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Factors Related to Frost Action in Soils

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Subgrade Moisture Conditions Under an Experimental Pavement

JOHN W. GUINNEE, Senior Engineer
CHARLES E. THOMAS, Materials Engineer
Division of Materials, Missouri Highway Department

THIS paper deals with one phase of a program to obtain moisture histories of subgrades and specifically covers the first step of this phase which investigates the subgrade condition under an experimental pavement of portland-cement concrete. This step has two purposes: (1) to attempt to evaluate the effects of design variables on subgrade moisture histories and (2) subsequently to aid in the evaluation of the design variations as they affect the pavement performance. This first report encompasses only that part of the complete investigation which deals with the results obtained from the measurement of subgrade moisture content by means of electrical resistance moisture cells.

The experimental pavement includes variables of (1) coarse aggregate (limestone or chert gravel); (2) base treatment (rolled stone, dense design, or sand gravel, open design, or no base); and (3) subgrade treatment (oiled earth treatment or plain earth, no treatment). These variables were combined into 12 comparable sections. These sections vary from 0.2 mile to 0.6 mile in length with the total experimental project approximately 5 miles long.

The moisture cells, installations, instrumentations, and calibration procedures are described. Construction notes and validating field moisture checks are included in the appendixes along with topography notes and the log of the subgrade soil horizons.

Data obtained from 96 moisture cells have been averaged in various combinations to compare the effect of differences in cell location with respect to pavement cover, since the cover changes the degree of exposure to the possible means moisture entrance from above. These data have also been arranged to show differences caused by the variations in construction features and to show the variations in moisture content among the 12 comparable sections. The effects of the oiled-earth subgrade treatment are of especial interest, along with the commentary concerning possible undesirable results of poor construction of the oiled-earth treatment. The influence of drouth on subgrade-moisture conditions is apparent, and possible reasons for the variations in the effects of this influence are discussed. This report is in the nature of a progress report and therefore does not attempt to present final conclusions.

● THE purpose of this investigation is to obtain information concerning the subgrade moisture content and its change, or movement, with time under some typical sections of portland-cement-concrete pavement with construction variables of pavement, base, or subgrade.

The information is to be used as a supplement to condition surveys in an attempt to evaluate the performance of the design variables in the experimental sections. It will also be used as pilot information to determine the advisability and practicability of expanded studies to include other types of pavement structures and the variety of soil types normally encountered in Missouri subgrades.

The ultimate need for this information is to aid in the development of rational design methods for: pavement thickness, base, subgrade, and drainage construction.

LOCATION

As shown in Figure 1, the project is located in Warren and St. Charles counties on US 40. It stretches from approximately 1.6 miles west of the county line at Foristell to 3.4 miles east. It ends approximately 44 miles west of St. Louis. This area is in the gently rolling hills north of the Missouri River.

PROJECT DESCRIPTION

This project was so designed as to involve variables of pavement, base, and subgrade. The variable of pavement design was confined to differences in the coarse aggregate, half of the project using a limestone aggregate and the other half using a chert-gravel aggregate. The St. Louis formation limestone came from the St. Charles Quarry located near St. Charles, the chert gravel came from the Meramec River near Pacific, and the sand for both mixes came from the Mississippi River at St. Louis.

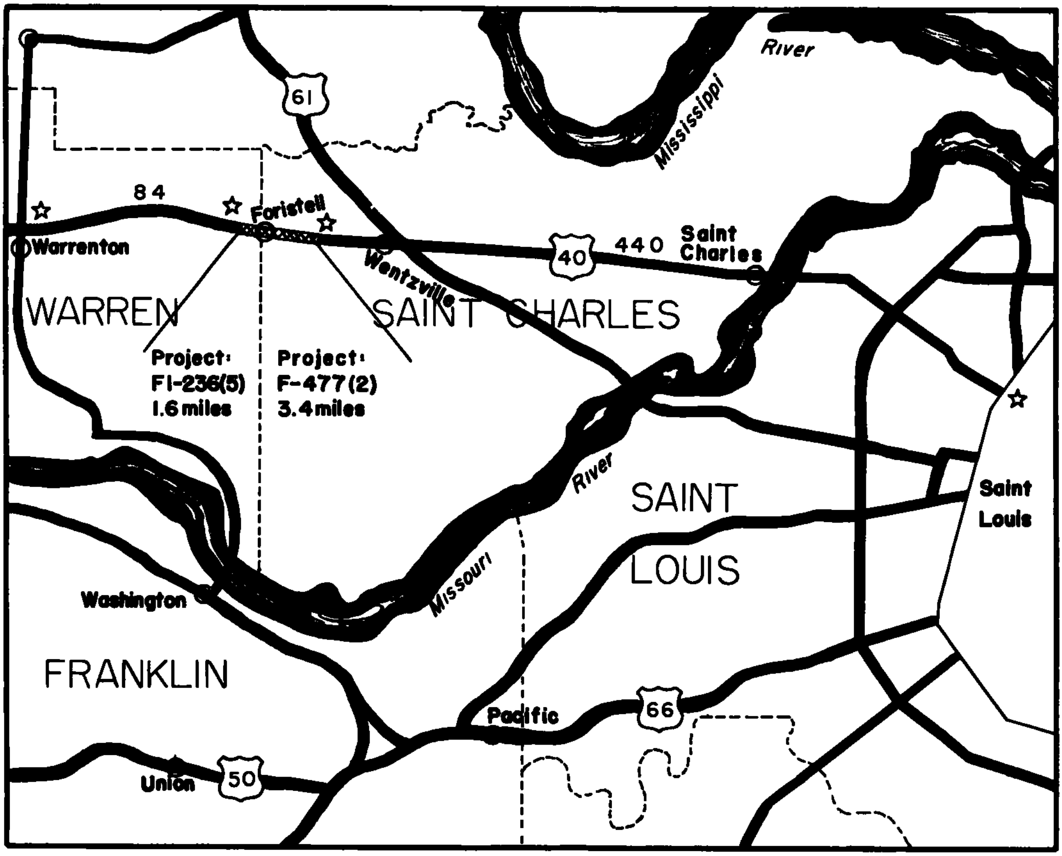


Figure 1. Location of experimental project.

The base variable consists of three types: rolled stone (dense design), sand gravel (permeable design), and no base. The rolled stone base material was from the Burlington formation from a quarry two miles east of Foristell and the sand gravel came from Peruque Creek approximately 1,000 feet upstream from the quarry.

The subgrade variable consists of two types: an oiled-earth treatment and plain earth (no treatment). The oil used was an SC-2 with a positive spot and was to be applied at the rate of 1.0 gal. per sq. yd. in three applications.

The variables of pavement, base, and subgrade were combined so that 12 comparable sections were established which range from 0.23 mile to 0.67 mile in length. Table 1 shows the combination of variables and this combination is also shown schematically in Figure 2.

This experiment is located on the new west-bound lanes of a separated four-lane highway, constructed utilizing parts of the old pavement for two of the lanes. It is 24 feet wide, having a uniform thickness of 8 inches and with a slope to the outside of ⅛ inch to 1 foot. The expansion joints were $\frac{3}{16}$ inch by 8 inches of premoulded filler with ⅞-by-16-inch smooth dowels, end capped and placed at 12-inch centers 4 inches deep in the concrete. The expansion joints were placed at 40-foot intervals, and a 49-lb. 6-by-12-inch wire fabric was used 2⅛ inches below the pavement surface.

TABLE 1

COMPARABLE EXPERIMENTAL SECTIONS

SECTION	AGGREGATE	BASE TREATMENT	SUBGRADE TREATMENT
1	Limestone	Rolled Stone Base	Oiled Earth
2	Chert	" " "	" "
3	Limestone	" " "	None
4	Chert	" " "	"
5	"	Sand Gravel Base	"
6	"	" " "	Oiled Earth
7	Limestone	" " "	" "
8	"	" " "	None
9	"	No Base	"
10	"	" "	Oiled Earth
11	Chert	" "	" "
12	"	" "	None

The rough grading on these sections was completed in the fall of 1947. Paving started on June 1, 1948, and was completed by August 7, 1948. Construction progress was delayed due to rain; therefore, some difficulties were encountered in the proper processing of this experiment. Notes concerning some of these difficulties with particular regard to the oiled sections are included in Appendix A.

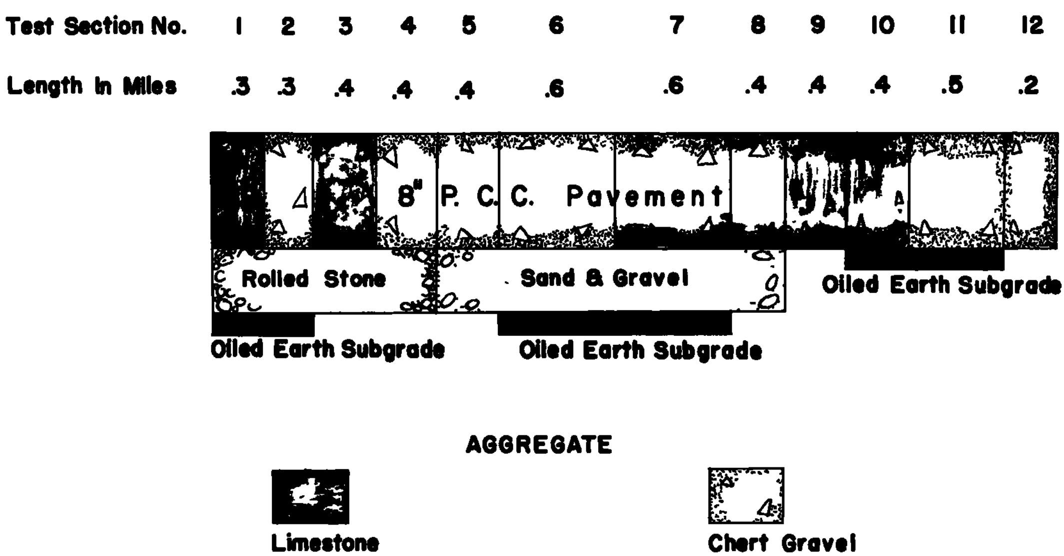


Figure 2. A 5-mile longitudinal section, showing construction variables.

The centerline cuts or fills vary generally up to 8 feet, although a few are up to 15 feet. A more-complete description of the topography of each selected location where this part of the investigation was conducted, is included in Appendix B.

The soil encountered throughout the range of this project is the Putnam Silt Loam.

The results of laboratory tests of typical samples of the various horizons are shown in Table 2.

TABLE 2

PUTNAM SILT LOAM CHARACTERISTICS

Horizon	A 1	A 2	B	C
% Retained No. 40	1.5	9.7	0.8	0.3
% Retained No. 200	3.4	12.2	4.6	1.0
% Sand 2.0 - 0.5 mm	7.5	16.5	6.8	3.2
% Silt	66.1	57.6	44.2	52.9
% Clay	26.4	25.9	49.0	43.9
% Colloids	7.0	7.5	25.0	20.0
Liquid Limit	28.2	26.4	61.2	46.2
Plastic Index	3.7	5.7	27.2	22.8
F. M. E.	26.8	23.3	44.6	29.9
Vol. Change at F.M.E.	7.3	3.5	63.2	28.4
Opt. Moisture	17.2	17.2	29.5	21.0
Max Dry Density	106.7	109.8	87.5	98.9
Classification	A4(8)	A4(8)	A7-5(19)	A7-6(14)

An attempt was made to locate these subgrade-moisture installations within subgrade limits which would be of the C horizon soil. The fill locations were of mixed horizons, due to grading operations; but nevertheless, the locations finally chosen were predominately C horizon soil. Appendix C contains a subgrade log of the entire project distance and a chart showing the results of subgrade moisture and density tests which were taken prior to paving operations.

INSTRUMENTATION

In an attempt to obtain a continuous record of subgrade-moisture conditions, moisture cells were installed at selected locations. These moisture cells are of the two-electrode, plaster-of-paris, electrical-resistance type as designed by Bouyoucos (1). The cells were manufactured according to specifications (1) by a Michigan Company (2).

They are rectangular cells, 2.4 by 1.3 by 0.5 inches with two internal electrodes 0.75 inch apart. The electrodes are the tinned ends of the No. 16 stranded, twin-lead wire. The length of the lead wire varies with the placement of the cell. The cells are illustrated in Diagram C of Figure 3.

The resistance of these cells varies inversely as the free moisture available to the cell. This available moisture maintains a certain relationship to the total moisture present in the soil; therefore, the cell resistance is an inverse measurement of the moisture content of the soil. The relationship between the free moisture and the total moisture content varies with the different soil types; therefore, it is necessary to calibrate the cell resistance against total moisture content for each soil encountered.

One of the bridges used to measure the cell resistance was furnished by the same Michigan Company as above. The bridge is described in great detail by Bouyoucos (3). Briefly, it is a Wheatstone bridge with an oscillator to overcome polarization. It has a series of capacitors with which to balance out the variable capacitance of field lead arrangements and connections. The null or balanced resistance point is determined by the use of earphones and a rheostat (log potentiometric). This bridge is rugged and portable, but difficulty has been experienced in obtaining readings with this earphone type due to extraneous and overshadowing noises occasioned by traffic. On roads of heavy traffic with many trucks this is, indeed, a problem.

The bridge finally selected for use under these conditions is an electric-eye type as manufactured by a New Jersey Company (4). This bridge is a self-contained alternating-current bridge using the cathode-ray eye as the visual indicator of the null point. Otherwise it has essentially the same arrangement as the bridge previously described. Although it does not have quite the capacity as the first bridge, the need for such a range has not become apparent in the field on this project. It was necessary to provide an eye shield for the set to keep the sunlight from interfering with the reading.

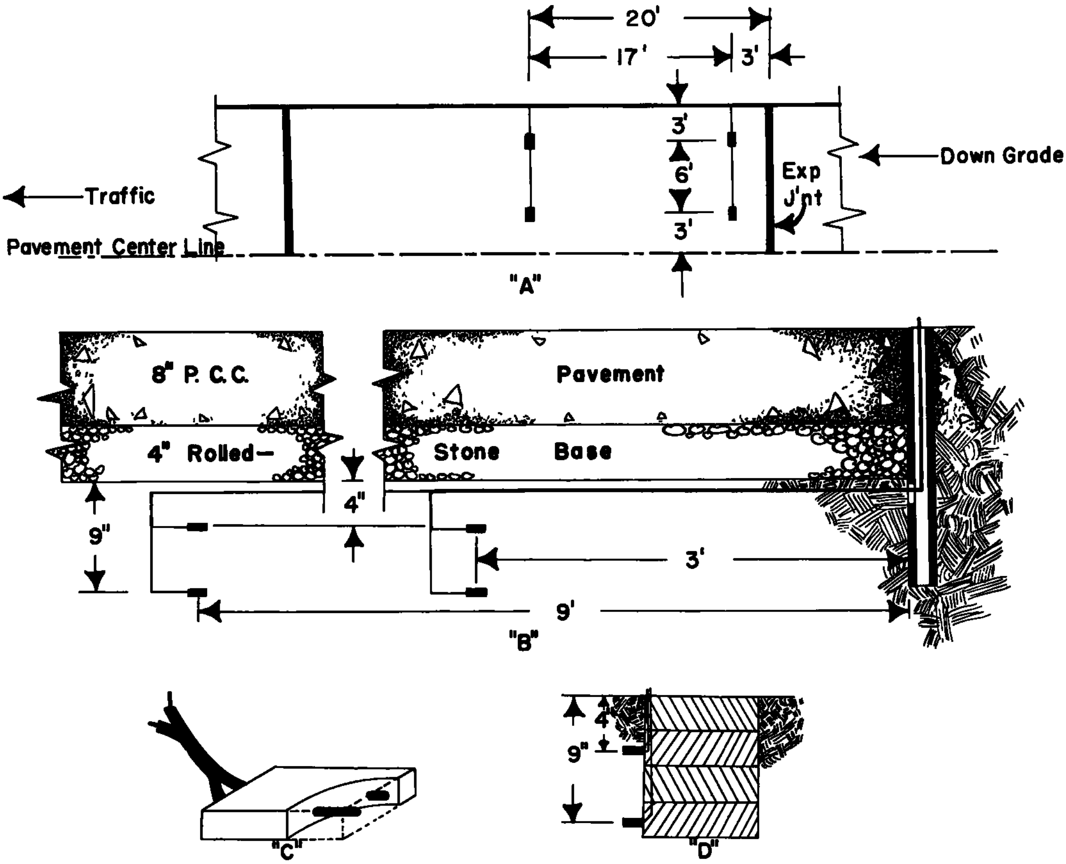


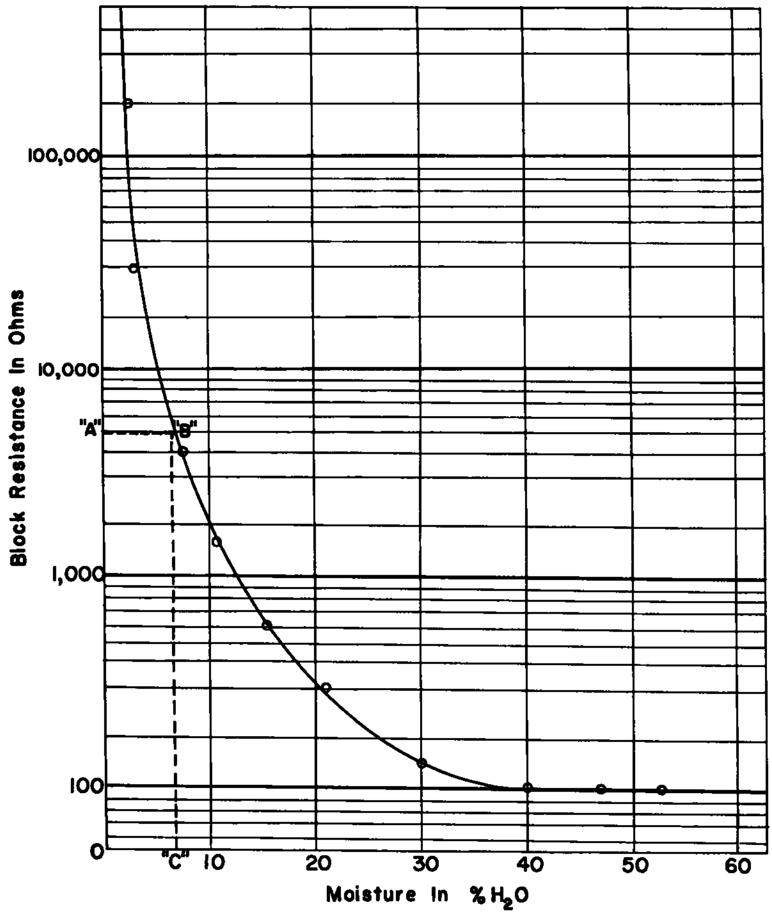
Figure 3. Installation details and cutaway view of a plaster cell.

Soil temperature is determined by a copper thermohm temperature soil, calibrated at 70 F. with a range from 0 to 130 F.

CALIBRATION

In order that the percent moisture based on the dry weight of the soil might be obtained, it is necessary that a calibration curve be drawn showing the relationship between cell resistance and actual percent moisture. Due to the fact that each soil type has a characteristic water-retention curve and that two soils exerting the same water-holding force will contain different percentages of total moisture, it is necessary to employ a calibration curve for each soil type to show the correct relationship.

The calibration procedure first tried was to place three cells in a gallon bucket filled with soil of a predetermined moisture content. Several buckets for each soil were prepared representing the range of moisture contents from almost complete dryness to complete saturation is a slurry-type mixture. These buckets were stored in a room having constant temperature and constant humidity for several days to insure equilibrium conditions. Resistance readings were also taken on these cells from time to time. When the cell had reached a constant-resistance reading, the readings were recorded and then the soil surrounding each cell was sampled for total moisture percent. From these data a calibration curve, as shown in Figure 4, was drawn with resistance on the log scale vertically and moisture on the normal scale horizontally.



Example Enter chart at "A" with corrected resistance of 4,900 ohms, proceed horizontally to "B" then vertically to "C" to obtain a moisture content of 6 8 % H_2O

Figure 4. Calibration curve for the plaster cells in Putnam silt loam.

Later, following a more-recent Bouyoucos article (5) on fabric cells, a different procedure was tried: placing a cell in a pan slightly larger than the cell, then adding enough soil to completely surround the cell and saturating it with an excess of water on the surface. These cell-pan setups were then weighed and resistance readings taken as they dried out. Knowing the total tare weight without water, the dry weight of soil used, and then subtracting to find the total weight of water used, the total percent water could be figured to correspond with the resistance reading taken at that time. These later readings were compared with the first and checked fairly well, although it is believed that this method should not be used alone due to lag in the plaster cells.

In the field, the first method was found to give the closest agreement with the actual conditions, so the curve (Figure 4) thus obtained was used. Tables were drawn up from this curve to provide a quick and consistent means of interpretation. The validating test data for the installed cells are presented in Appendix D.

INSTALLATIONS

The cells were installed in specific relation to the transverse and longitudinal joints at the selected locations. Four installation holes were dug, one at each of the following positions; 3 feet from the transverse joint and 3 feet from the edge; 3 feet from the transverse joint and 9 feet from the edge; 20 feet from the transverse joint and 3 feet from the edge; and 20 feet from the transverse joint and 9 feet from the edge. At all locations the cells are located downgrade from the transverse joint. Figure 3 (A) illustrated the installation. The 96 cells were installed eight to a section downgrade from the joint locations as shown in Table 3.

As shown in Figure 3 (B), the cells were placed at 4-inch and 9-inch depths in the subgrade. They were placed in the sides of holes, Figure 3 (D), which had been dug to obtain density tests at the 4-inch and 9-inch depths. A wedge of just slightly larger dimensions than the cell was jacked into the side of the hole at the 9-inch depth. It was then withdrawn and the cell placed in the resulting cavity. The cell was securely wedged in and any excess space was tamped full with the soil that had been removed from that hole. The lead wires were then carefully drawn tight and the hole filled, with the soil which had been removed, to above the cell level.

The same procedure was then used in placing the 4-inch-depth cell and the hole was backfilled with the original soil and tamped to as nearly its original condition as was possible. The leads were placed in a 2-inch trench in the subgrade and led to the edge where they were buried in a 2-inch-diameter pipe 18 inches long just under the form line, if the forms had not yet been set. If the forms were already down, the pipes were buried so that they would extend part way under the slab when it was poured.

After the slab had been poured, the forms removed and some shoulder soil pulled back against the slab to fill the space left by the forms, the pipes were set upright. The pipes were dug out and the leads cleaned. The pipes were driven into the soil until the top of the pipe was 2 inches below the top of the slab. Concrete was then poured around the pipes sloping off from the top of the pipe and also bonding to and under the slab. A removable plug was screwed into the end of the pipe.

DATA

Soil-temperature data were obtained each time the cell resistances were measured so that corrections could be made. Temperature correction must be made when the soil temperature varies from the 70 F. calibration temperature. Correction curves (Figure 5) taken from Bouyoucos were used. Temperature higher than 70 F. gives a resistance lower than the 70 F. resistance, and temperature lower than the 70 F. gives a resistance higher than the 70 F. resistance. Corrections are slight around 70 F. and increase as the temperature is lowered. The correction for temperatures above 70 F. levels off close to 90 F. and remains almost constant.

The nearest weather station to this project is at Warrenton, Missouri, about 10 miles from the center of the experimental section, which is approximately 5 miles long. The rainfall data used are from this weather station. While perhaps not completely accurate because of possible variations in rainfall over that length of project, these data represent the best available. The Weather Bureau of U.S. Department of Commerce, with W. T. Zimmerman the observer, furnished the data. The rainfall data have been plotted against days of the month and shown in relation to cell moisture variations and data averages.

