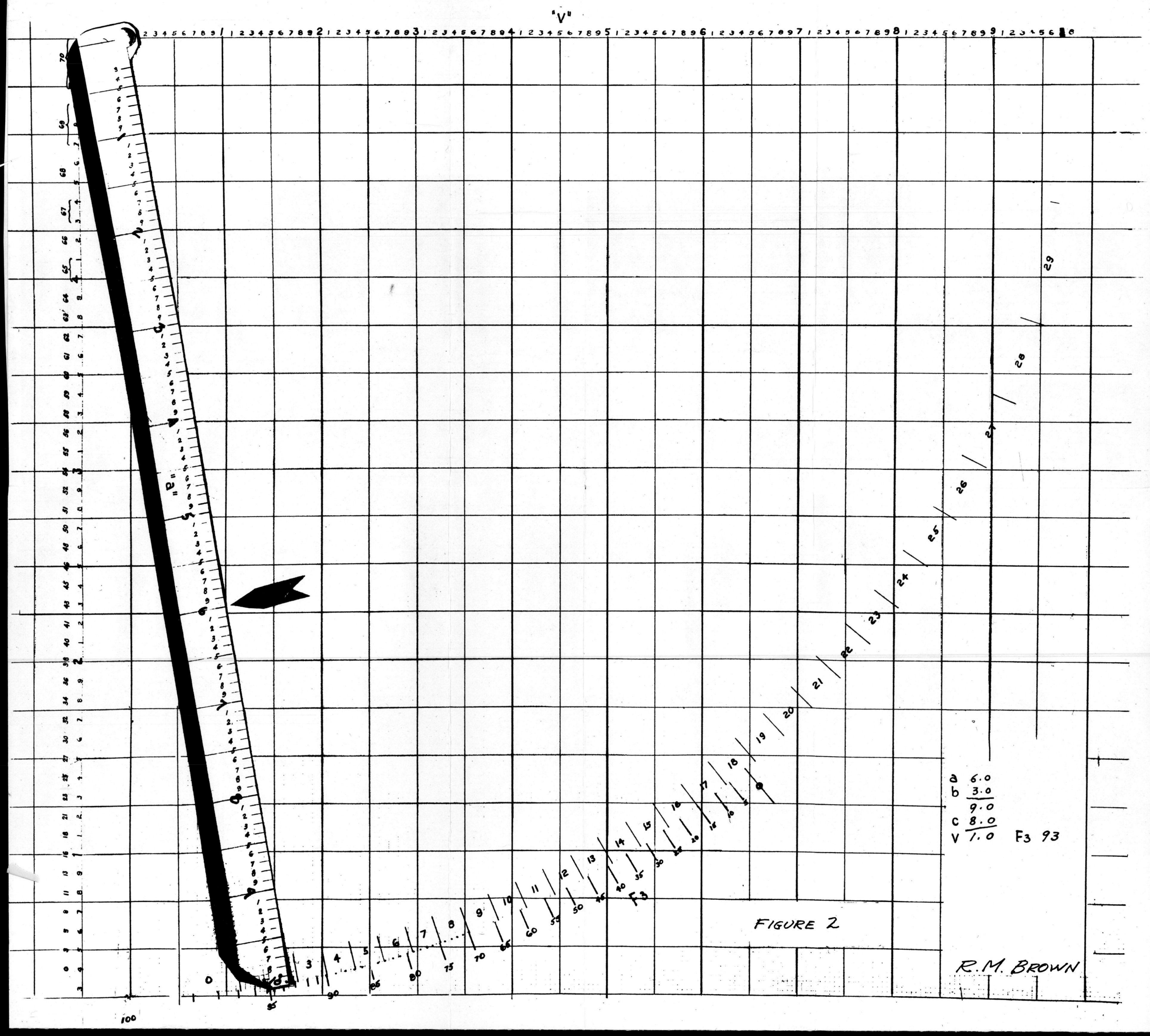

FIGURE 2

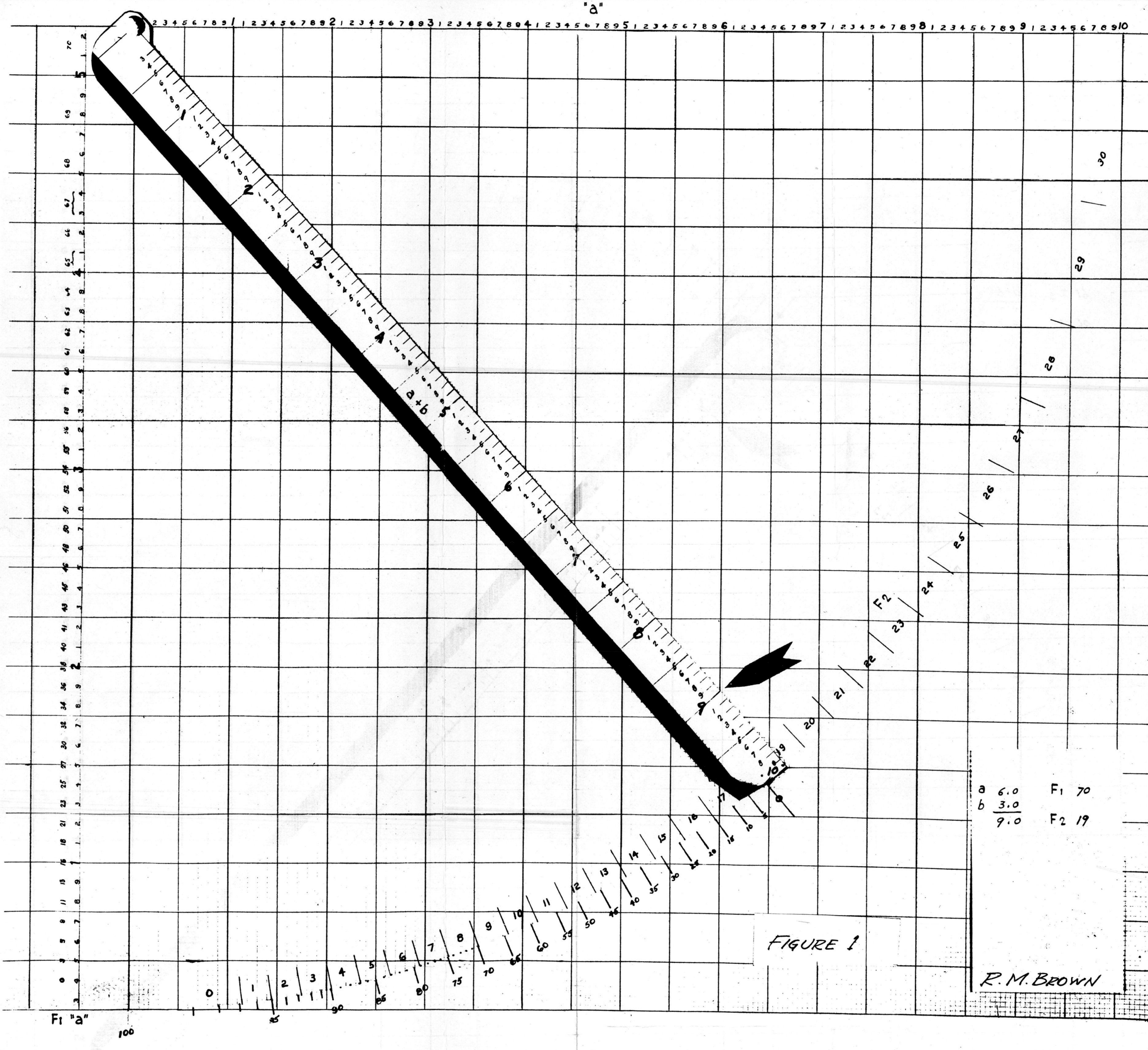

FIGURE 1

F1
a
v
C
b
F3
F2
No. of vehicles
vehicle usage
usage
STRING NO. 1
POSITION NO. 1
STRING NO. 1
POSITION NO. 2
STRING NO. 2
POSITION NO. 1
STRING NO. 2
POSITION NO. 2
R.M. BROWN
FIGURE 3

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