The cell-resistance readings were taken by one observer in the field at intervals of a week to a month. During the first 2 years of this investigation, the readings were taken at least once a week if at all possible; thereafter the interval was lengthened on occasions, as it was not considered too important to maintain the short interval. At the start of the experiment the field observer merely recorded the resistance and temperature readings and sent them to the main office, where the corrections for temperature and the translation (from calibration curves) to percent moisture were computed. Later

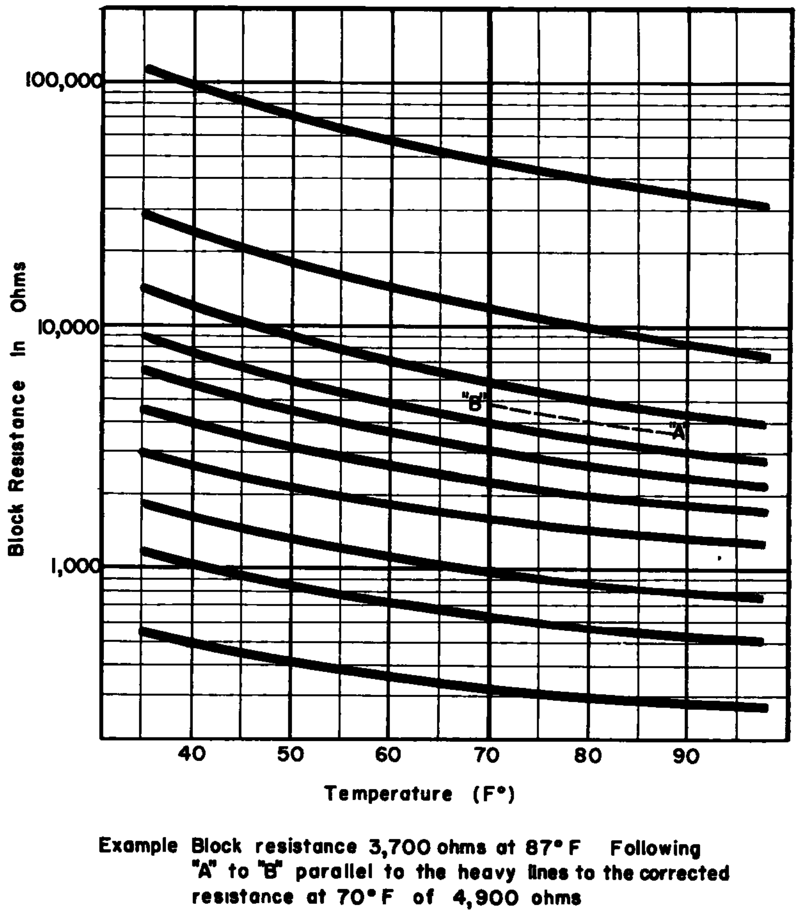


Figure 5. Temperature correction chart for plaster cells (After Bouyoucos).

the field observer was supplied with temperature-correction-and-calibration charts, and thereafter he submitted the data in a completed, averaged form.

In order to facilitate these measurements, a small delivery-type panel truck was equipped with the electric-eye bridge and with multiple-lead cables so that the readings could be taken from within the truck away from the strong light. This truck was also equipped for use on another project and had multiple-lead cables and connectors able to establish 80 connections at one time.

ANALYSIS

The data obtained have been averaged in many different ways for study and analysis. Since this phase of the investigation deals with subgrade moisture conditions and since there is a particular interest in the effects of the oiled earth subgrade, the data have not been separated to show the difference, if any, in the effects of the coarse aggregate.

Graphs have been constructed to show, through the use of moisture averages, the effect of construction or location variables. Each graph, in addition to the climatological data, contains four lines, representing the 4-inch and 9-inch depths in a plain earth subgrade and the 4-inch and 9-inch depths in an oiled-earth subgrade.

TABLE 3

CELL LOCATIONS

County	Joint Station	Section
Warren	842 + 00	1
"	857 + 00	2
"	865 + 03	3
"	883 + 03	4
"	911 + 44	5
St. Charles	38 + 05	6
"	70 + 05	7
"	77 + 25	8
"	107 + 25	9
"	124 + 05	10
"	143 + 65	11
"	174 + 85	12

These graphs have been arranged in sets, each set showing the results of averaging all the data from the 96 moisture cells in such a way as to point up the difference in some particular set of variables, i.e., location, base, etc. Set A contains Figures 6 and 7; Set B contains Figures 8 and 9; Set C contains Figures 10, 11, 12, and 13; and Set D contains Figures 14, 15, 16, and 17. It must be remembered that the same data are used in each of the four sets, but they are averaged and compared in a different way.

The data also have been set up to show the difference in moisture variations between all of the 12 sections.

Set A contains those graphs which compare the effects of distance from the edge. Figure 6 shows the comparison between the plain-earth sections and the oiled-earth sections at a distance of 3 feet from the pavement edge, while Figure 7 shows the comparison between the plain-earth subgrade and the oiled-earth at a distance of 9 feet from the pavement edge.

Set B contains those graphs which compare the effects of distance from the transverse joint. Figure 8 shows the comparison between the plain-earth sections and the oiled-earth subgrade at a distance of 3 feet from the transverse joint. Figure 9 shows the comparison between the plain-earth sections and the oiled-earth subgrade at a distance of 20 feet from the transverse joint.

Set C contains those graphs which compare the effects of the base variables. Figure 10 shows the comparison between two sections of rolled stone base on oiled earth. During the construction of the oiled-earth subgrade on experimental Section 1, difficulties (see Appendix A) were encountered which led observers to believe that this section should be regarded with suspicion. Later results tended to bear out this view; so this section has been left out of all averages but is shown here to compare with the rolled-stone base on oiled-earth section, in which it appeared that a fairly good oiled-earth mulch had been obtained.

Figure 11 shows the comparison between the rolled-stone base on plain-earth sections and the rolled-stone base on oiled-earth sections. Figure 12 shows the comparison between the sand-gravel base on plain-earth sections and the sand-gravel base on oiled-earth sections. Figure 13 shows the comparison between the no-base on plain-earth sections and the no-base on oiled-earth sections.

Set D contains those graphs which compare the effects of both the distance from the edge and the distance from the transverse joint. Figure 14 shows the comparison between those cells placed 3 feet from the edge and 3 feet from the transverse joint on the plain-earth and on the oiled-earth subgrades. Figure 15 shows the comparison between those cells placed 9 feet from the edge and 20 feet from the transverse joint. Figure 16 shows the comparison between those cells placed 9 feet from the edge and 3 feet from the transverse joint. Figure 17 shows the comparison between the cells placed 3 feet from the edge and 20 feet from the transverse joint.

The entire state of Missouri experienced a drouth starting in 1952 and extending into 1954. Naturally, the severity varied with the locality. While the area in which this experiment was constructed was not one of the hardest hit, it still was quite apparent, as shown by Table 4.

The effects of the drouth on the subgrade moisture content are shown in each of the Figures 6 to 17 by the decrease in the average moisture content of the wetter locations, generally starting about the middle of 1952. The few exceptions to this general decline are apparent among the graphs which follow. A discussion of these deviations is made later in the text.

TABLE 4
DROUTH DATA

Year	Total Rainfall in Inches	Deviation from Average
1948	41.41	+2.62
1949	44.42	+5.63
1950	33.54	-5.25
1951	42.80	+4.01
1952	29.34	-9.45
1953	23.98	-14.81
1954[1]	11.41[1]	-4.29[1]

[1]Through May 31st.

The comparison of Figure 6 with Figure 7 shows the effect of edge exposure. The distance from the edge evidently has the effect of decreasing variation in moisture content. That is, when the subgrade nearer the center of the pavement becomes wet, it is retarded in its ability to give up that moisture due to the cover above it. While the subgrade close to the edge may get wetter, it also dries more quickly; therefore, the average moisture content remains somewhat even. These tendencies are observed in the case of the plain-earth subgrade, but the oiled-earth sections, which represent an effort to construct moisture barriers, show plainly the effect of the pavement protection and the subsequent reduction of protection at the edge.

In 1952 the oiled sections showed practically the same moisture content as the plain-earth sections. Figure 7 shows that the effect of pavement protection was gradually

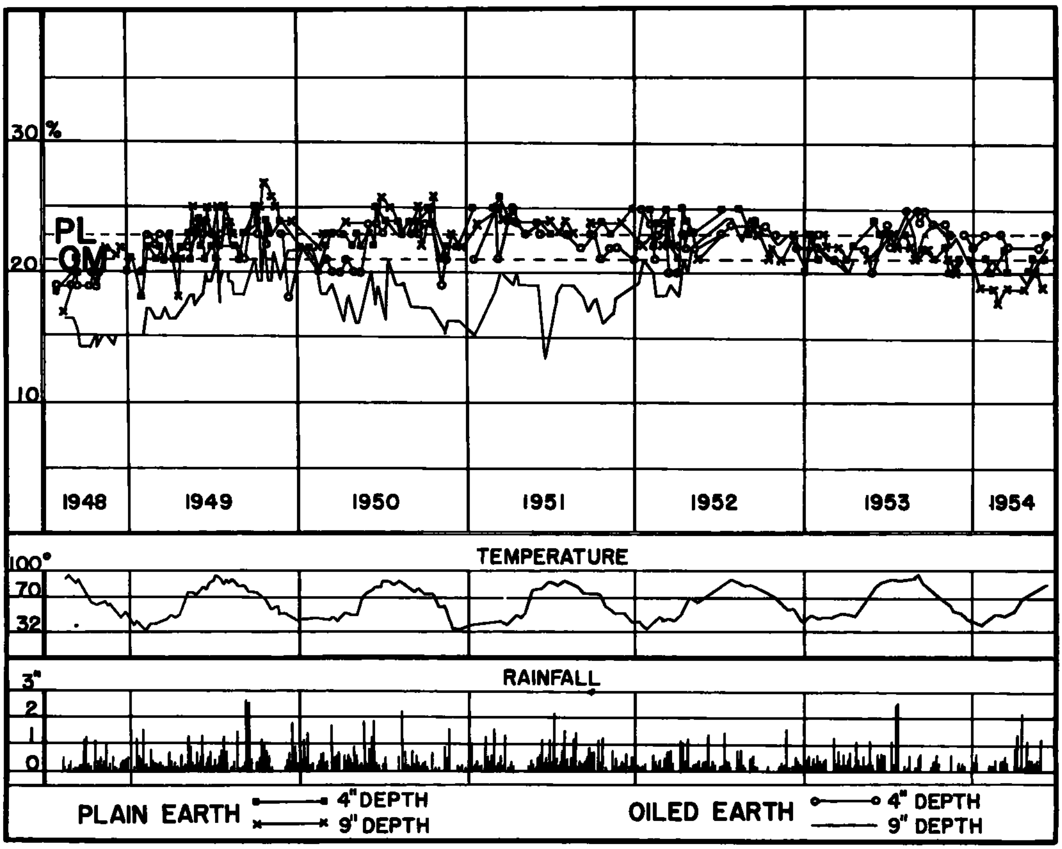


Figure 6. Comparison of the moisture averages of all cells 3 feet from the pavement edge.

being overshadowed, perhaps by the interference with the normal evaporative conditions, the accumulation of moisture under the oil barrier, or again by any number of other contributory conditions. The oiled sections then felt the drouth in 1952 and again showed the effects of edge exposure. An interesting point to note is that the oiled sections did not start their decline until sometime after the plain-earth sections had started decreasing.

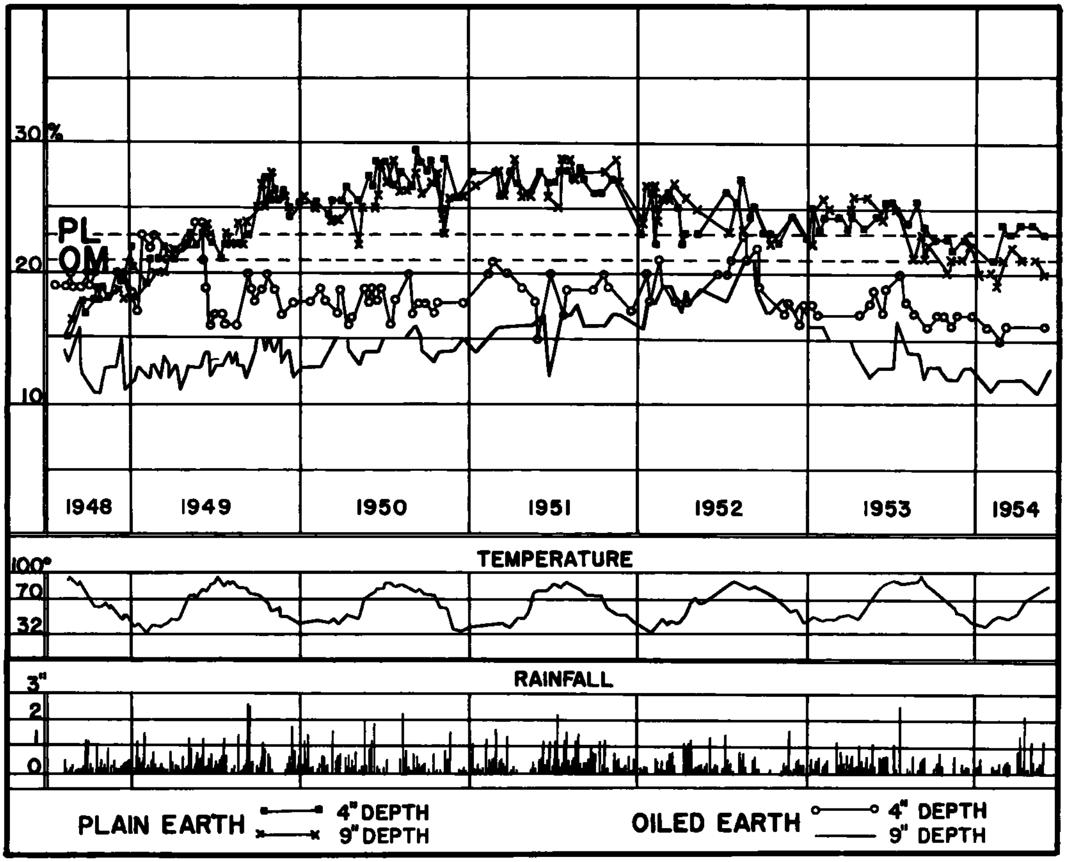


Figure 7. Comparison of the moisture averages of all cells 9 feet from the pavement edge.

The comparison of Figure 8 and Figure 9 were made in an attempt to show the influence of the exposure from the transverse joints. The lack of any significantly great difference between these graphs indicates that the effect of the edge exposure greatly overshadows the effect of the joint exposure. In other words, water can and does enter more easily (and in greater volume) from the edge than from the joint.

The results of all of the cells are used in this averaging so half of the cells 3 feet from the edge are averaged in the group 3 feet from the joint. The reverse was true in Figure 6, where half of the cells 3 feet from the joint are averaged in the group 3 feet from the edge. This fact points up the overshadowing effect of the edge exposure. In all probability the importance of edge-versus-joint exposure is relative to time and maintenance procedure. Efficient and timely crack and joint maintenance will serve to preserve the difference in moisture level with the locations nearer the edge staying the wetter.

Figures 10, 11, 12, and 13 were drawn to show subgrade moisture difference under different types of base treatment. Figure 10 shows the differences in subgrade moisture under two sections of rolled-stone base on oiled-earth subgrade, one of which is considered bad because of a poorly worked subgrade which did not attain the expected results under construction. The oiled subgrade was badly rutted and the seal was broken in such a way that the subgrade had a checkerboard appearance. Since this was purely an experimental project, the best method of subgrade construction was not known ahead of construction. As a result, the desired subgrade treatment was not obtained on this section. Con-

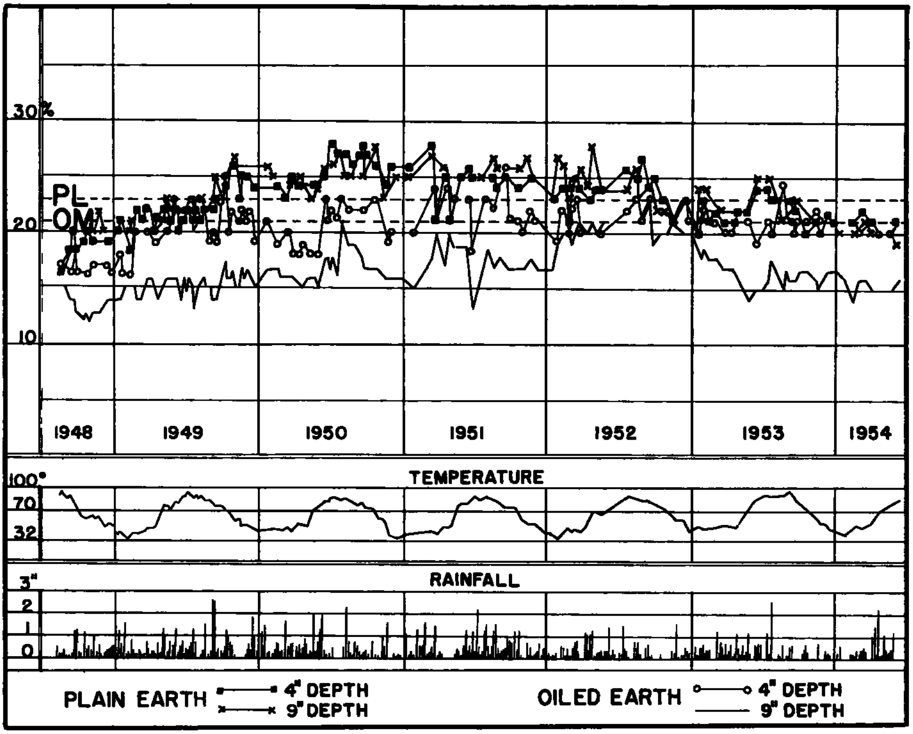


Figure 8. Comparison of the moisture averages of all cells 4 feet from the transverse joint.

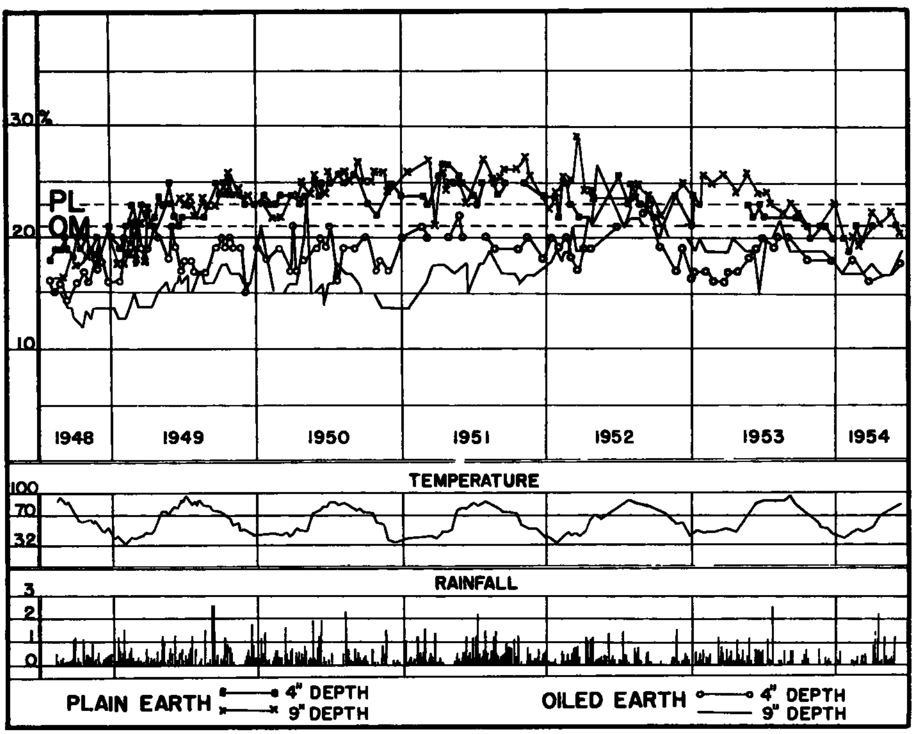


Figure 9. Comparison of the moisture averages of all cells 20 feet from the transverse joint.

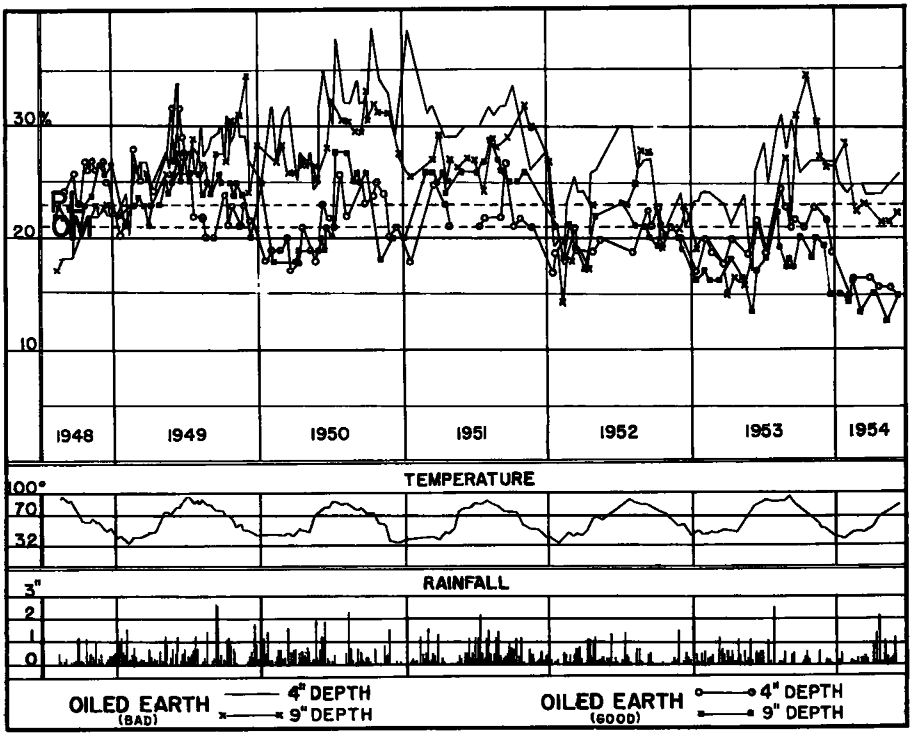


Figure 10. Comparison of the moisture averages of all cells under rolled stone base which had been placed on oiled earth subgrade.

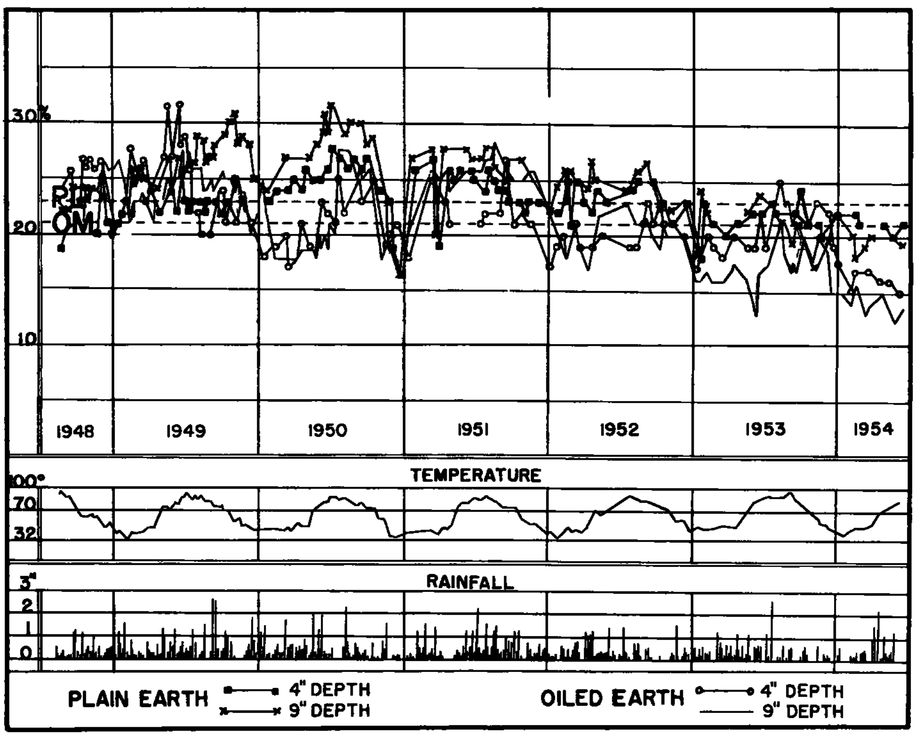


Figure 11. Comparison of the moisture averages of all cells under rolled stone base.

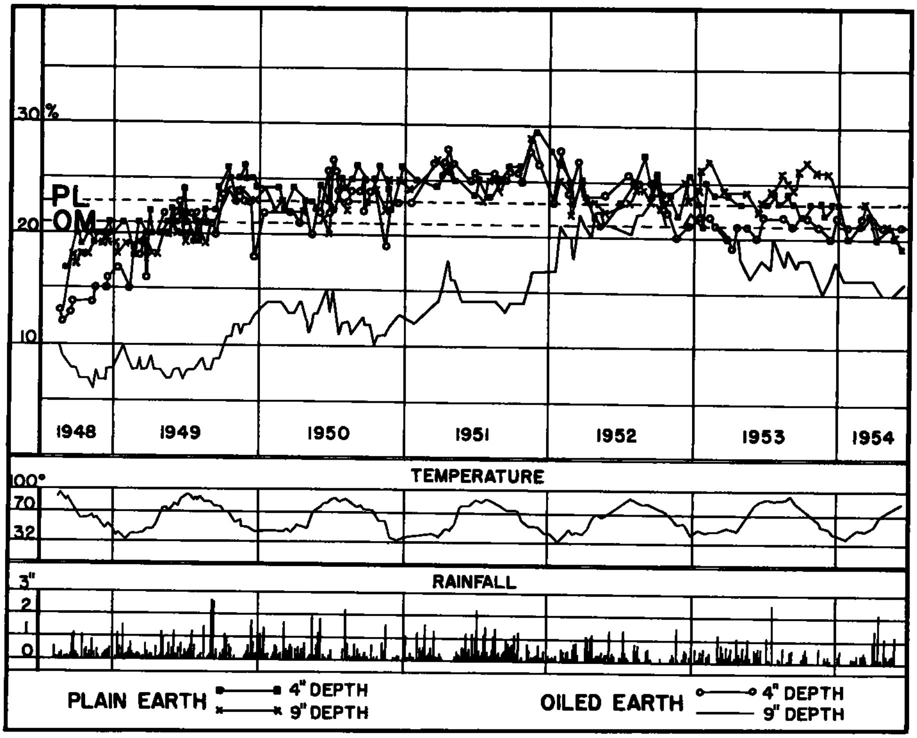


Figure 12. Comparison of the moisture averages of all cells under sand-gravel base.

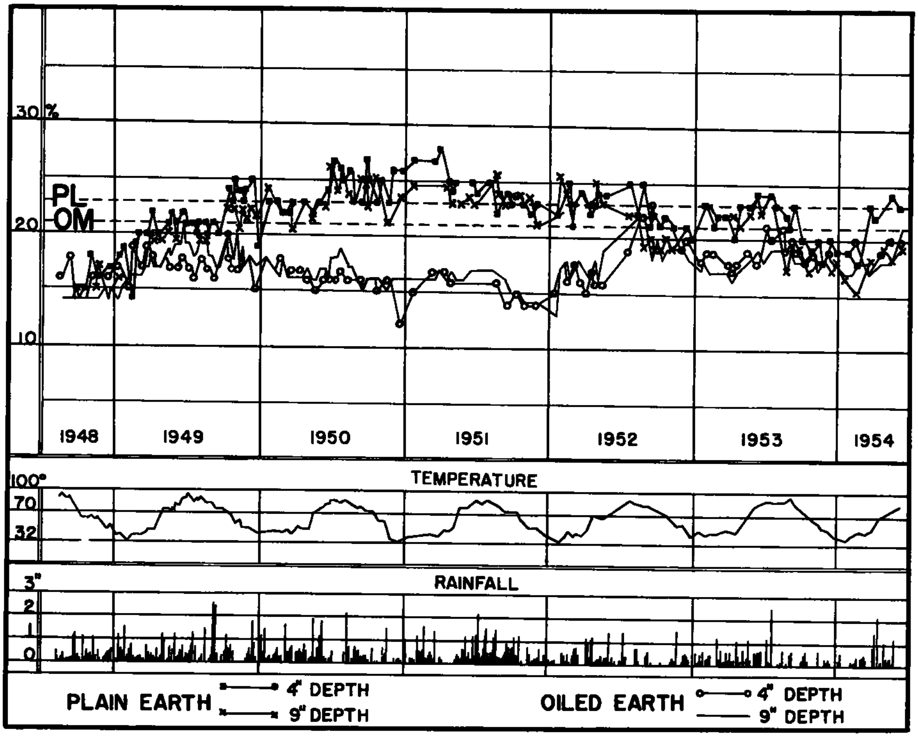


Figure 13. Comparison of the moisture averages of all cells under no base.

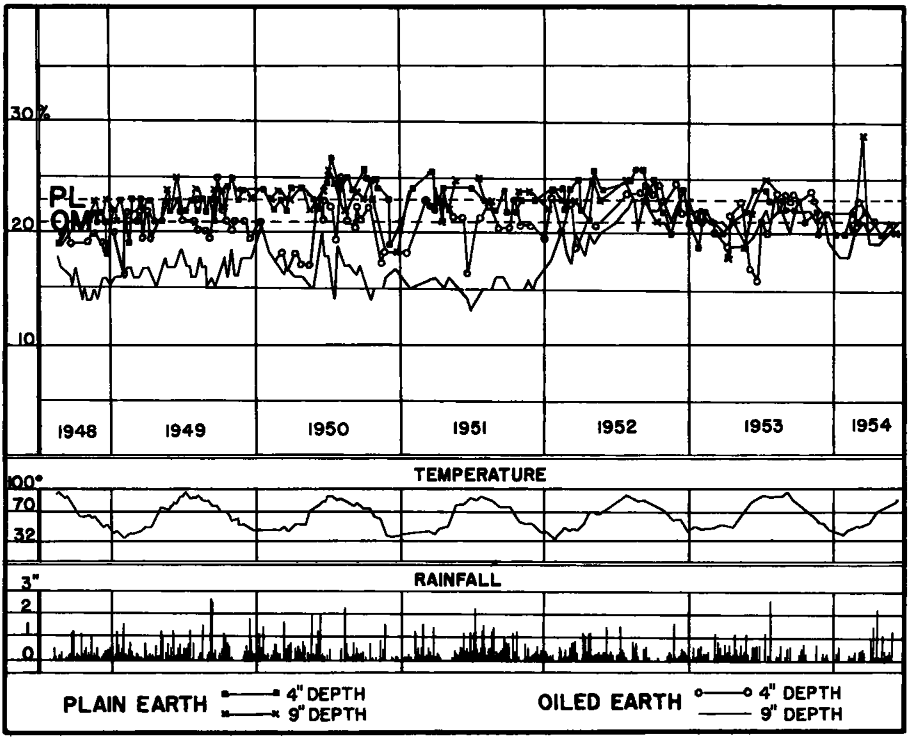


Figure 14. Comparison of the moisture averages of all cells 3 feet from the edge and 3 feet from the joint.

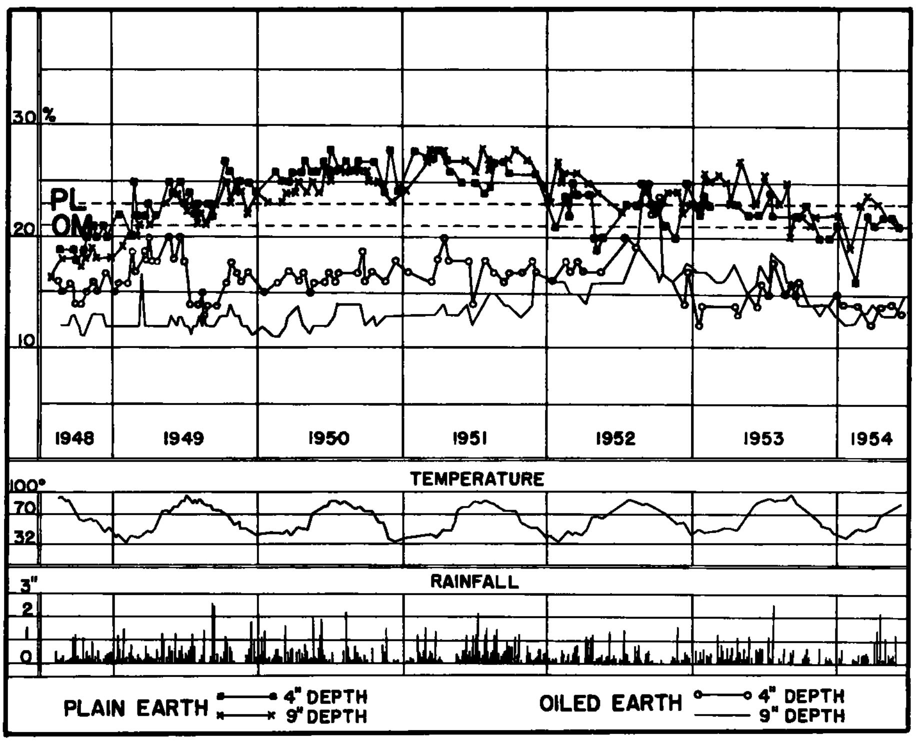


Figure 15. Compariosn of the moisture averages of all cells 9 feet from the edge and 20 feet from the joint.

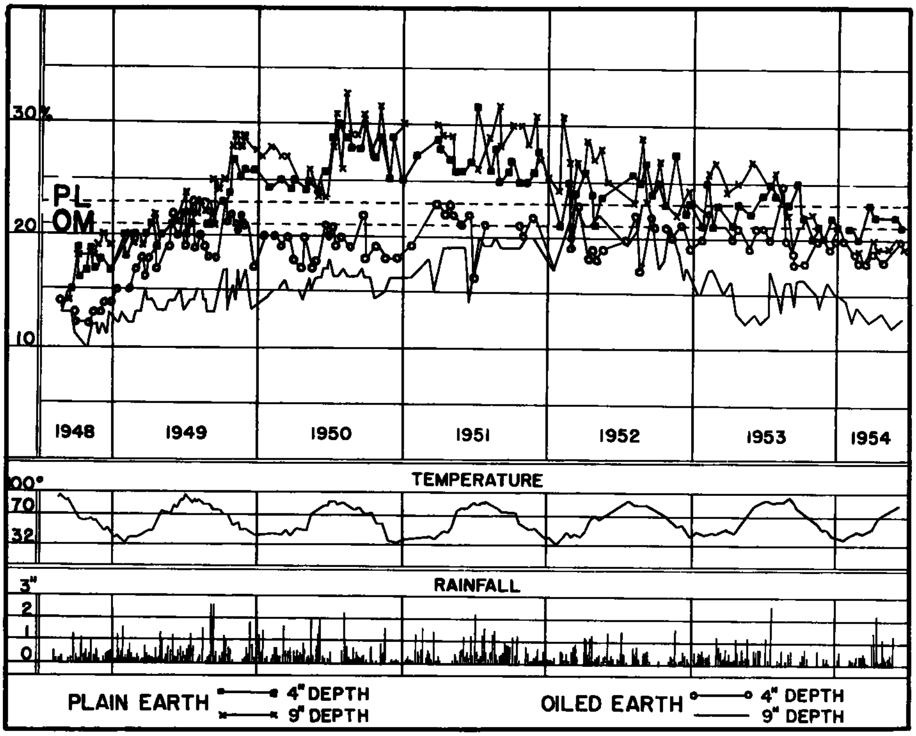


Figure 16. Comparison of the moisture averages of all cells 9 feet from the edge and 3 feet from the joint.

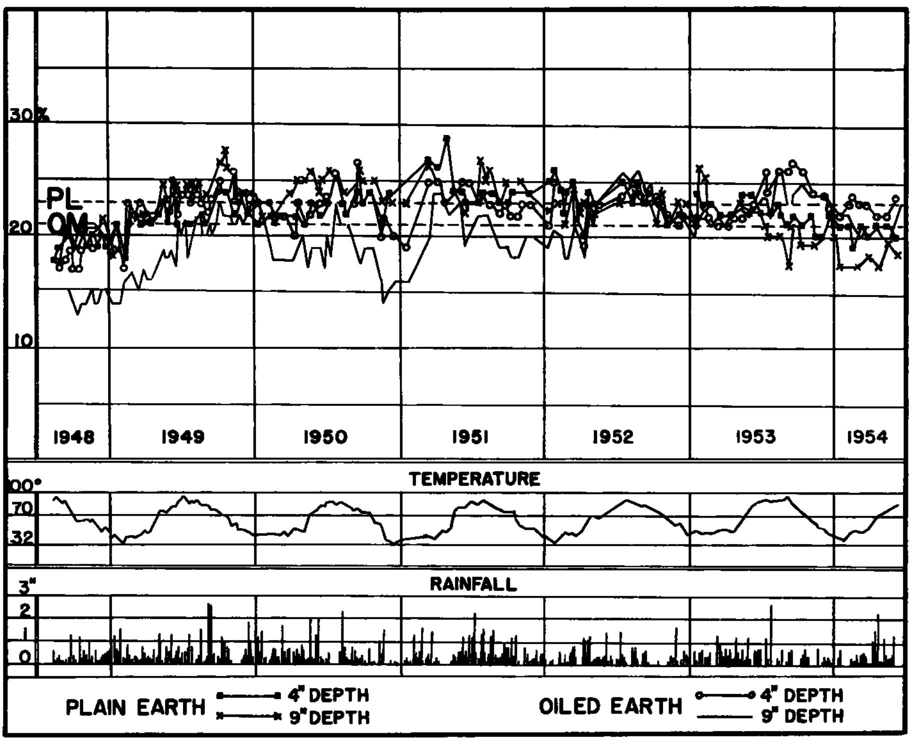


Figure 17. Comparison of the moisture averages of all cells 3 feet from the edge and 20 feet from the joint.

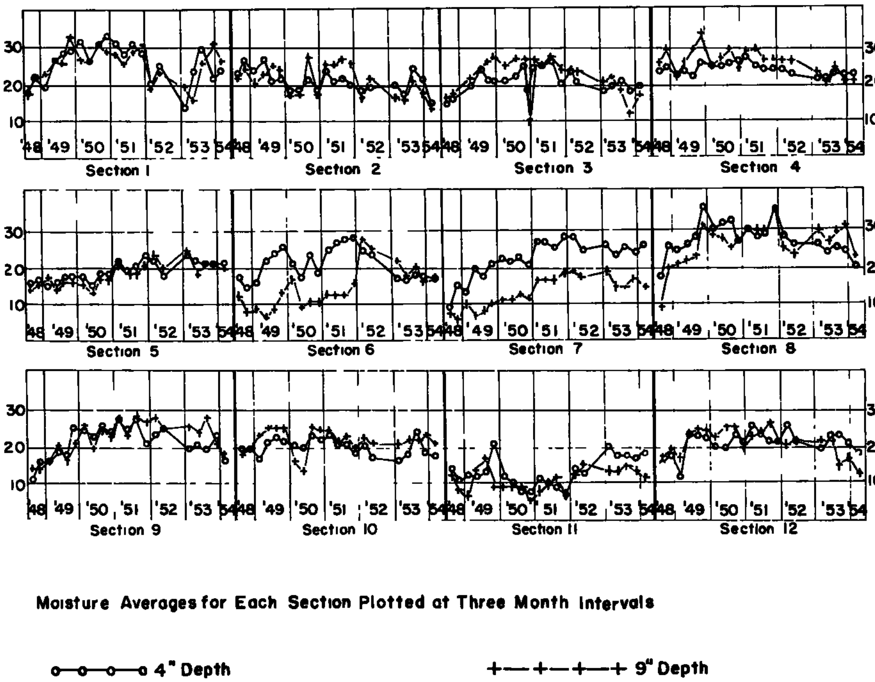


Figure 18. Comparison of the moisture averages of each comparable section.

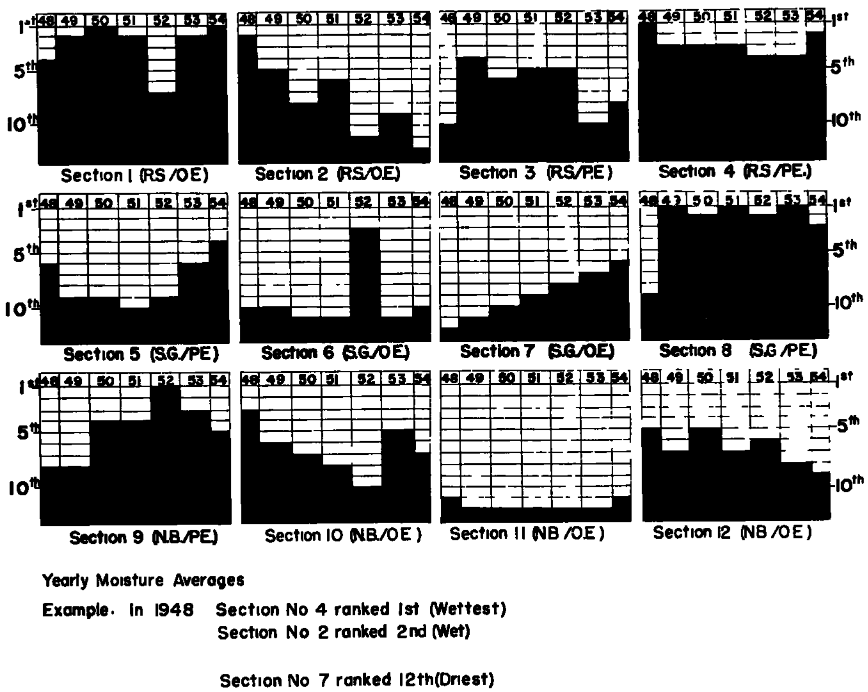


Figure 19. Relative wetness of each section.

struction notes covering these difficulties are contained in Appendix A.

In considering Figure 10, it is apparent that a certain type of oiled-earth construction will greatly contribute to the continuing support of a high subgrade-moisture level. The drouth did not have such an appreciable effect upon the subgrade moisture content of this section (No. 1) as it did on other sections.

The comparison of Figures 11, 12, and 13 raises many questions which will only be answerable on this project in years to come, after the performance of the various sections under traffic has been evaluated. The true benefits of this study will be realized as the evaluation is then made. Such data will serve as reference points for these comparisons. Most performance investigations up to this time have not contained sufficient subgrade-moisture history, and this fact has probably been ignored in the evaluations.

As an example, the question is raised as to the benefit (as far as subgrade moisture is concerned) of a base course of either type, for the graphs show the no-base sections to be relatively drier than the base sections, this being especially true for the oiled sections (Nos. 10 and 11). The conjecture is also put forth as to how long this condition might last. This question may be answerable in the future but not at this time.

If subgrade moisture were the criterion by which the base courses were to be judged, (considering rigid pavement only), the first choice as the best base treatment would have to be no-base on oiled-earth subgrade. Next would be sand-gravel base on oiled earth, then no-base on plain-earth subgrade, and last would be the rolled-stone base on a poorly constructed (bad) oiled-earth subgrade. It is the authors' opinion, however, that the last-mentioned section should be viewed as unsatisfactory and unintentional design, and the data should be used and considered only in this light.

It seems to be indicated that the average moisture contents of all the sections were coming to a common level at the early part of 1952 (at the start of the drouth). Again a question is posed as to what might have happened had not the drouth set in.

One must not jump to the conclusion that all of the sections would have then remained at the same general moisture level and that because of this lack of differentiation that there was no benefit from the base material or construction.

The oiled-earth sections, under each of the base types, average drier than the plain-earth sections for practically the complete period of time under consideration. This is especially true of the no-base sections.

It is interesting to note the spread in moisture contents which was almost constantly maintained between the 4-inch depth and the 9-inch depth in the oiled-earth subgrade under the sand-gravel base. Apparently when the checking action of the oiled earth is present, the greater permeability of the sand-gravel base does not allow the penetration of moisture into the subgrade that the less-permeable rolled-stone base permits. This is probably a question of the available-time element of moisture entrance. It has been observed that rolled-stong bases, constructed of relatively soft stone, have a tendency toward a type of wicking action which might loosely hold and transfer water over a period of time.

Figures 14, 15, 16, and 17 were drawn to show the differences in average moisture contents at the various locations with respect to the edge and joint. As has been expected the cell with the greatest exposure (Figure 14) maintained the wettest average, while the cells with the least exposure (Figure 15) maintained the driest average. The drier average on the part of the least-exposed cells is mainly due to the consistent dryness of the cells in the oiled-earth subgrade. The erratic moisture condition of the cells 9 feet from the edge and 3 feet from the joint and the spread between the plain-earth averages and the oiled-earth averages of these cells is indicative of the exposure from the joint and its relation to the exposure from the edge. Figures 16 and 17 indicate, as noted before, the effect of the construction of a barrier to moisture entrance from above when the barrier has the additional aid of less exposure. These graphs serve to emphasize the greater severity of edge exposure.

Figure 18 was constructed to show the moisture average of each section at 3-month periods. These periods correspond to the subgrade sampling dates of another investigational project. The averages plotted are the averages for particular days at 3-month intervals and not the average over the 3-month periods.

Figure 19 was constructed to show the relative wetness of any section by means of yearly averages. Figures 18 and 19 taken together show the relative wetness and also give the moisture level of the various sections. In evaluating Figure 19, the more massive the black area for a section the higher was the relative wetness of that section. The first-four sections have rolled-stone base, the second-four have sand-gravel base, and the last-four have no base.

As can easily be seen, the first-four are the wettest and the last-four are a little drier than the sand-gravel. With the exception of Section 1 (the poorly constructed, oiled subgrade) the oiled sections are drier than the plain-earth sections under the different base types.

The gathering of soil-moisture data by means of cells is, of necessity, being concluded on this project, because the moisture cells have reached or are about to reach the limit of their expected life due to the deterioration of the leads and the insulation around them. It is not considered feasible to attempt to replace the blocks and leads, so subsequent check on subgrade-moisture conditions on this project will be by manual sampling.

Final conclusions as to which construction variation is the most efficient will only be reached in the future by the evaluation of the respective service records. To date the condition surveys have shown no appreciable differences which can be considered as indicative of pavement performance. This, report, therefore, should be considered as one of progress only. At the present time it should serve as a guide and stepping stone to further experimentation along these lines, especially in terms of studying the effectiveness and efficiency of moisture-barrier design.

The variations of some of the oiled sections in not reflecting the drouth as noticeably as other sections has led to conjectures as to the causes. One reason put forth is that the barrier of oiled earth has a reluctance to permit the release of moisture through surface evaporation, due to its combination with the wicking type base and high humidity. On the other hand, the pavement cover serves to maintain the high humidity level occasioned by even sparse rainfall.

CONCLUSIONS

The following conclusions seem to be justified by the data presented in this report.

1. The edge of the pavement is the main entrance of moisture into the subgrade (not the only).
2. The oiled earth serves to at least slow down the entrance of moisture into the subgrade.
3. Oiled-earth membrane or treated layer, if poorly constructed, can be detrimental as far as subgrade moisture is concerned.
4. There appears to be some benefit from the oiled subgrade in damping the fluctuations of the subgrade moisture content due to the climatic conditions (drouth, extremely wet seasons, etc.).

ACKNOWLEDGMENTS

The authors wish to acknowledge the valuable guidance and encouragement which F. V. Reagel, engineer of materials, and W. C. Davis, chief, Geology and Soils Section, gave during the compilation of this report. Thanks also go to M. S. Lattimore, materials engineer, for his considerate efforts in obtaining the field data and to all the laboratory technicians for their tedious work herein involved.

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2. Wood and Metal Products Company, Bloomfield Hills, Mich.
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Appendix A: Notes on Materials & Construction

ROLLED STONE BASE

Material for rolled stone base was produced by the Tobin Quarries Co. from a ledge of Burlington limestone, located two miles southeast of Foristell, in the SE ¼ Sec. 29., T-47-N, R 1-E, St. Charles County.

The stone was produced under the following specification, which was set forth in the Special Provisions.

Passing	1 inch sq.	opening sieve	- 100 percent
"	No. 4	"	- 40 - 60 percent
"	No. 40	"	- 15 - 35 percent

A 5 percent tolerance was allowed between the 1-inch and 1½-inch sq. sieves. The minus No. 40 fraction shall have a liquid limit not greater than 25 and a plastic index not greater than 6.

The results of tests on samples taken during construction shows that the rolled stone base material conformed to the gradation requirements. The average of all tests being as follows; 100 percent passing the 1-inch square opening sieve, 53.1 percent passing the No. 4 sieve and 21.6 percent passing the No. 40 sieve.

The base was constructed in two layers; the bottom layer approximately 3-inches thick followed by a topping layer approximately 1-inch thick.

Compaction was obtained with pneumatic-tired rollers. A flat-wheel roller was used in conjunction with final finishing immediately ahead of pavement construction.

The dry density of the completed base averaged 126.5 lb. per cu. ft. and the moisture content averaged 3.8 percent. Based on a maximum dry weight per cu. ft. of 144.3 lb., which was determined by Lab. Test No. 48-1161, the obtained average weight of 126.5 lb. per cu. ft. is only 87.6 percent of the standard.

SAND-GRAVEL BASE

The sand-gravel base material was furnished by the Tobin Quarries Co., from a deposit located in Peruque Creek, 2 miles southeast of Foristell and it was produced subject to the following gradation specification.

Passing	1½-inch round screen	------------100 percent
"	½-inch " "	------------50-8- percent
"	⅛-inch sieve	------------30-55 percent
"	No. 20 "	------------15-35 percent
"	No. 100 "	------------0-10 percent

The fraction passing the No. 40 sieve shall be nonplastic.

According to the test results on samples taken during construction, all of the sand-gravel base material met the specified requirements for gradation. The maximum, minimum and average of all tests being as follows:

		Maximum	Minimum	Average
Passing	1½-inch screen	-	-	100 percent
"	½-inch "	75 %	59 %	68 "
"	⅛-inch sieve	53 %	42 %	47 "
"	No. 20 "	35 %	22 %	27 "
"	No. 100 "	7 %	4 %	5 "

The base was constructed in a single 4-inch layer and compaction was obtained with pneumatic-tire rollers. A 5-ton flat-wheel roller was used in conjunction with final finishing to produce a compact uniform surface immediately ahead of pavement constructions.

A maximum dry weight of 126.1 lb. per cu. ft. and an optimum moisture content of 8.4 percent was determined for this material by laboratory tests. It was found impracticable to make accurate density determinations of the base during construction, however, the moisture content was maintained slightly above the optimum and rolling was continued until the base was firm and there was no appreciable movement under the pneumatic-tired rollers.

It is the opinion of the observer that the density of 126.1 lb. per cu. ft. was obtained, however, when paving was commenced the batch trucks ravelled and dislodged the base to such an extent that it was necessary to operate the paver on the road shoulder.

OILED SUBGRADE

The experimental variation included several sections of oiled earth subgrade and the special provisions specified that this treatment shall consist of 1-gal. of SC-2 liquid asphaltic material per sq. yd., applied in 3 applications. This type of treatment was used under rolled stone base, sand-gravel base and where no aggregate base was specified.

Between stations 827+69 and 861+20 the subgrade was scarified, using a motor patrol with scarifier attached. The subgrade was quite dry at the time and as a result the loosened material contained many hard soil clods, some of which were 4-inches and even larger in diameter. Efforts were made to pulverize the oversize clods by disking and harrowing, but as this proved unsuccessful the loosened material was recompacted, as much as possible, by pneumatic-tire rolling and SC-2 oil was applied at the rate of 0.92 gal. per sq. yd. It was hoped that the coating of oil would tend to soften the clods and satisfactory compaction could then be obtained by subsequent rolling. However, very little softening of the clods occurred and the results obtained by rolling did not produce the uniform, dense surface desired.

Experience on this section would seem to indicate that scarifying is not a satisfactory method of preparing the subgrade for oil treatment.

Between stations 10+00 and 75+31 a motor patrol grader was used to form a windrow of soil material along the center of the road. This was done by thinly shaving the subgrade until the quantity in windrow was sufficient to produce a loose soil mulch, approximately ¾-inch thick when spread over the full width of subgrade.

Soil clods in the windrowed material were then further reduced by disking and harrowing, following which the material was respread uniformly over the full width of subgrade.

In spite of the care used in these operations the soil mulch after spreading generally contained from 10 to 20 percent of soil clods between 1 and 2-inches in diameter. Some of the difficulty experienced in efforts to obtain a greater degree of pulverization are attributed to the rains which occurred frequently while this work was being done.

On a part of this section, between stations 25+00 and 40+00, a total of 1 gal. of SC-2 oil was applied in 3 applications, at the following rates; first application 0.52 gal., second application 0.26 gal., and third application 0.22 gal. per sq. yd. On the balance of the section SC-2 oil was applied in 2 applications, the first application averaging 0.70 gal. and the second averaging 0.35 gal. per sq. yd., making an average total of 1.05 gal. per sq. yd.

In most instances both applications were made on the same day and each application was followed almost immediately by rolling with both flat-wheel and pneumatic-tired rollers. It might seem noteworthy, in this regard, that "pick-up" by the rollers was practically negligible.

Although the securing of more definite information as to the practicability and efficiency of this method of procedure was clouded by abnormally wet weather conditions, it appears that better results were obtained than on the previously described section on which the initial operation involved subgrade scarifying.

Between stations 120+00 and 164+61 the pavement was constructed directly on the oiled subgrade and a somewhat different procedure was followed in preparing the subgrade for oil treatment.

The subgrade was pulverized to a depth of approximately 1-inch by operating the subgrade machine on the pavement forms, which had been previously set to grade. This produced a well graded soil mulch with all of the material passing a 1-inch screen.

Oiling operations were interrupted several times by heavy rains. An idea of the delays due to intermittent rains can be had when it is stated that the first application was made o June 14 and the final application on July 2. The rains caused the mulched material to flow together and set and as a result it was necessary to re-loosen the subgrade with a spike-tooth harrow, over a considerable portion of the section before oil was applied.

Oil was applied in 3 applications at the following average rates; first application 0.60 gal. per sq. yd., second application 0.25 gal. per sq. yd., and the third application 0.25 gal. per sq. yd.

Rolling with pneumatic-tired and flat-wheeled rollers was done almost immediately following each oil application.

Several measurements which were made after the final rolling showed the oiled mulch thickness to be about ¾-inch.

Appendix B: Topography Maps and Notes

A description of the position of each set of eight blocks in relation to the surrounding topography is as follows:

Station 842+00 and 842+17 Test Section No. 1. Limestone aggregate, rolled stone base on oiled earth.

Centerline cut 4 ft.; 3 ft. special ditches on both sides; blocks placed just west of top o 1200 ft. VC from a +1.38 percent grade; eastbound roadway about 4 ft. higher. Surrounding ground is generally higher and is gently rolling, drainage to east and west.

Station 857+03 and 857+20 Test Section No. 2. Chert gravel aggregate, rolled stone base on oiled earth.

Centerline cut 2 ft.; special V ditch to south and standard roadway ditch to north; block placed on -0.25 percent grade; eastbound roadway 1 ft. to 2 ft. higher. Surrounding grou gentle to north and east, but slightly rolling to south and west.

Station 865+06 and 865+23 Test Section No. 3. Limestone aggregate, rolled stone base on plain earth.

Centerline cut 5½ ft.; standard roadway ditch to north and standard parkway ditch to south. Blocks placed on-0.25 percent grade; eastbound roadway 6 ft. higher. Surroundin ground gentle to north and west but slightly rolling to rolling in south and east.

Station 883+06 and 883+23 Test Section No. 4. Chert gravel aggregate, rolled stone base on plain earth.

Centerline cut 3½ ft.; standard roadway ditch to north and standard parkway ditch to south; blocks placed at top of 600 ft. VC going from -0.25 percent to -1.15 percent grade; eastbound roadway 4 ft. higher. Surrounding ground gentle to north and west, slightly rolling to south and east.

Station 911+24.1 and 911+41.1 Test Section No. 5. Chert gravel aggregate, sand-gravel base on plain earth.

Centerline fill 2 ft.; sidehill cut and fill. Special ditch both sides. One inch sand and gravel extended from base sand and gravel through shoulder 11 ft. from pavement edge on a slope of ½ inch per ft. Extends from Station 902+62 to 912+50. Underdrain across road at 911:50. Blocks placed at about original ground level on a +2.282 percent grade. Eastbound roadway 3 ft. lower. Rolling ground to north and west; slightly rolling to south and gentle to east.

Station 37+85 and 38+02 Test Section No. 6. Chert gravel aggregate, sand-gravel base on oiled earth.

Centerline cut 6 ft. standard roadway ditch to north and standard parkway ditch to sout

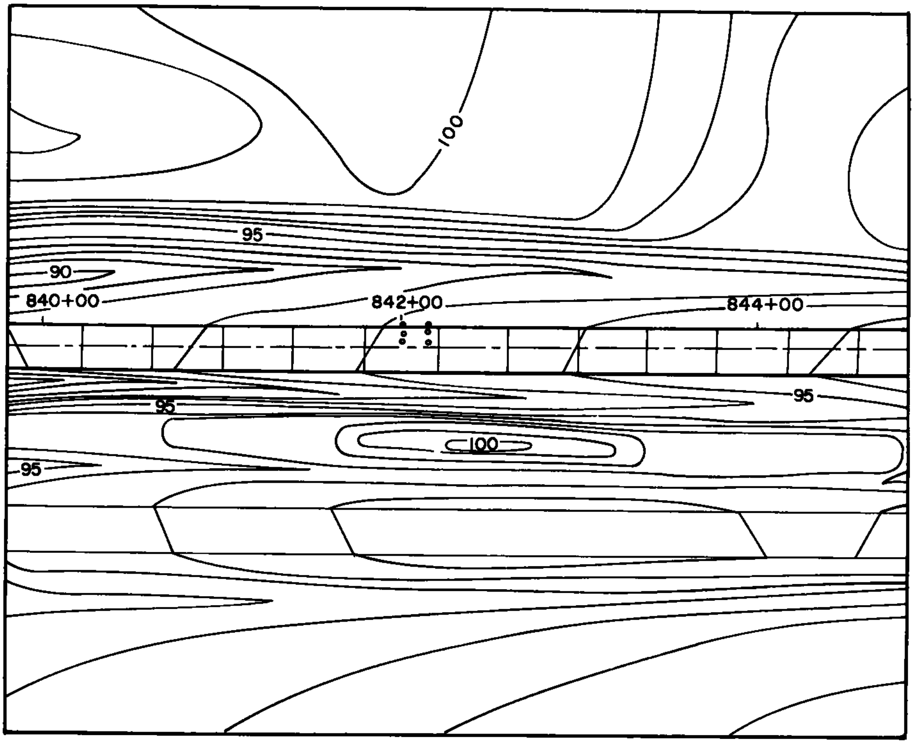


Figure A. A 1-foot-contour map showing relative elevations at cell locations in experimental Section 1.

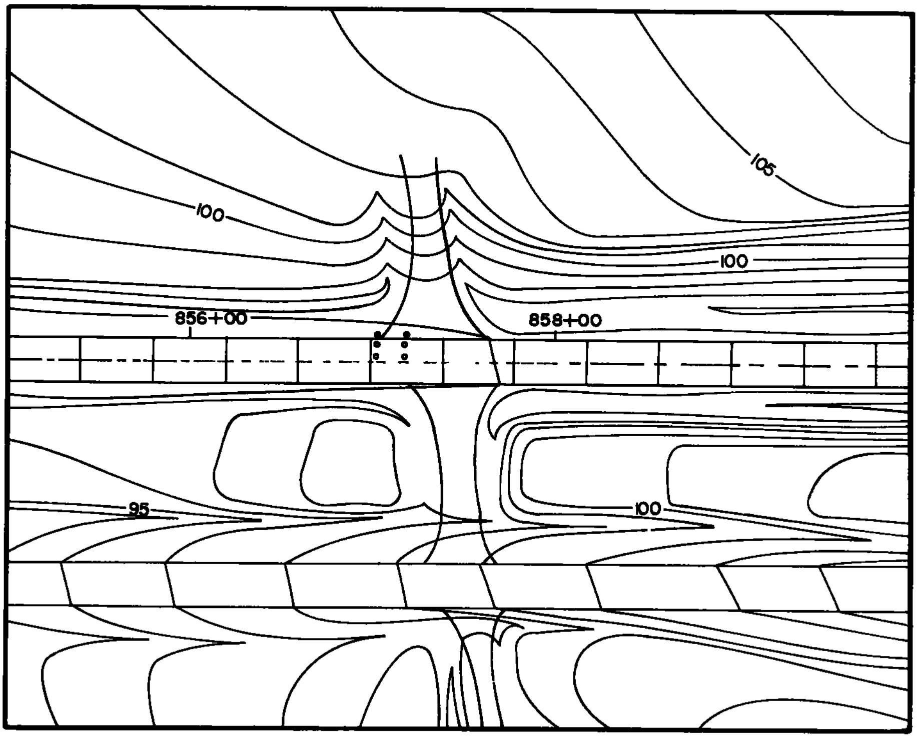


Figure B. Section 2.

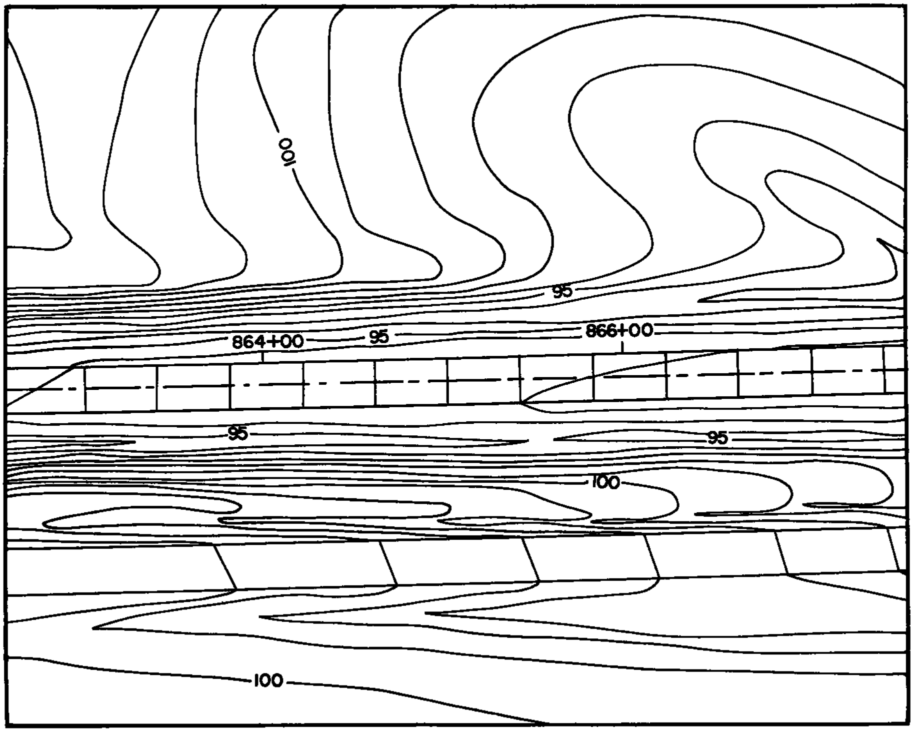


Figure C. Section 3.

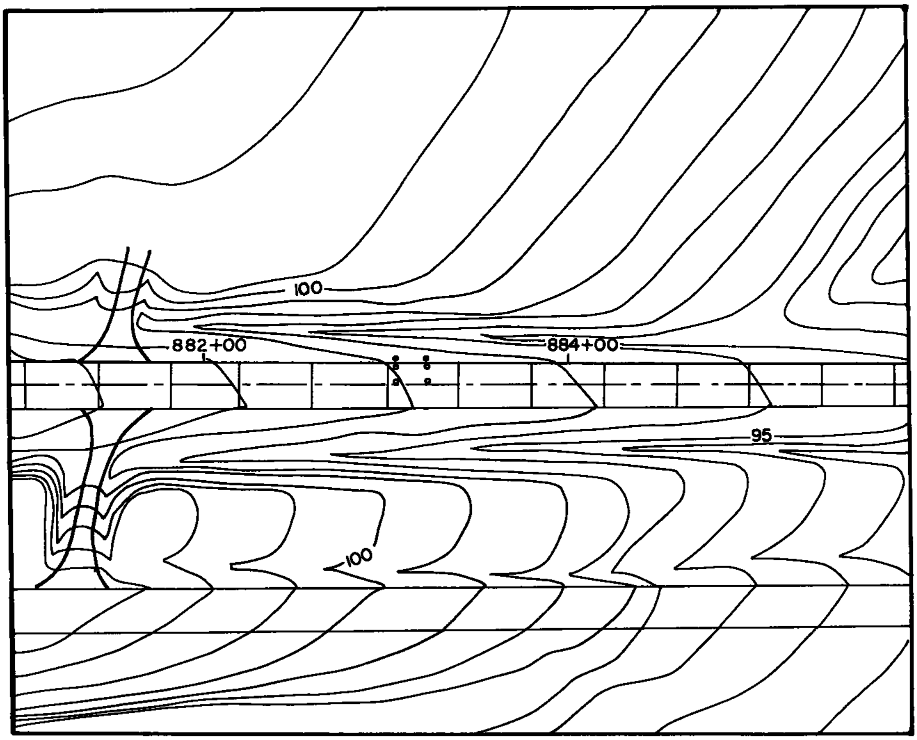


Figure D. Section 4.

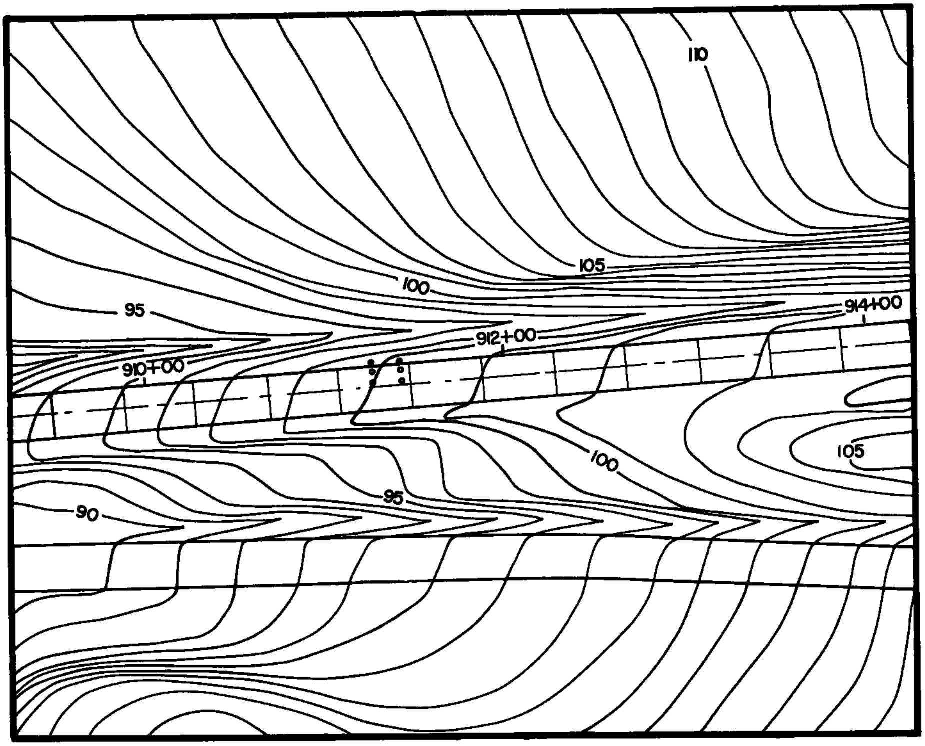


Figure E. Section 5.

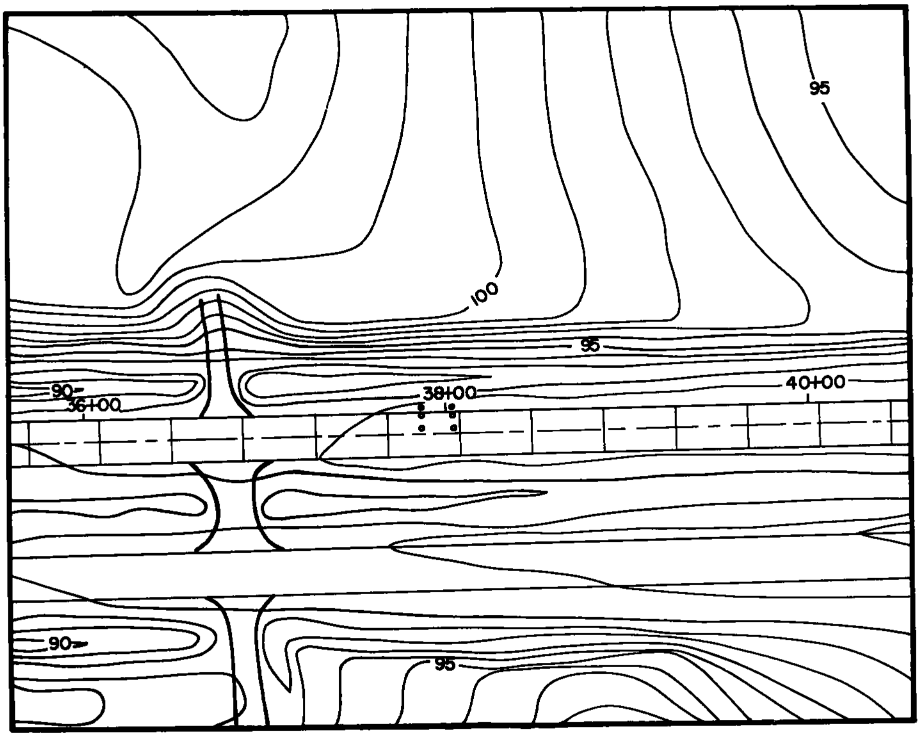


Figure F. Section 6.

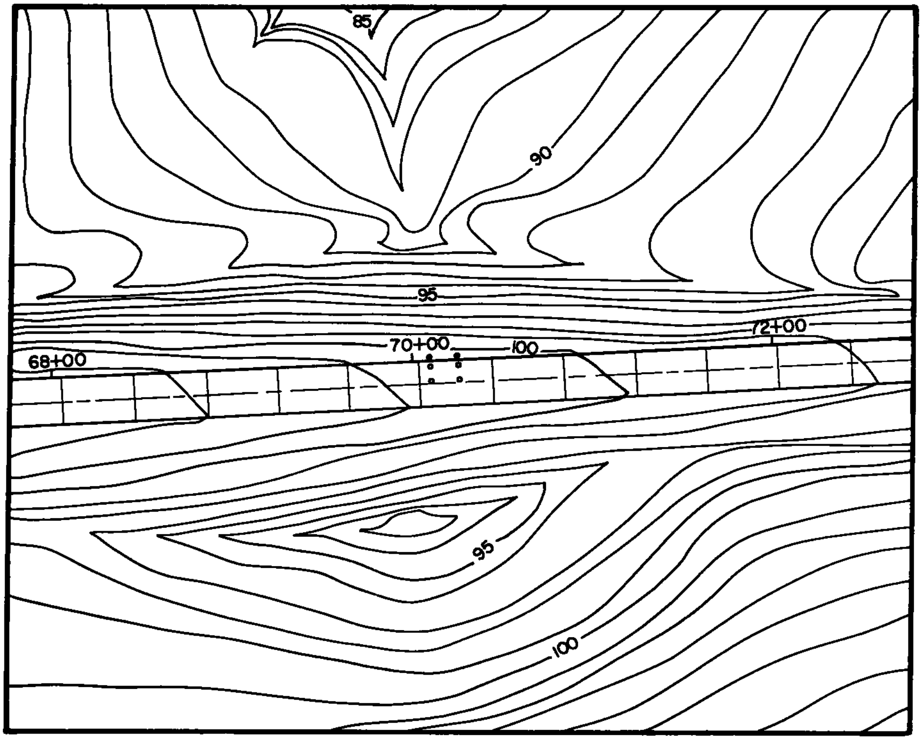


Figure G. Section 7.

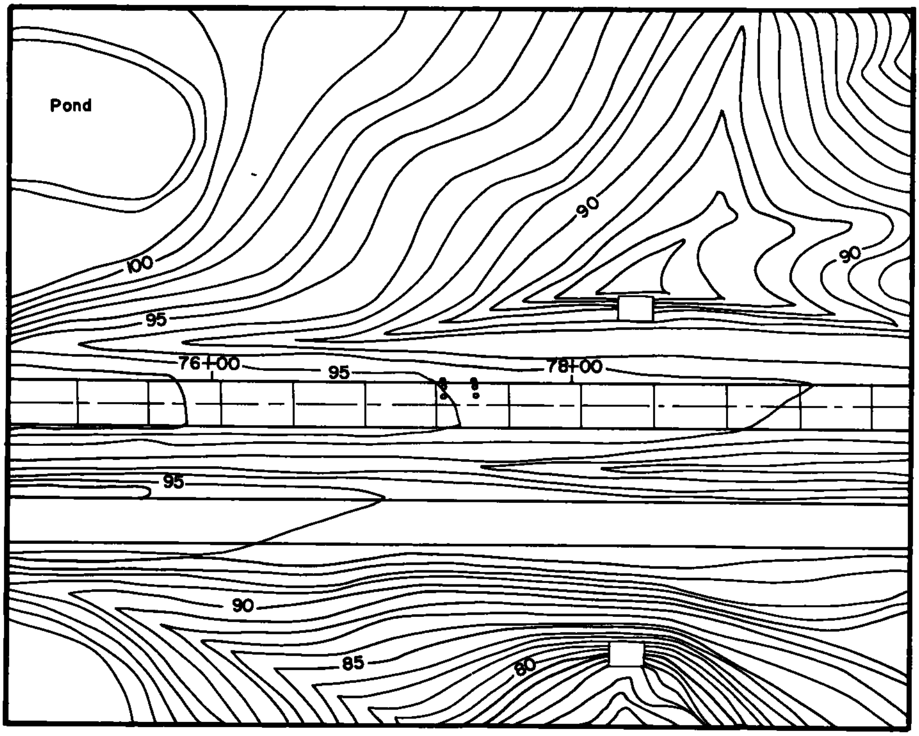


Figure H. Section 8.

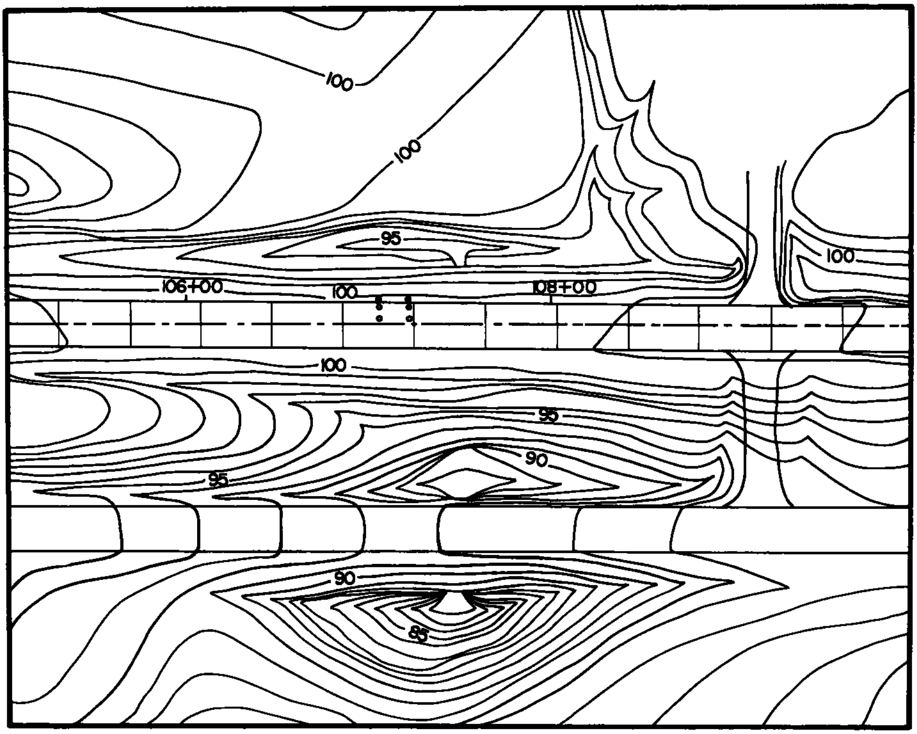


Figure 1. Section 9.

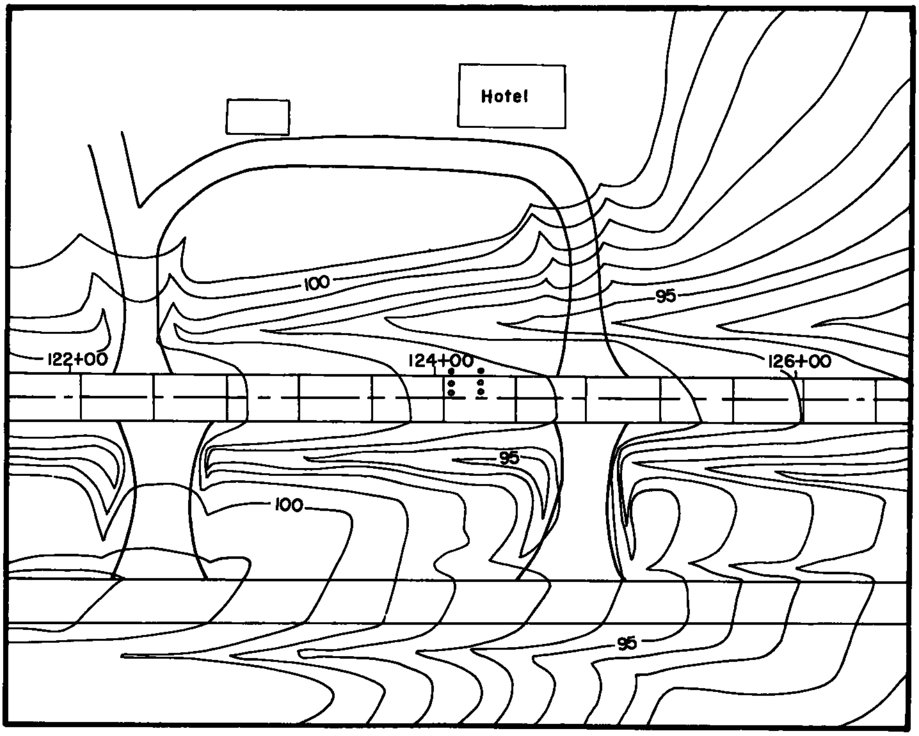


Figure J. Section 10.

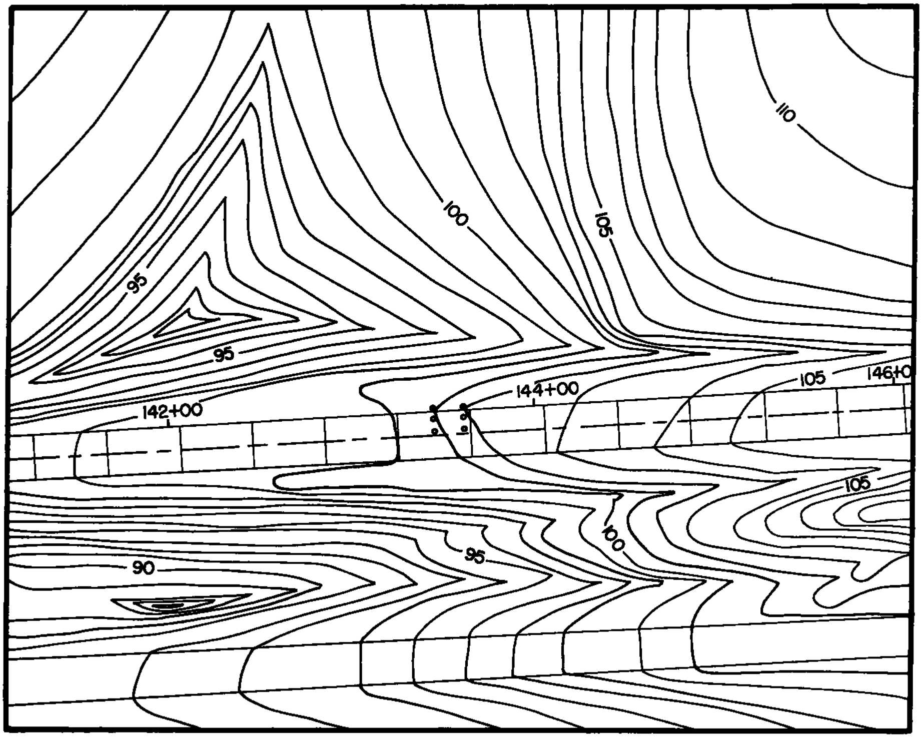


Figure K. Section 11.

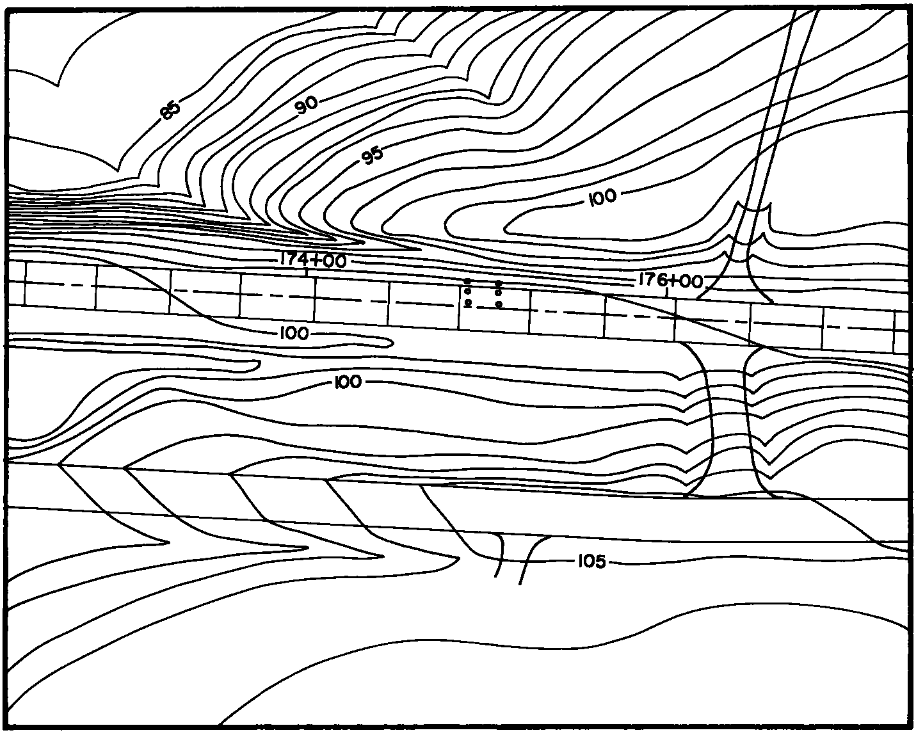


Figure L. Section 12.

Blocks placed on +0.314 percent grade; eastbound roadway even; topography rather level to the north, south, and east and rolling off to the west.

Station 70+08 and 70+25 Test Section No. 7. Limestone aggregate, sand-gravel base on oiled earth.

Centerline fill 7 ft.; special 3 ft. ditch to south and culvert drain to north. Culvert crosses roadway at 70+00. Blocks placed on -0.26 percent grade; eastbound roadway 4 ft. higher; general drain of all surrounding ground is gently to the north directly north of the blocks.

Station 77+28 and 77+45 Test Section No. 8. Limestone aggregate, sand-gravel base on plain earth.

Centerline fill 9 ft.; special 3 ft. ditch to north and standard parkway ditch to south. Blocks placed on -0.26 percent grade; eastbound roadway even on high fill; hill to northwest slopes sharply to south and east. Cattle pass under roadway at 78+50.

Station 107+05 and 107+22 Test Section No. 9. Limestone aggregate on plain earth.

Centerline fill 8 ft.; special 3 ft. ditch to north and standard parkway ditch to south. Blocks placed on a +0.39 percent grade; eastbound roadway about 8 ft. lower. Ground slopes gently to south and to culvert on north side at 107+38.9.

Station 124+08 and 124+25 Test Section No. 10. Limestone aggregate on oiled earth.

Centerline cut 2½ ft.; standard roadway ditch to north and standard parkway ditch to south. Blocks on 1100 ft. VC going into a -2.00 percent grade; eastbound roadway about 3 ft. higher; ground gentle to north, west and south, rolling to east.

Station 143+45 and 143+62 Test Section No. 11. Chert gravel aggregate on oiled earth.

Centerline fill 6 ft.; special 3 ft. ditch on both sides. Blocks placed on +1.62 percent grade; eastbound roadway about 6 ft. lower. Ground slightly rolling to north and east, drainage to west and south.

Station 174+88 and 175+05 Test Section No. 12. Chert gravel aggregate on plain earth.

Centerline cut 5 ft.; standard roadway ditch to north and standard parkway ditch to south. Blocks placed on -0.26 percent grade; eastbound roadway about 3 ft. higher; ground slightly rolling in all directions, general drainage to northeast and northwest.

Appendix C: Subgrade Log of Experimental Section and Chart of Subgrade Moisture and Density Tests

Begin Cut Sta. 827+50
Begin Exp. Section 827+69

827+69 - 828+75 Horizon B
828+75 - 831+00 Horizon C
End Cut 831+00

831+00 - 831+50 No. side cut-So. side fill
831+00 - 831+50 Horizon B (At ℄)

Begin Fill 831+50
831+50 - 832+50 Mainly Horizon B
832+50 - 835+00 Mainly Horizon C
835+00 - 835+50 Mainly Horizon B

Begin Cut 835+50
835+50 - 836+00 Undergraded-backfilled with Horizon C
836+00 - 835+50 Horizon B
836+50 - 837+50 Horizon C
837+50 - 841+25 Blue clay layer of Glacial Till
841+25 - 844+75 Horizon C
844+75 - 845+00 Horizon B

Begin Fill
845+00 - 846+00 Mainly Horizon B
846+00 - 854+50 Mainly Horizon C
854+50 - 856+00 Mainly Horizon B

Begin Cut
856+00 - 856+25 Undergraded-backfilled with Horizon B
856+25 - 856+75 Horizon B
856+75 - 858+50 Horizon C
858+50 - 864+25 Blue clay layer of Glacial Till
864+25 - 866+75 Horizon C
866+75 - 867+00 Horizon B

Equation
$859+52^{\underline{68}}$ Bk. = $859+55^{\underline{40}}$Ah.
-2.72'

Side hill No. side fill-So. side cut
867+00 - 867+50 Undergraded-replaced with Horizon C

Begin Fill
867+50 - 868+50 Mainly Horizon A
868+50 - 879+00 Mainly Horizon C
879+00 - 879+50 Mainly Horizon B

Side hill No. side cut-So. side fill
879+50 - 880+00 Undergraded-replaced with Horizon C

Begin Cut
880+00 - 880+25 Horizon B
880+25 - 881+75 Horizon C
881+75 - 882+30 Blue clay layer of Glacial Till
882+30 - 884+21 Horizon C
884+21 - 884+50 Horizon B

Begin Fill
884+50 - 885+50 Mainly Horizon B
885+50 - 889+75 Mainly Horizon C
889+75 - 890+50 Mainly Horizon B

Side hill So. side cut-No. side fill
890+50 - 891+00 Undergraded-replaced with Horizon C

Begin Cut
891+00 - 891+60 Horizon B
891+60 - 893+25 Horizon C
893+25 - 893+70 Blue clay layer of Glacial Till
893+70 - 896+88 Glacial Till
896+88 - 897+17 Blue clay layer of Glacial Till
897+17 - 897+25 Horizon C
897+25 - 898+00 Horizon B

Side hill No. side cut-So. side fill
898+00 - 898+65 Undergraded-replaced with mixture

Begin Fill
898+65 - 901+00 Mainly Horizon B

Side hill So. side cut-No. side fill
901+00 - 903+00 Horizon B

Begin Fill
903+00 - 905+50 Mainly Glacial Till
905+50 - 906+50 Mainly Blue clay layer of Glacial Till
906+50 - 909+00 Mainly Horizon B

Side hill No. side cut-So. side fill
909+00 - 910+00 Horizon C (Fill at ℄)
910+00 - 912+66 Horizon B
912+66 - 913+00 Horizon C

Begin Cut
913+00 - 914+50 Horizon C
914+50 - 914+88 Blue clay layer of Glacial Till

Cut
914+50 - 914+88 Blue clay layer of Glacial Till
0+00 - 1+25 Blue clay layer of Glacial Till
1+25 - 3+20 Glacial Till
3+20 - 5+72
5+72 - 7+00 Horizon C

Side hill No. side cut-So. side fill
7+00 - 8+00 Horizon B

Begin Fill
8+00 - 10+50 Mainly Horizon C

Begin Fill
8+00 - 10+50 Mainly Horizon C
10+50 - 13+38 Mainly Mixture of Blue clay & Glacial Till
13+38 - 15+20 Mainly Horizon B
15+20 - 16+00 Mainly Horizon C

Side hill No. side cut-So. side fill
16+00 - 16+20 Horizon B
16+20 - 16+50 Horizon C

Begin Cut
16+50 - 17+50 Horizon C
17+50 - 24+15 Glacial Till
24+15 - 25+10 Horizon C
25+10 - 25+50 Horizon B

Equation
$22+58^{\underline{8}}$ Bk. = $22+67^{\underline{7}}$ Ah
= -8.9'

Begin Fill
25+50 - 33+50 Mainly Glacial Till

Begin Cut
33+50 - 33+90 Horizon B
33+90 - 34+51 Horizon C
34+51 - 35+15 Blue clay layer of Glacial Till
35+15 - 35+85 Glacial Till
36+85 - 38+52 Blue clay layer of Glacial Till
*38+52 - 40+82 Horizon C
*40+82 - 43+00 Horizon B

Begin Fill (slight)
*43+00 - 44+50 Mainly Horizon B

Begin Cut (slight)
*44+50 - 45+40 Horizon B
*45+40 - 46+30 Horizon C
*46+30 - 48+00 Horizon B

Begin Fill
48+00 - 53+50 Mainly Horizon B

Begin Cut
53+50 - 54+15 Horizon B
54+15 - 61+65 Horizon C
61+65 - 62+50 Horizon B

Begin Fill
62+50 - 64+25 Mainly Horizon B
64+25 - 72+50 Mainly Horizon C

Begin Cut
72+50 - 73+40 Horizon B
73+40 - 75+75 Horizon C

Side hill No. side cut-So. side fill
75+75 - 76+24 Horizon C
76+24 - 76+80 Horizon B

Begin Fill
76+80 - 80+00 Mainly Horizon C

Begin Cut
80+00 - 81+35 Horizon C
81+35 - 86+85 Blue clay layer of Glacial Till
86+85 - 99+25 Horizon C

Note: Sec. 8 ends at Sta. 97+21

Cut
86+85 - 99+25 Horizon C
99+25 - 101+50 Blue clay layer of Glacial Till
101+50 - 104+40 Horizon C
104+40 - 105+50 Horizon B

Begin Fill
105+50 - 106+20 Mainly Horizon B
106+20 - 108+40 Mainly Horizon C
108+40 - 109+00 Mainly Horizon B

Side hill No. side cut-So. side fill
109+00 - 110+50 Horizon C (topsoil removed left of ℄ and replaced with Horizon C)

Begin Cut (slight)
110+50 - 113+00 Horizon B

Begin Fill (slight)
113+00 - 120+00 Mainly Horizon B (6 in. of topsoil removed from 113+00-120+00 and replaced)

*4 inches removed below base grade and replaced with Horizon C soil from borrow pit from 4 to 12 inches below basegrade is as indicated.

Fill (slight)
120+00 - 121+00 Mainly Horizon B

Begin Cut
121+00 - 121+60 Horizon B
121+60 - 125+60 Horizon C
125+60 - 126+00 Horizon B

Begin Fill
126+00 - 126+25 Mainly Horizon B
125+25 - 128+25 Mainly Horizon C
128+25 - 128+50 Mainly Horizon B

Side hill No. side cut-So. side fill
128+50 - 129+65 Horizon B
129+65 - 135+00 Horizon C

Begin Cut
130+00 - 133+15 Horizon C
133+15 - 133+50 Horizon B

Side hill No. side fill-So. side cut
133+50 - 136+00 Horizon B

Begin Fill
136+00 - 141+00 Mainly Horizon C
141+00 - 143+50 Mainly Blue clay layer of Glacial Till
143+50 - 144+50 Mainly Horizon C
144+50 - 145+00 Mainly Horizon B

Begin Cut
145+00 - 145+25 Horizon B
145+25 - 145+75 Horizon C
145+75 - 151+50 Blue clay layer of Glacial Till
151+50 - 152+50 Horizon C

Side hill No. side cut-So. side fill
152+50 - 153+60 Horizon C
153+60 - 157+00 Horizon B

Begin Fill
157+00 - 160+00 Mainly Horizon B
160+00 - 162+50 Mainly Horizon C

Side hill No. side fill-So. side cut
162+50 - 163+25 Horizon C

Begin Cut
163+25 - 165+60 Horizon C
165+60 - 166+25 Horizon B

Begin Fill
166+25 - 168+00 Mainly Horizon B
168+00 - 173+25 Mainly Horizon C
173+25 - 174+00 Mainly Horizon B

Begin Cut
174+00 - 175+50 Horizon C
175+50 - 176+45 Blue clay layer of Glacial Till

End of Experiment

Survey made by M. S. Lattimore

TABLE A

SUBGRADE MOISTURE CONTENT AND DENSITY PRIOR TO BASE CONSTRUCTION

Experimental Section Number	Station	Distance from Joint	3 Ft from Edge Pavement, 4 In Depth, % H_2O	3 Ft from Edge Pavement, 4 In Depth, #/cu ft	3 Ft from Edge Pavement, 9 In Depth, % H_2O	3 Ft from Edge Pavement, 9 In Depth, #/cu ft	9 Ft from Edge Pavement, 4 In. Depth, % H_2O	9 Ft from Edge Pavement, 4 In. Depth, #/cu ft	9 Ft from Edge Pavement, 9 In Depth, % H_2O	9 Ft from Edge Pavement, 9 In Depth, #/cu ft
1	832+00	20	21.8	103 6	21 6	102.6	18.1	108.4	28.1	97.6
	832+17	3	21 3	105.5	26.8	98.4	20.9	106.4	22.6	102.7
	833+00	-	16 8	112.7	19.0	103.6	29 3	96 2	16 6	96 2
	837+00	-	19 1	102.5	23 1	106 0	15.7	114.2	20 5	102.9
	840+00	20	18.3	104.8	26 6	99 6	26 3	95 7	28 0	97 1
	840+17	3	28 8	95 4	27.5	97 0	25.3	99 8	27.3	97 8
	842+00	20	28 6	94.7	23 5	104 2	27 2	100.8	27.7	99.5
	842+17	3	26.4	94.0	24 4	101.7	21.9	102 4	26.3	87.3
2	855+20	20	24 3	102.7	14 7	111.2	16.5	113.1	19 0	109 3
	855+37	3	17 6	113 2	13 7	104 3	18.3	111 6	19.9	109.1
	857+03	3	20.9	107.4	21.3	105.9	21.4	106.6	20.2	106.7
	857+20	20	19 5	110.3	20 4	108.7	19.8	108 8	20 2	110 1
	859+83	3	20.9	107 4	22 1	105.7	21.2	108 4	21.7	106 6
	860+03	20	23 0	99.6	21.1	109.3	22.9	105 5	21.8	104 7
3	865+06	3	20 9	104 6	23 8	102 6	19 1	103 7	21 6	105 1
	865+23	20	22 8	101 1	28 3	96.6	22 7	102.6	24.0	101 2
	875+23	20	18 8	108 3	20 6	102 6	25 2	100 0	24.0	97 7
	875+40	3	16 6	107 1	20.5	107.4	20 3	110 2	21.9	103 4
	881+23	20	22 6	107.8	24.0	103.8	21 4	108.1	22.5	105 9
	881+40	3	24 8	103 0	22.9	104 3	28.8	97 3	27 3	99 8
4	883+06	3	18 5	112 2	18.2	107.9	18.8	111.9	23 1	105 7
	883+20	20	19 5	110 2	20 4	106 2	19 2	111 9	20 5	107.8
	891+06	3	17 8	110 2	21 7	97 0	15 9	104 9	19.6	94 6
	891+23	20	16 6	109.4	23 7	99 4	19 5	108 5	27 9	90 6
	895+06	3	16.6	111 6	18.5	94 3	18 1	104 4	19 7	98 7
	895+23	20	25 5	104 3	15 5	108.4	17 4	106 4	19 9	105 0
5	903+07	3	22 2	102 3	20.3	101.1	21 8	98 0	16.2	104 1
	903+24	20	21 8	105.8	18 8	104.0	18.9	107.5	18 9	104 2
	911+24	20	17.9	97.3	17 5	92.9	15 0	102.3	17 3	94 4
	911+41	3	17.5	87.2	18.6	96.3	17.7	95.1	17 4	93 6
	2+56	20	18.3	110 3	15.6	136 4	20 1	95 8	14.9	119.8
	2+73	3	19.5	109 6	18.3	112 8	19 0	110 5	17 8	114 5
6	11+59	3	16.4	99.3	19 2	107 8	27.6	100.6	19.9	108.1
	11+76	20	15 8	91.6	20.0	108 3	21 0	107.9	17 7	93 5
	28+25	20	18 3	114 1	24 3	98 8	17.9	106.0	18.8	109.3
	28+42	3	19 3	110.9	18 2	96 7	18 4	112 2	18 3	112 5
	37+85	20	23 3	102 7	21 4	107 6	23 8	103 1	19 8	109 0
	38+02	3	21 5	104 8	21 6	102 5	23.9	102 2	20.8	105.5
7	45+05	20	21 6	102.5	32 6	89.3	29 8	94 8	30 4	90 3
	45+22	3	25 2	98 5	28 9	94.6	28.6	95.7	31 4	92 2
	59+68	3	19 7	100 1	21 3	99 3	15 9	102 2	17 4	100 5
	59+85	20	18 9	110 9	19.4	108 9	14 0	113 2	15 1	112.2
	70+08	3	17.2	114 0	16 6	115 4	18.2	105.1	16 3	99 0
	70+25	20	18 9	112.0	16 8	106 5	18.9	112 7	16 2	101.1
8	77+28	3	17 9	112 3	22 2	105.1	21 5	105.8	17 8	102.2
	77+45	20	19 3	113.2	22 0	106 6	22 1	107 0	16 7	113.2
	81+28	3	26 2	100 6	23 9	103 7	26 1	99.3	25 4	99 9
	81+45	20	23 5	103 2	20 0	109 7	23 2	103 1	21 4	107 1
	91+05	20	21.0	107.4	23.0	105.6	18 6	109 6	22 8	104.6
	91+22	3	19 8	110.4	20.6	108 3	18 4	112 4	19 2	110.2
9	100+25	20	24 2	102.6	30 9	91 9	26 2	91.8	32.3	91.1
	100+42	3	35 0	89 0	34 9	87 9	36.1	88 2	40 8	82.6
	107+05	20	24.1	92.4	26 0	100 9	20.3	110.1	21.2	108.0
	107+22	3	22 4	105.6	21 8	108 0	20 5	109.7	18 9	113 8
	111+05	20	27.7	94.9	36.0	87.9	20 0	105 8	35 5	87 4
	111+22	3	21 2	100.1	19 0	100.3	19.7	108.9	32.3	89.8
10	124+08	3	19 9	103.8	16 0	108 1	14 8	109.8	14.4	108 5
	124+25	20	16 0	104.6	13.1	108.8	13 4	110.1	13 1	108 5
	127.28	3	23 8	101.7	23 5	102 0	20 2	98.0	20.2	102 2
	127+45	20	26.4	95.5	23.2	102 3	20 0	94 3	20 0	104.3
	134+48	3	21 7	105 7	14 5	111.9	16.6	114.3	22 4	106 5
	134+65	20	17.3	111.1	12.9	104.4	28 6	97.2	19.2	110 4
11	143+45	20	22 1	93 9	15 3	103 0	21 5	106.9	24.7	94.3
	143+62	3	23.2	104.2	23.9	97 4	23.5	102.8	22.6	90 5
	148+25	20	21 2	100.7	25 0	99 0	29.9	94.6	27.3	98.4
	148+42	3	26 8	95 0	30 7	92 0	21 5	99.0	32 8	88.9
	155+05	20	19.3	102.8	26 6	96 2	21.3	103 7	21 8	104.4
	155+22	3	30.3	92.6	29.7	92.9	17.9	110.6	27.2	96 5
12	167+28	3	21.2	101.5	20.9	88 3	21.4	103.2	18.2	92.5
	167+45	20	21.0	103.0	23 4	109.6	20.2	104 3	22 5	98.4
	171+28	3	16 6	114 9	18.5	105 5	20.3	106.8	20.8	105 8
	171+45	20	18.3	107.6	24 3	107 8	25.6	103 1	16 6	105.5
	174+88	3	16 6	109.8	18.5	106 3	18 4	105 5	17.2	110.6
	175+05	20	20 1	107.3	21.4	105.6	27.7	106.6	20.5	107.1
	Average		21 2	104 1	21.8	103.0	21 3	104 5	21.9	102.0

Appendix D: Validating Tests

The following tables show the results of validating tests which were made adjacent to the installed blocks. These tests were made by drilling the pavement above the blocks and manually sampling the subgrade to determine the moisture content by the dry weight method. The first table shows the comparison of the individual blocks while the second table shows the comparison of the moisture averages.

The moisture averages by the resistance block method vary as much as two percentage points from the manual sampling method. The individual moisture percentage vary from 4.8 percentage points higher to 3.9 percentage points lower with the average deviation as .37 percentage points higher.

TABLE A
VALIDATING TESTS

Block Number	Original Moisture	1954 Validation Resistance Method	1954 Validation Manual Sampling	Block Number	Original Moisture	1954 Validation Resistance Method	1954 Validation Manual Sampling
1	28.5	29.5	31.9	49	17.2	29.0	28.1
2	23.5	25.9	28.0	50	16.6	16.3	18.6
3	29.2	30.7	29.2	51	18.2	24.6	22.7
4	27.7	27.5	29.0	52	16.3	14.9	16.3
5	26.4	28.3	30.0	53	18.9	26.9	24.6
6	24.4	27.0	27.8	54	16.8	21.0	19.6
7	21.9	29.0	27.3	55	18.9	31.0	30.2
8	26.3	35.6	36.7	56	16.2	18.1	19.0
9	20.9	16.5	18.0	57	17.9	21.4	22.3
10	21.3	12.8	16.4	58	22.2	21.4	23.7
11	21.4	26.6	24.2	59	21.5	30.4	29.3
12	20.2	10.9	14.6	60	17.8	31.0	28.3
13	19.5	14.2	18.9	61	19.3	23.7	22.1
14	20.4	21.3	20.6	62	22.0	20.6	23.1
15	19.8	18.6	19.0	63	22.1	28.4	29.5
16	20.2	9.4	13.8	64	16.7	31.8	31.5
17	20.9	9.3	14.1	65	24.1	27.0	27.3
18	23.8	19.0	19.6	66	26.0	27.0	27.5
19	19.1	19.1	18.9	67	20.3	12.6	14.0
20	21.6	8.7	11.6	68	21.2	18.9	20.4
21	22.8	17.8	20.0	69	22.4	26.6	25.6
22	28.3	27.5	25.6	70	21.8	17.9	18.4
23	22.6	16.5	18.4	71	20.5	33.6	32.6
24	24.0	8.8	12.3	72	18.9	21.8	20.8
25	18.5	21.8	20.7	73	19.9	20.5	19.7
26	18.2	17.1	19.0	74	16.0	29.6	29.0
27	18.8	26.2	25.8	75	14.8	25.9	25.1
28	23.1	21.0	19.6	76	14.4	22.2	20.9
29	19.5	17.7	15.3	77	16.0	25.1	27.1
30	20.4	21.4	18.9	78	13.1	20.8	20.3
31	19.2	20.4	20.3	79	13.4	18.3	20.4
32	20.5	18.6	20.0	80	13.1	26.2	25.0
33	17.9	22.1	23.8	81	22.1	26.2	26.6
34	17.5	16.4	18.5	82	15.3	31.8	29.8
35	15.0	22.3	21.7	83	21.5	28.0	29.0
36	17.3	23.4	22.2	84	24.7	24.7	23.9
37	17.5	17.3	18.9	85	23.2	30.4	29.2
38	18.6	17.7	19.2	86	23.9	29.0	27.0
39	17.7	21.0	21.1	87	23.5	18.4	19.6
40	17.4	23.9	24.2	88	22.6	20.0	22.2
41	23.3	31.8	31.8	89	16.6	23.5	21.9
42	21.4	20.4	21.6	90	18.5	10.4	14.6
43	23.8	12.8	14.3	91	18.4	21.5	22.0
44	19.8	9.8	12.6	92	17.2	22.5	21.6
45	21.5	35.6	34.0	93	20.1	21.5	21.0
46	21.6	28.6	29.4	94	21.4	12.6	13.3
47	23.9	29.0	29.9	95	27.7	22.0	20.3
48	20.8	14.9	16.0	96	20.5	24.4	23.2

TABLE B
VALIDATING TESTS: MOISTURE AVERAGES

Figure Number	Position	Original Average	1951 Validating Ave. Resistance Method	1951 Validating Ave. Manual Sampling	1954 Validating Ave. Resistance Method	1954 Validating Ave. Manual Sampling
	O. E. 4" Depth	20.3	23	23	25.6	25.8
	O. E. 9" "	18.6	18	19	23.2	23.2
6	P. E. 4" "	19.8	24	23	20.8	21.1
	P. E. 9" "	21.5	22	23	19.1	19.6
	O. E. 4" "	19.9	20	20	23.3	23.4
	O. E. 9" "	18.8	17	18	17.1	18.5
7	P. E. 4" "	20.2	25	24	22.8	22.8
	P. E. 9" "	19.7	25	25	21.3	21.2
	O. E. 4" "	20.3	22	22	24.2	23.9
	O. E. 9" "	18.6	17	18	19.3	20.4
8	P. E. 4" "	19.1	24	23	21.4	22.2
	P. E. 9" "	19.5	24	25	19.7	20.5
	O. E. 4" "	19.9	20	22	24.8	25.3
	O. E. 9" "	18.9	17	19	20.9	21.3
9	P. E. 4" "	21.0	25	23	22.2	22.1
	P. E. 9" "	20.8	24	23	20.6	20.9
	O. E. 4" "	20.4	21	22	18.9	20.0
	O. E. 9" "	24.0	26	25	29.4	29.7
10	P. E. 4" "	20.5	25	25	13.6	16.3
	P. E. 9" "	25.6	36	34	29.0	30.4
	O. E. 4" "	20.4	21	22	18.9	20.0
	O. E. 9" "	20.5	25	25	13.6	16.3
11	P. E. 4" "	20.2	22	23	18.6	19.2
	P. E. 9" "	22.5	24	24	17.8	18.3
	O. E. 4" "	20.7	28	27	27.6	26.9
	O. E. 9" "	18.7	17	17	18.0	19.1
12	P. E. 4" "	18.6	28	28	23.3	23.6
	P. E. 9" "	18.7	27	27	23.3	23.8
	O. E. 4" "	19.3	14	15	24.1	24.6
	O. E. 9" "	17.9	14	16	25.6	24.8
13	P. E. 4" "	21.3	22	22	23.6	23.1
	P. E. 9" "	20.7	21	20	19.5	20.0
	O. E. 4" "	20.7	22	22	24.8	24.8
	O. E. 9" "	18.1	16	17	22.2	23.1
14	P. E. 4" "	19.3	22	22	20.8	21.6
	P. E. 9" "	21.0	22	23	18.5	20.5
	O. E. 4" "	19.9	18	17	23.1	24.0
	O. E. 9" "	18.6	14	17	17.7	19.2
15	P. E. 4" "	21.6	25	23	23.6	23.7
	P. E. 9" "	19.8	25	25	21.5	22.0
	O. E. 4" "	19.9	22	22	23.6	23.1
	O. E. 9" "	19.1	19	20	16.5	17.6
16	P. E. 4" "	18.9	25	24	22.0	21.9
	P. E. 9" "	20.4	26	26	20.9	20.6
	O. E. 4" "	19.8	23	23	26.4	26.8
	O. E. 9" "	19.2	20	21	24.1	23.4
17	P. E. 4" "	20.3	24	24	20.8	20.5
	P. E. 9" "	22.1	23	23	19.6	19.6

Frost Penetration Under Bituminous Pavements

MILES S. KERSTEN, Professor of Civil Engineering, University of Minnesota, and
RODNEY W. JOHNSON, Engineer, Hazelet and Erdal, Chicago, Illinois

This study was made near Minneapolis in the winter of 1953-54. Nine sites were selected, each a bituminous-surfaced road. At each of these locations borings were made to a depth of 8 feet to obtain soil textural information, moisture contents, and a few densities. During the winter season borings were made by a machine-driven auger at about 3-week intervals to determine the frost depths. Daily maximum and minimum air temperatures were recorded.

The subgrade soils at the test sites selected represent a variety of textures. The finest-grained soil was a silty clay loam with moisture contents between 25 and 30 percent. Other soils included loam, sandy loam, sandy loam till, and fine sands. Moisture contents in some of the sands were as low as 5 percent.

Observed frost penetrations at the end of the freezing season varied from about 3 1/2 feet in the silty clay loams to more than 5 feet for some of the sandy soils.

Theoretical calculations of frost penetration were made by means of the Stefan equation, utilizing an air-surface correction factor and the layered system solution developed by the Corps of Engineers. The moisture content and density data for the soils and the air temperatures are the only information required for these solutions. (Thermal conductivity coefficients were taken from published charts).

Comparisons of the observed and calculated frost depths indicate that an air-surface correction factor of 0.8 gives best results for the sites investigated. Utilizing this value, the difference between actual and calculated depths of freeze is commonly about 1/2 foot, plus or minus. Considering the many factors which affect frost penetration, this comparison is thought to be good.

In securing average daily temperatures by averaging maximum and minimum values for 24-hour periods, it was found that the time of recording made a significant difference. For the freezing season studied, the use of a period from 6 P.M. to 6 P.M. gave a freezing index about 6 percent less than that for a midnight to midnight period.

The investigation indicates the feasibility of making reasonable frost depth predictions for known soil and temperature conditions. Additional measurements should yield further refinements. Further study of the air-surface is considered particularly desirable. Similar procedures could also be applied to a study of the thawing of soils.

● ALTHOUGH the need for reasonably accurate methods for predicting depth of frost penetration has long been recognized, the difficulties of handling numerous variables, such as those of the soil, temperature, and other factors of climate, have until recently hindered such a development. In recent years work of the Corps of Engineers, Department of the Army, has indicated some promise in the use of the Stefan equation for calculating depths of freeze and thaw utilizing air temperatures, an air-surface temperature correction factor, and moisture content and density data on the soil section.

Carlson and Kersten, in a previous paper before the Highway Research Board (1), reported on observations and calculations for some airfield pavement sections in Alaska which showed a good agreement between measured depths of freeze and thaw by thermocouple readings and borings with values calculated on the basis of the Stefan equation. The literature records few studies of a similar nature in continental United States.

This study is an attempt to apply calculations by the Stefan equation to some test locations in Minnesota and compare these calculated depths with observed depths. The calculations are based on such easily collectable data as air temperatures, soil texture, and moisture content. No ground temperature installations, such as thermocouple strings, were utilized. For comparison with calculated depths of freeze, auger borings were made periodically during the winter season and the depth of freeze in the holes noted.

All test points were on bituminous-surfaced roads which were kept free of snow. The subgrade soils at the test points represent a variety of textures and also moisture contents. It is thought that the comparisons of calculated and observed depths afford a check on the degree of accuracy that may be expected for predictions of frost depth based on such procedures. It is also hoped that studies such as this will assist in indicating the magnitude of air-surface correction factors for northern United States.

EQUATION FOR FROST DEPTH

The Stefan equation as modified in studies by the Corps of Engineers has been presented in prior publications (1, 2) and the explanation will not be repeated here. It is based on the simplification that the only heat flow that need be considered in frost penetration is that represented by the latent heat of fusion of the water in the soil. The Corps of Engineers modification includes the use of an air-surface correction factor to convert degree-days of freeze for air temperatures to those for the pavement surface. Also, the solution is applied to a layered system by considering the individual properties such as volumetric latent heat of fusion and thernal conductivity for each strata of soil. The equations as applied in this study may be stated as follows:

Consider a layered system, such as a bituminous mat, gravel base, and layers of underlying soil, numbered from the top down.

The degree-days required to freeze layer 1 is

$$F_1 = \frac{L_1 h_1^2}{48 k_1} = \frac{L_1 h_1}{24} \cdot \frac{R_1}{2}$$

in which F_1 = degree-days of freeze at surface required to freeze layer 1 in degrees Fahrenheit.

L_1 = volumetric latent heat of fusion in Btu. per cu. ft.
= 1.434 wd
w = moisture content of soil in percent
d = dry density of soil in pounds per cubic foot
h_1 = thickness of layer in feet
k_1 = thermal conductivity in Btu. per square foot per degree F. per foot per hour.
R_1 = thermal resistance $= \frac{h_1}{k_1}$

For layer 2

$$F_2 = \frac{L_2 h_2}{24} \left(R_1 + \frac{R_2}{2}\right)$$

For layer n

$$F_n = \frac{L_n h_n}{24} \left(R_1 + R_2 + \ldots\ldots + R_{n-1} + \frac{R_n}{2}\right)$$

$$= \frac{L_n h_n}{24} \left(\Sigma R_{n-1} + \frac{R_n}{2}\right)$$

in which ΣR_{n-1} = the summation of the thermal resistances of all layers above layer n.

To utilize these equations it is necessary to know certain characteristics of each layer, namely, moisture content, density, and thermal conductivity. Moisture contents and densities can be measured in the field at the test location. In the field studies reported herein, extensive moisture-content data were collected. Only a few density tests were made, but it is thought that reasonably accurate values could be estimated from textural information. Values of the coefficient of thermal conductivity have been selected from diagrams based on extensive laboratory studies previously reported (3, 4). Experience thus far has indicated that these values are reasonably correct.

The other item of information required for utilization of the equations is temperature data. Average daily temperatures during the freezing season were obtained by use of a maximum-minimum thermometer. Readings were recorded at 6 P.M. each day and

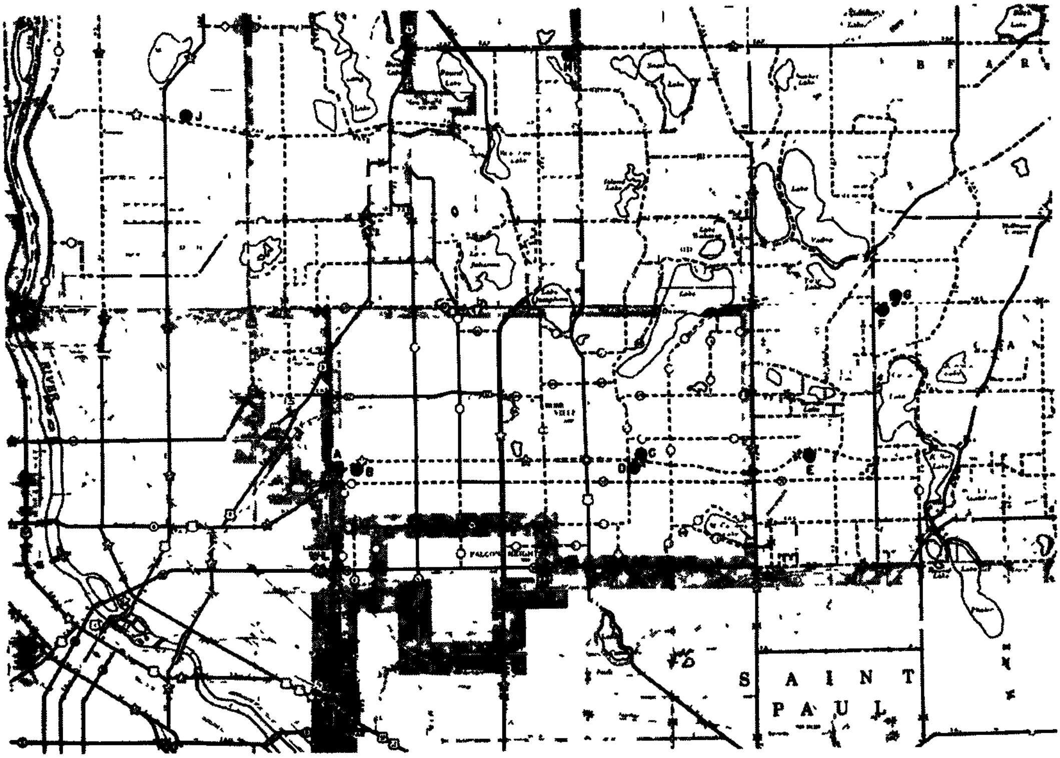


Figure 1. Map showing site location.

the two values averaged for calculation of the degree-days. Also, average daily temperatures were obtained from the U. S. Weather Bureau at Minneapolis.

The degree-days of freeze at the pavement surface, the term F in the equation, was taken as the degree-days of freeze based on air temperatures multiplied by an air-surface correction factor, C_f. The studies in Alaska had indicated a value of $C_f = 0.6$ to be correct. Therefore this same value was initially assumed for this investigation. However, other values of C_f were also considered in this study to attempt to obtain the best correlation between observed and calculated frost depths.

SITES SELECTED FOR STUDY

The field sites selected for the study are shown in Figure 1. The area is just north of St. Paul, Minnesota; all nine locations are within an area of about 50 square miles. All locations are points on roads with bituminous surfaces. It was attempted to select areas with a variety of textures of subgrade soils. The detailed information for each test point is given in the following paragraphs and in Figures 2 to 10, inclusive.

Soil textures were determined by visual and manual manipulation with numerous checks by hydrometer analysis; moisture content tests were made for every 0.5 foot; most of the densities are estimates based on the texture of the soils and some laboratory compaction tests at natural field moisture contents. Moisture contents for all test sites are given in Table 1 for ready comparison.

Site A

The location of Site A is on Minnesota Trunk Highway 36 just outside the corporate limits of Minneapolis (see Figure 1). The area surrounding this location is relatively level, having little cover with the exception of scattered brush to the south of the pavement. The road itself is a tangent section about 1/2 mile long, connecting relatively

TABLE 1
MOISTURE CONTENTS - FIELD TEST SITES

Depth in Feet[a]	Moisture Content, Percent of Dry Weight Site								
	A	B	C	D	E	F	G	H	J
0 - 0.5	3.2	2.0	3.3	7.9	8.1	17.6	19.1	6.9	6.6
0.5 - 1.0	14.5	3.7	6.7	6.2	7.2	17.4	27.3	8.2	4.8
1.0 - 1.5	13.5	6.3	5.1	7.1	10.3	20.2	23.9	10.3	4.8
1.5 - 2.0	15.0	11.7	5.4	5.1	9.4	28.6	24.1	13.3	5.8
2.0 - 2.5	13.0	11.8	5.8	6.6	8.7	25.1	24.3	15.7	6.1
2.5 - 3.0	14.0	10.4	5.4	7.3	8.8	24.7	22.8	15.5	5.9
3.0 - 3.5	14.7	9.5	5.1	7.9	8.7	25.3	23.8	18.3	6.7
3.5 - 4.0	14.7	10.2	5.0	6.9	8.3	24.3	25.7	19.8	6.0
4.0 - 4.5	20.6	9.1	4.5	5.4	9.0	25.7	28.0	19.2	5.7
4.5 - 5.0	19.8	9.4	4.5	4.8	9.4	25.7	30.7	17.6	7.5
5.0 - 5.5	22.2	12.3	4.2	5.0	9.4	25.9	32.3	16.8	8.3
5.5 - 6.0	22.0	13.5	4.6	4.7	9.9	24.6	34.7	18.8	10.1
6.0 - 6.5	19.8	12.4	9.2	4.4	10.0	23.1	31.1	17.6	9.8
6.5 - 7.0	19.0	8.2	9.1	4.4	9.9	24.7	29.4	17.2	6.8
7.0 - 7.5	15.5	8.3	7.5	5.5	10.3	24.9	29.3	17.5	6.9
7.5 - 8.0	16.2	9.5	6.1	10.4	10.3	26.3	26.1	17.7	7.4

[a] All depths measured from top of bituminous surface.

flat reversed curves. The cross-section is level, with moderate ditches; the entire area offers little cover or wind protection.

The soil encountered in this location is described in the Soil Survey of Ramsey County, United States Department of Agriculture, as Miami loam. Hydrometer analysis of the soil gave a loam texture (United States Bureau of Chemistry and Soil Classification) for the entire depth of the hole. The particle size of 0.02 mm. is sometimes considered as being significant in the frost-susceptibility of a soil. About 33 percent of this soil is smaller than 0.02 mm.

Site B

Site B is just 0.1 mile east of Site A. Cover conditions are similar, the road being in an exposed location. The soils at this site, however, differ from those at Site A. The subgrade is a mixture of sandy loams of both gray and red drift origin. On the soil map this location is on the apparent border of Miami and Gloucester soils.

Sites C and D

Sites C and D can be considered together, since their locations are close to one another and the soils are the same. This section of the road and the area adjacent to the sites are relatively level; there is no brush or trees within several hundred feet. Pasture and cultivated areas abut the roadway both north and south. The natural moisture content of the soil in borings made October 11, 1953, showed 6 to 9 percent at Site C and 5 to 8 percent at Site D. The soil at both locations was a relatively clean fine sand. The area is mapped as the Merrimac series.

Site E

Site E has more protection from wind and less exposure to sun than most of the other sites. The highway at this location is wooded on both sides (see Figure 6); the site was chosen in order to have a soil of the Gloucester fine sandy loam texture. There soils are derived from glacial red drift; the stony nature of this material proved to be an

TABLE 2
MEASURED DEPTHS OF FROST PENETRATION

Site	Depth of Frost Penetration, Inches[a]							
	1953					1954		
	Dec. 11	Dec. 18	Dec. 19	Dec. 23	Dec. 31	Jan. 15	Feb. 5	Mar. 9
A	7	20			27	36	47	76[b]
B	6	19			27	36	55	90[c]
C	9	12[b]			31	40.5	51.5	32[b]
D	10			19	33.5	43.5	60.5	30[b]
E	8		18		27	40.5	70	37[d]
F	6	18			23	32	43	40
G	4	17			23	31	43.5	41
H	5		19		27	36	52	42[b]
J	0[b]		15		32	43.5	68	52[e]

[a] All depths measured from top of bituminous surface.
[b] Reliability of data in doubt.
[c] Thawed depth approximately 2 to 2.5 feet.
[d] Thawed depth approximately 2 feet.
[e] Thawed depth approximately 1.5 feet.

undesirable feature. In the actual boring operation the presence of stones, cobbles, etc., made boring difficult and determination of depth of freeze was not readily made since the granular nature of the soil did not result in a positive line of demarcation between the frost zone and nonfrozen soil. This made the value of Site E as a test location somewhat dubious.

Sites F and G

Sites F and G will also be considered together, since the soils are the same and the two are thought of as verifications of each other. The location is on a county road with a narrow (20-foot) road-mix bituminous surface. The surrounding terrain is level and cultivated fields abut north and south. The top soil is black with high organic content. A relatively poor natural drainage accounts in part for the high moisture content of 23 to 30 percent. The texture of the soil is sandy clay loam. The soil survey map shows the area as the Clyde series. A mechanical analysis of this material indicates about 22 percent of clay and 60 percent of silt. These two locations represent the heaviest soils which could be found in the general test area. The entire study would have benefited could a location having a heavy clay soil been found.

Site H

The highway in the area of Site H is level with open space both north and south affording no wind or sun protection. The subgrade soil is a sandy loam containing about 13 percent of clay and 27 percent of silt. The variations in moisture content are attributed to some differences in the amount of silt and clay at different depths.

Site J

Site J is a departure from the cover and wind exposure conditions of the other sites as it is in a deep cut section of highway with trees bordering to the north and south. The soil survey map of Anoda county shows this soil to be a fine sandy loam of the Miami series. The hydrometer analysis indicated the soil to be a gravelly sand near the surface and a sandy loam till derived from red glacial drift at deeper depths. The good grading and gravel content of the soils resulted in the highest densities, 110 to 118 pcf., of the soils tested.

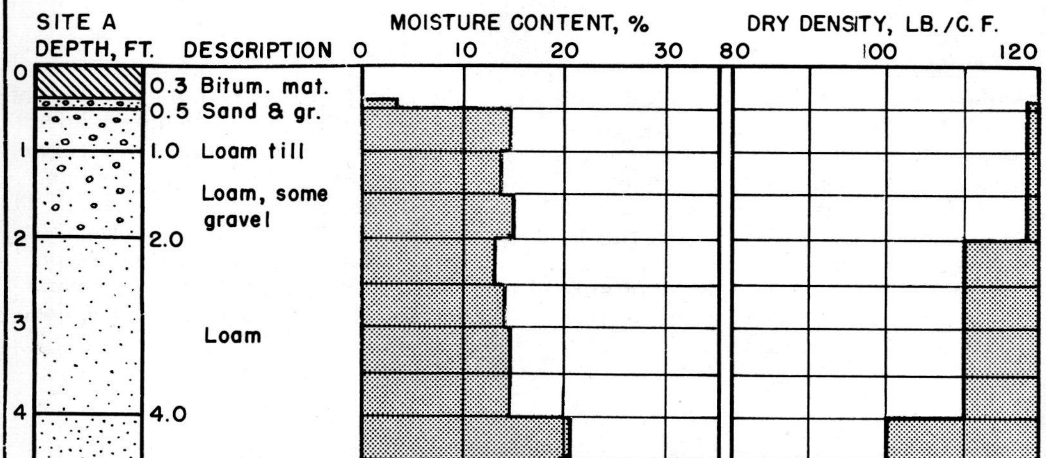
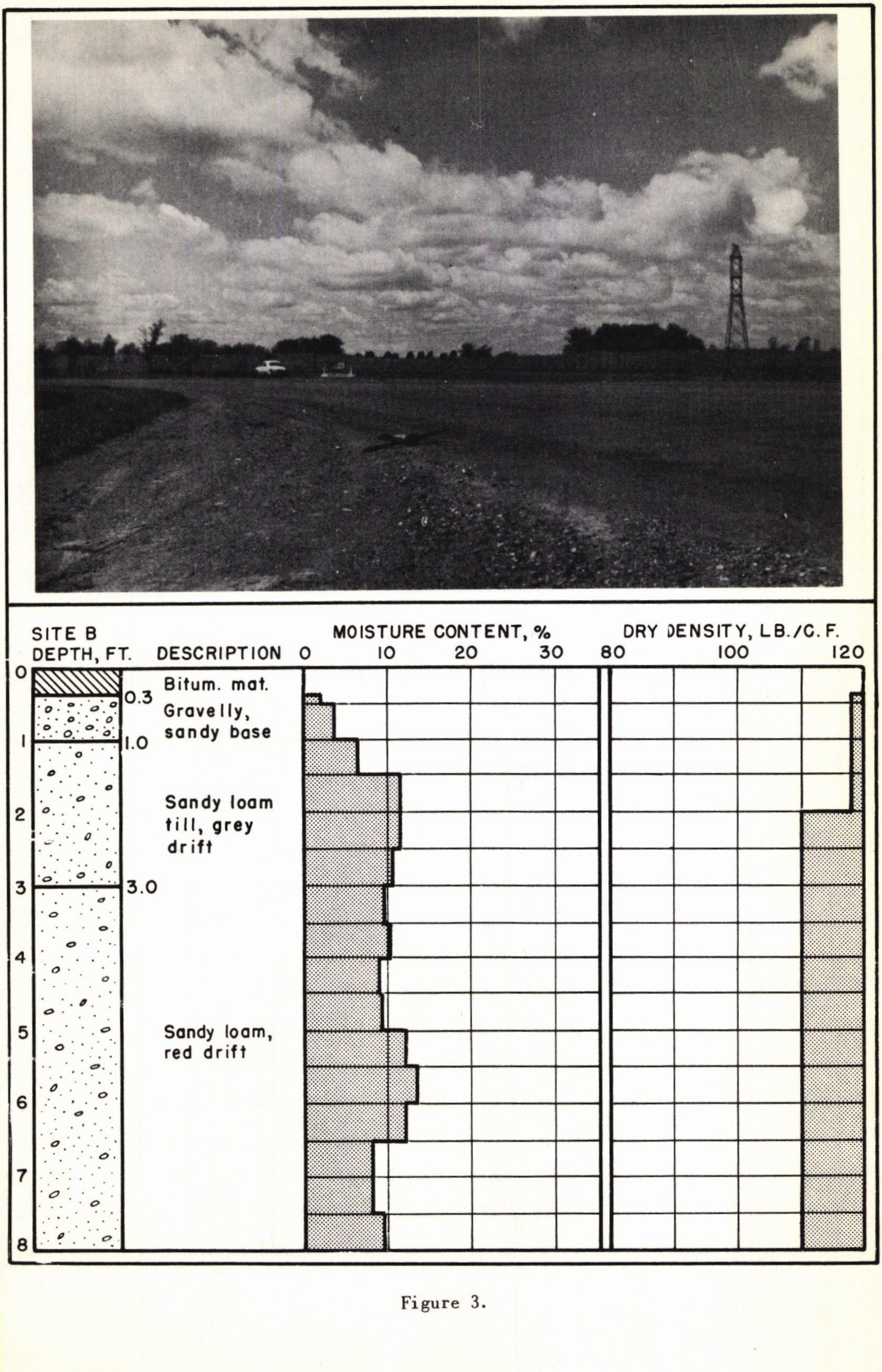

SITE A
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C. F.
0 10 20 30
80 100 120
0 1 2 3 4 5 6 7 8
0.3 Bitum. mat.
0.5 Sand & gr.
1.0 Loam till
Loam, some gravel
2.0
Loam
4.0
Loam w/silty streaks
6.0
Loam

Figure 2.



SITE B
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C. F.
0 10 20 30
80 100 120
0 1 2 3 4 5 6 7 8
0.3
1.0
3.0
Bitum. mat.
Gravelly, sandy base
Sandy loam till, grey drift
Sandy loam, red drift

Figure 3.

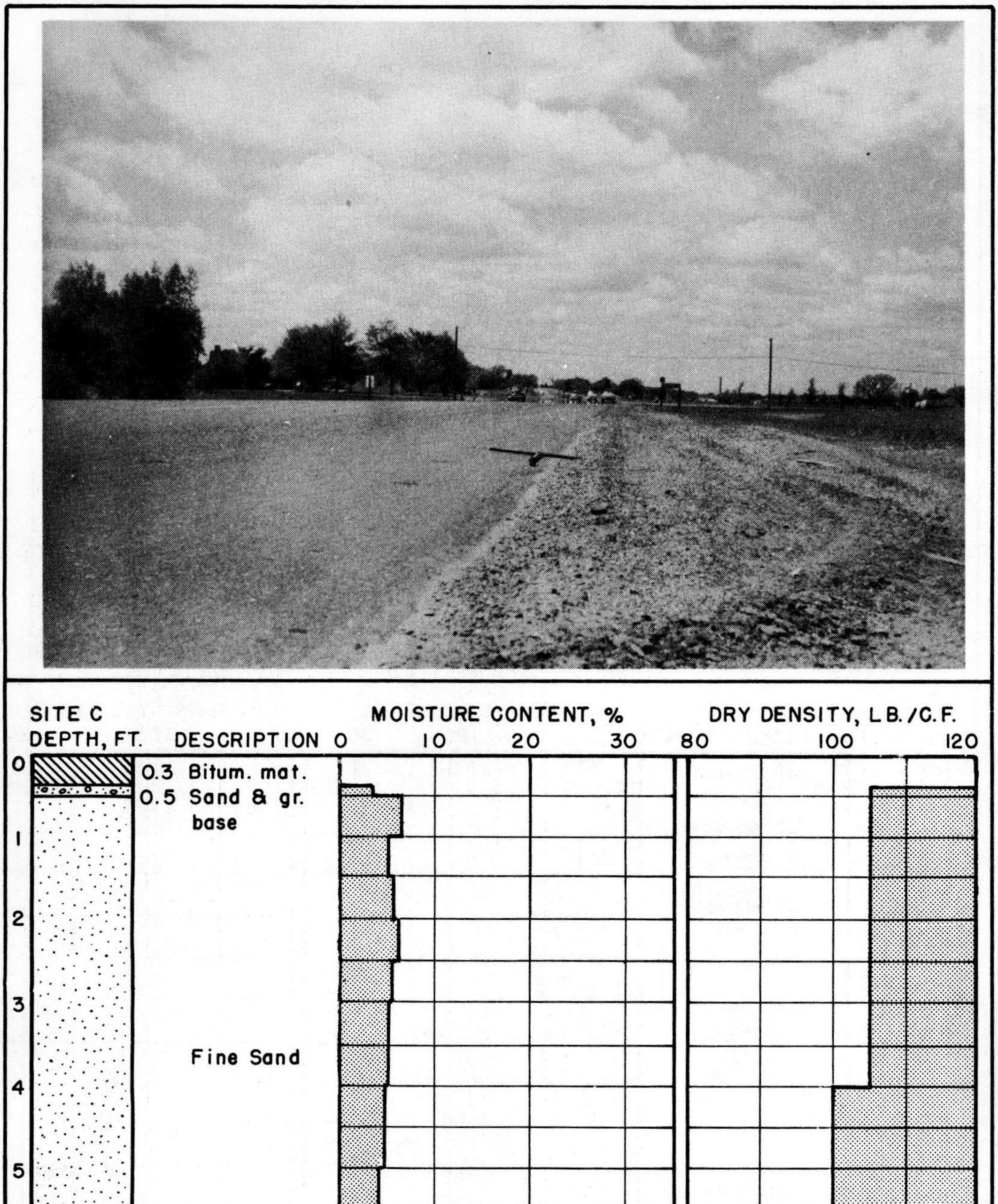

SITE C
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C.F.
0
10
20
30
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
0.5 Sand & gr. base
Fine Sand

Figure 4.

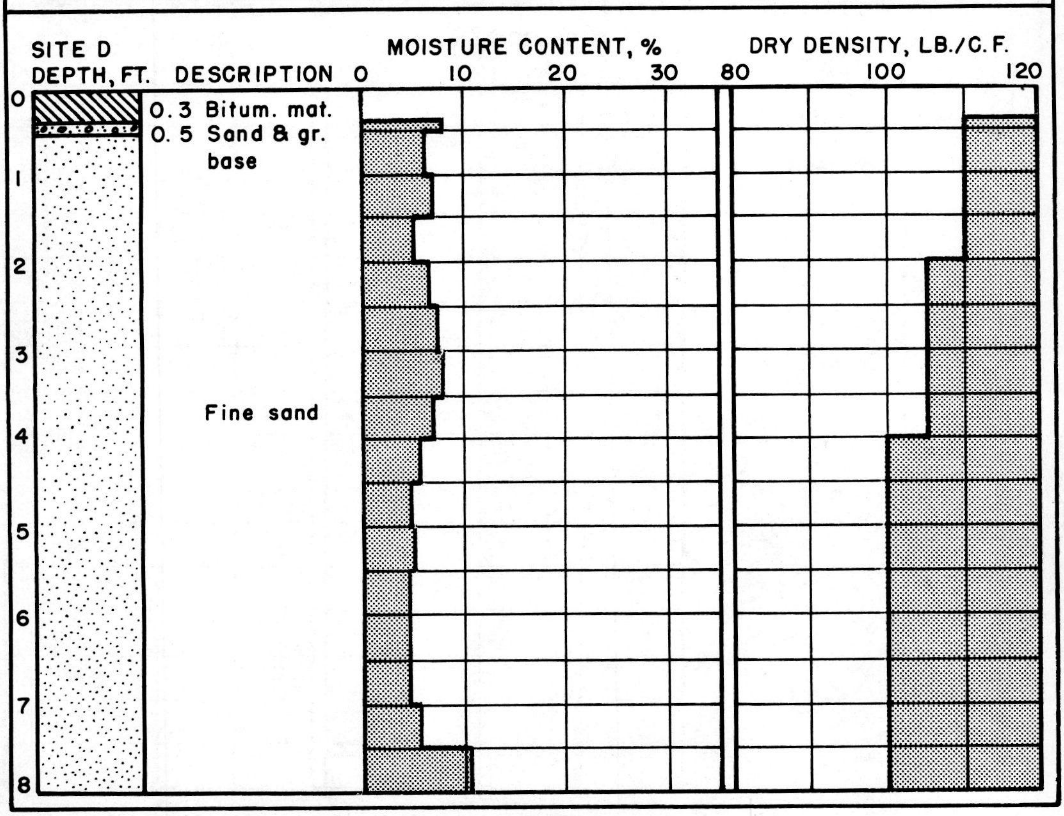

SITE D
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C.F.
0
10
20
30
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
0.5 Sand & gr. base
Fine sand

Figure 5.

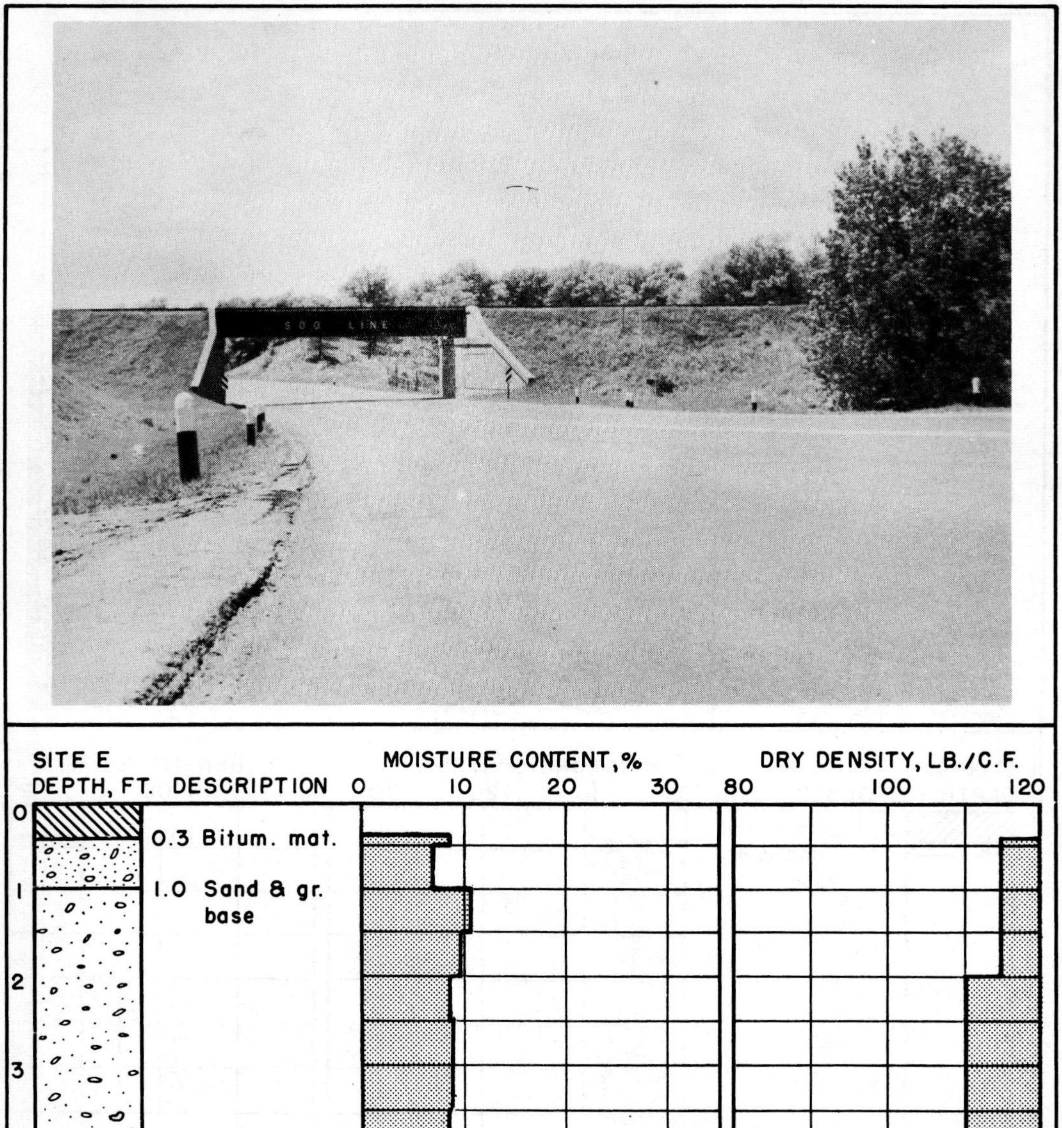

SOO LINE
SITE E
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
0
10
20
30
DRY DENSITY, LB./C.F.
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
1.0 Sand & gr. base
Sandy loam, stoney

Figure 6.

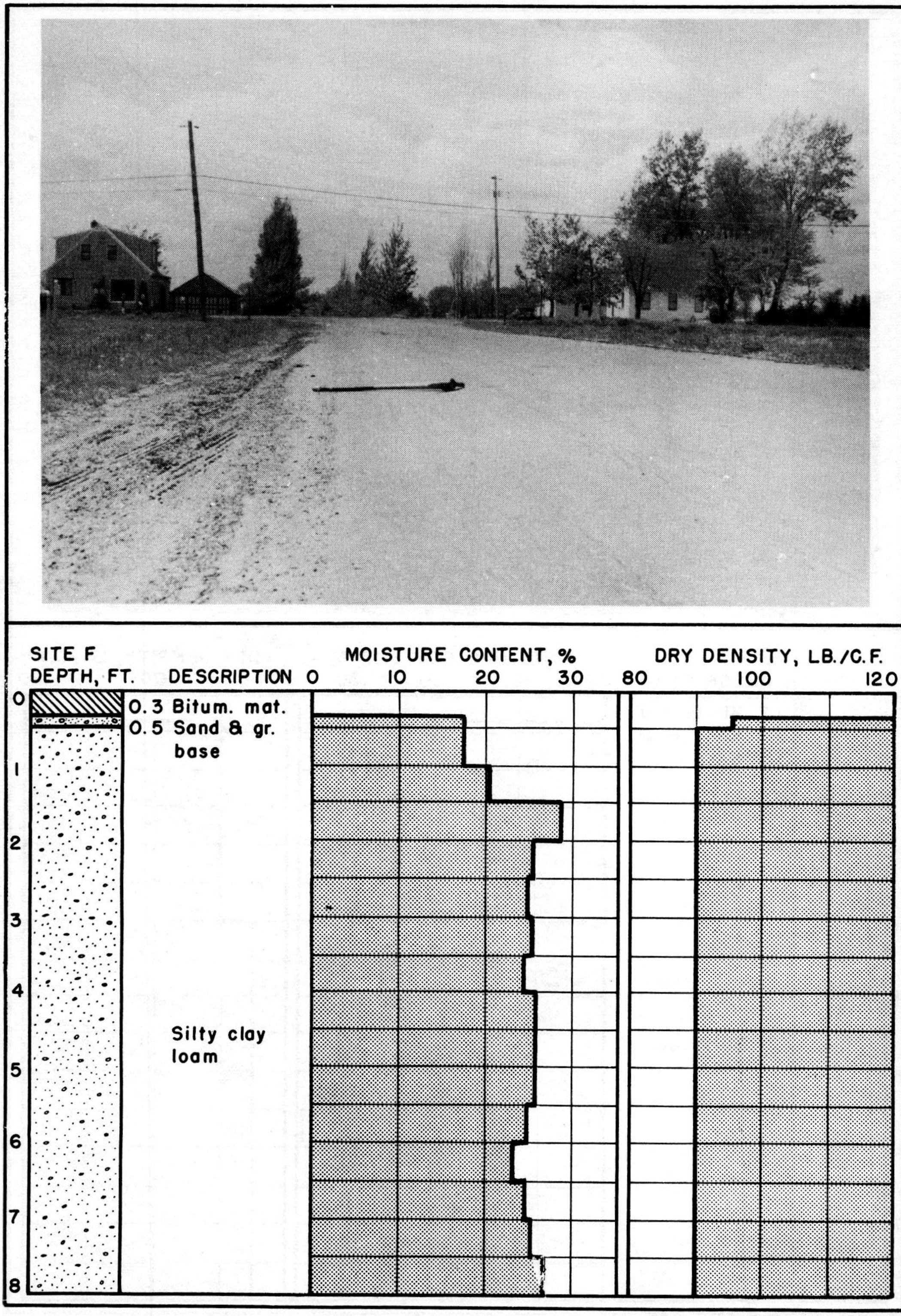
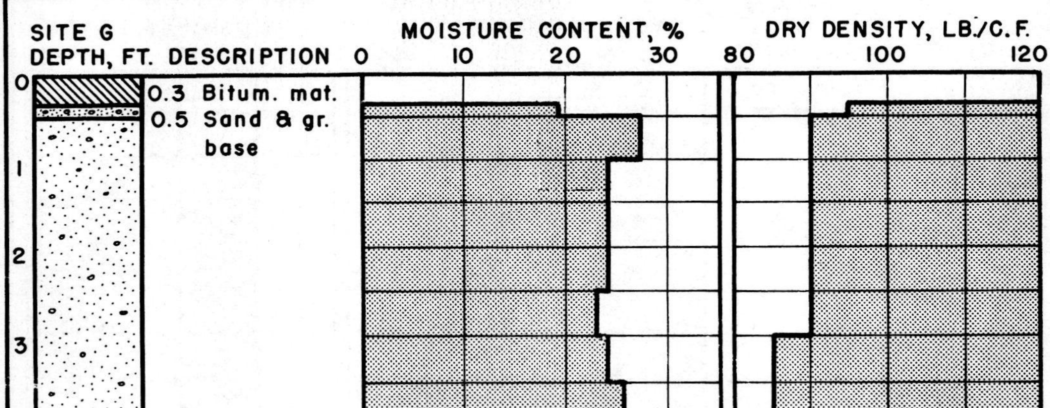

SITE F
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C.F.
0
10
20
30
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
0.5 Sand & gr. base
Silty clay loam

Figure 7.



SITE G
MOISTURE CONTENT, %
DRY DENSITY, LB./C.F.
DEPTH, FT. DESCRIPTION
0 10 20 30 80 100 120
0.3 Bitum. mat.
0.5 Sand & gr. base
Silty clay loam
0 1 2 3 4 5 6 7 8

Figure 8.

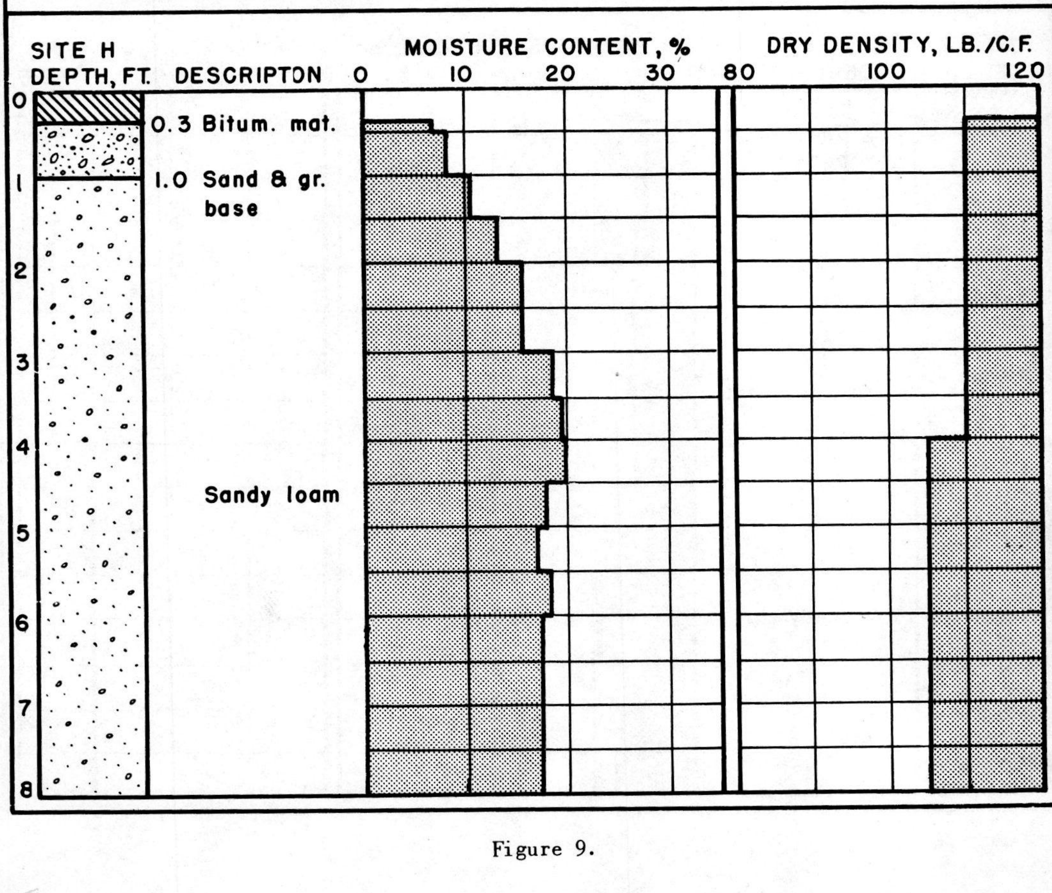

SITE H
DEPTH, FT.
DESCRIPTON
MOISTURE CONTENT, %
0
10
20
30
DRY DENSITY, LB./C.F.
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
1.0 Sand & gr. base
Sandy loam

Figure 9.

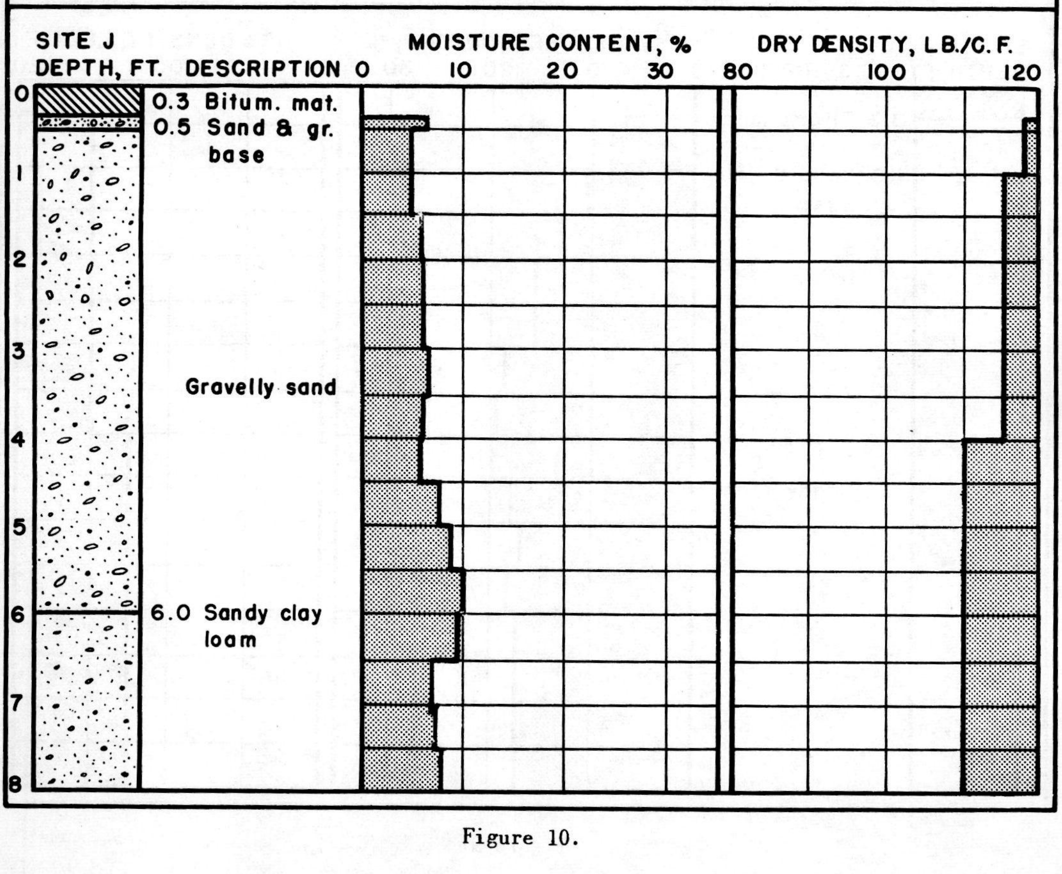

SITE J
DEPTH, FT.
DESCRIPTION
MOISTURE CONTENT, %
DRY DENSITY, LB./C.F.
0
10
20
30
80
100
120
0
1
2
3
4
5
6
7
8
0.3 Bitum. mat.
0.5 Sand & gr. base
Gravelly sand
6.0 Sandy clay loam

Figure 10.

CLIMATIC DATA

The location of the station at which temperatures were measured was about 2½ miles southwest of Site A (Figure 1). This was the residence of the investigator and was selected for that reason. A maximum-minimum recording thermometer was mounted in a location away from sun and wind yet representative of the actual air temperature. From November 1, 1953, to April 30, 1954, daily maximum and minimum temperatures were recorded at 6 P.M., from which the observed average daily temperature was determined. The location of the U.S. Weather Bureau Station was near the Twin City Metropolitan Airport, approximately 10 miles south of the location of the other recording station. Average daily temperatures at this station were based on the maximum and minimum values for a midnight to midnight period.

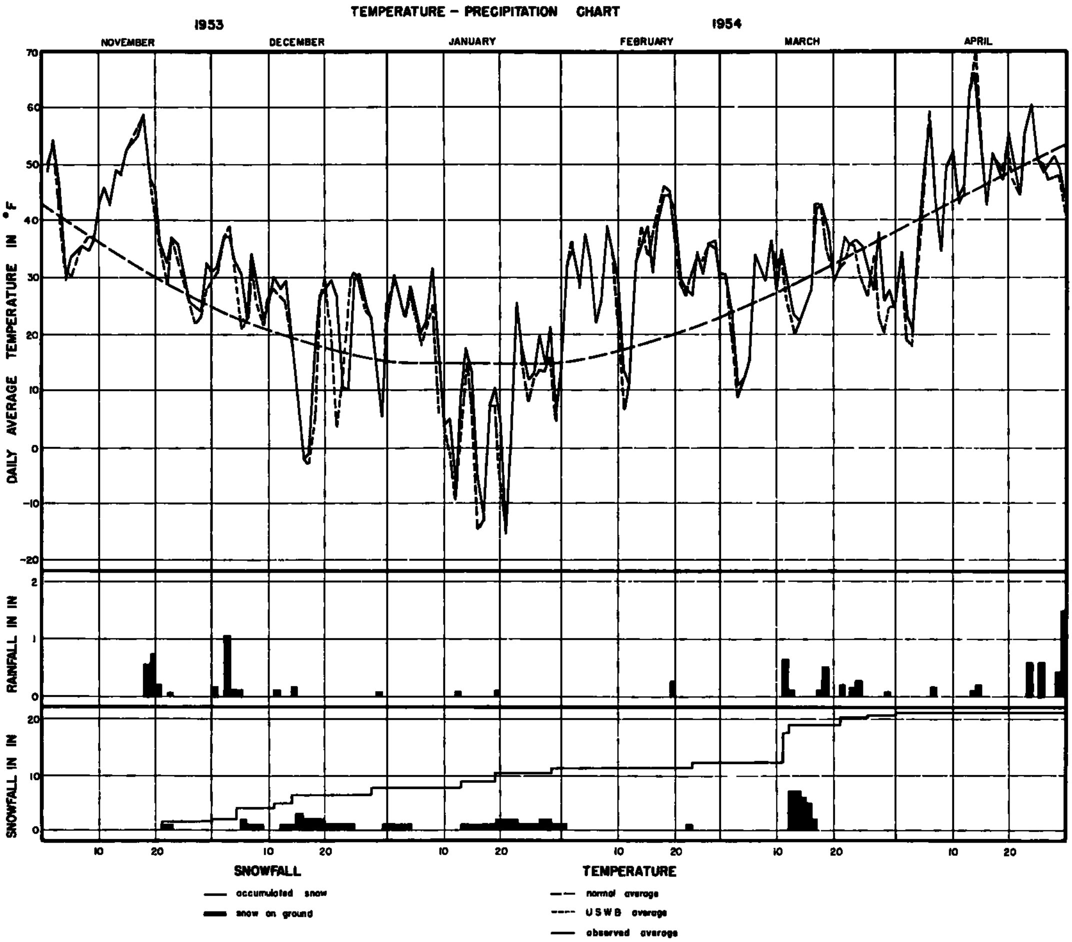


Figure 11.

It is believed that the temperatures at the airfield and frost study sites approximate each other rather closely.

The freezing season in Minneapolis according to records of the U.S. Weather Bureau normally begins about November 17 and ends about March 17, a period of four months. During this time wide fluctuations of temperature occur with highs of 60 F. to 70 F. recorded at the beginning and end of the season and lows of -20 F. during the middle of the season. The mean temperature of this 4-month period is 20 F.

Temperature and precipitation data are shown in Figure 11. The winter of 1953-54 differed from a normal winter in two ways: (1) a very-warm February and (2) a small amount of snow. Total snowfall was only 23.3 inches as compared to an average fall of 42 inches. The light snow had little or no effect on the frost study, since road surfaces are always kept clean. The abnormally warm February, however, cut off the study at an early date, since there was no increase in frost depths after the first part of February.

The measurement of cold for frost-penetration calculations is by degree-days. A degree-day is defined as each degree in any one day that the average daily air temperature varies from 32 F. The difference between the average daily temperature and 32 F. equals the degree-days for that day. The degree-days are minus when the average daily temperature is below 32 F. and plus when above. For temperate climates, where hourly fluctuations of temperature are common, a better system would be the degree-hour, which would be a more-accurate measurement of cold. This is apparent, since the average daily temperature is the basis for calculation of degree-days. Hence, a sudden rise or fall in temperature, especially near the recording time, would reflect an incorrect amount of cold, the actual average being several degrees higher or lower. The use of the degree-hour is considerably more laborious, however, and makes its value a practical question. A variation of this idea will be shown later when differences arise from time of recording.

Figure 12 is a plot of degree-days of freeze for a normal winter and for the winter of

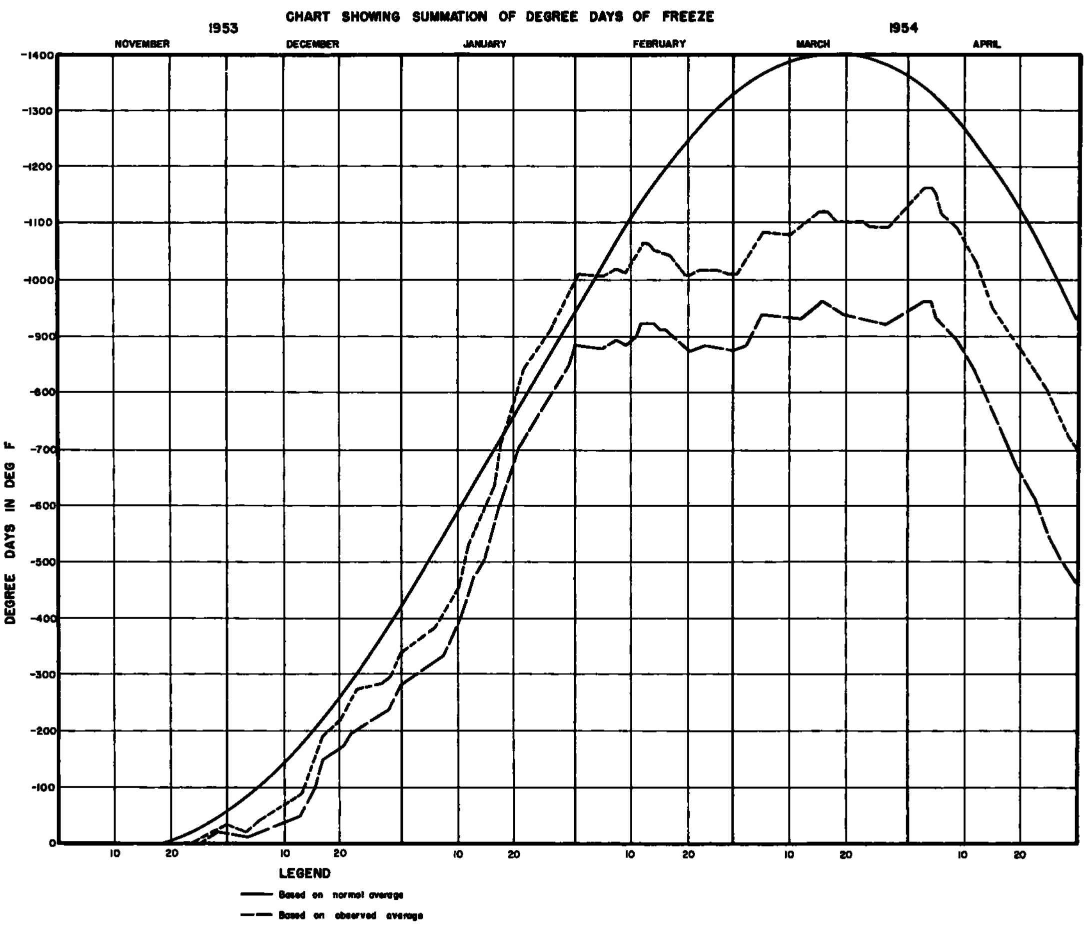


Figure 12.

1953–54 using both the temperature data near the test site (observed) and the U.S. Weather Bureau values. The differences in values for these two curves is believed to be caused by two factors which influence the daily average temperature.

The first of these factors is that of location of the recording stations, the station for observed data being a more-sheltered urban area of Minneapolis, while that of the Weather Bureau is in an exposed suburban area with less protection from climatic variations. This fact, it is believed, could account for the somewhat lower daily average temperature and a higher accumulation of cold.

The other (and perhaps most-important) factor is that of time of recording. The time of recording for the observed data was 6 P.M. while the USWB averages are computed on a 12 M. to 12 M. basis. Using the hourly temperature recordings from the official USWB climatological data sheets, the maximum and minimum daily temperatures for a 6 P.M.-to-6 P.M. period were compiled. The averages of these temperatures were computed and from these a new total degree-days of freeze was figured for the entire freezing period.

The following results were obtained:

	Total Degree-days of Freeze		
	U.S.W.B. 12 M to 12 M	U.S.W.B. 6 PM to 6 PM	Observed 6 PM to 6 PM
February 13, 1954	-1064	-982	-923
April 3, 1954	-1162	-1021.5	-965.5

The result of the change to 6 P.M. recording time is that the total degree-days using USWB data are 140 days closer to the total of observed and only 56 days greater (April 3 totals). This difference in totals is considered reasonable, being within 6 percent of the observed degree-days total. A difference of 5 percent is deemed allowable for location difference.

Closer agreement between the calculated and observed depths of frost penetration could be shown were these USWB (6 P.M.-to-6 P.M.) data used instead of the 12 M.-to-12 M.data However, the concept is a new and unproven one, and therefore the official data were used.

It should be mentioned here that although it was at first intended to include a comparison of actual and calculated depths of thaw for the same locations, the unseasonal weather in February and early March made such a study impossible. The value of such a comparison is unquestionable, its application to the fields of highway and airport engineering being perhaps greater than that of frost penetration.

FIELD OBSERVATIONS OF FREEZE

At the present time two methods of field determination of depth of freeze are in general use. These are the thermocouple and the auger, either hand or powered. Each method offers advantages and disadvantages over the other.

The hand method using the simple helical auger or the "Iwan" posthole auger is the easiest and most practical where only a few borings are to be made. This method is adapted for recovery of unfrozen shallow depth samples for laboratory analysis, since the drill hole is kept clean and actual depths of samples can be closely determined. This method is not very satisfactory for depth of freeze determination. The use of this method for frozen soil borings appears to be limited to about a foot. For this study the hand auger, helical type, was used only for determination of the depth of freeze at the onset of the cold season. The frost lines is readily found, however, from the break-through point.

The variety of power augers and boring rigs is wide and varies from hand supported, electric powered, small-diameter helical augers to large, truck-mounted, large-diameter augers and percussion drilling rigs. After the frost line had reached depths exceeding a foot, a jeep-mounted power auger supplied through the cooperation of the Minnesota Department of Highways was used. This unit had a bore 3 inches in diameter and used a continuous helical screw auger.

In practice the actual point of break through the frozen to unfrozen soil could not be closely determined during augering. Instead, a sampling spoon having a hooked lip was scraped along the walls of the hole until the spoon hooked the frozen earth. The depth was then measured to the lip of the spoon. The only disadvantage of this method was that in particularly granular soils (Sites E and J) the pebbles and rocks held firmly to the walls, so the spoon catching one of them gave the impression of being at the frost line. No difficulty was experienced in soils such as those found at Sites C, D, F, and G as the frost line at these locations was sharp and definite and no pebbles were present in the soil.

Conceivably a stoney clay, silt, or loam could occur which ordinarily would have a texture permitting easy frost-depth determination in the homogeneous state; however, due to the stoney texture some ambiguity could occur in frost determination. Manipulation by hand of scrapings from the sides of the bore was of some assistance in locating the frost line and, also, frozen soil often having the appearance of containing ice or ice crystals as compared to unfrozen soil having a moist appearance.

After stabilization of the freezing season in December, the borings were continued throughout the winter at intervals of two to three weeks.

The depths of freeze as determined are tabulated in Table 2. Depth of freeze versus time plots are shown on Figures 13 to 21, inclusive.

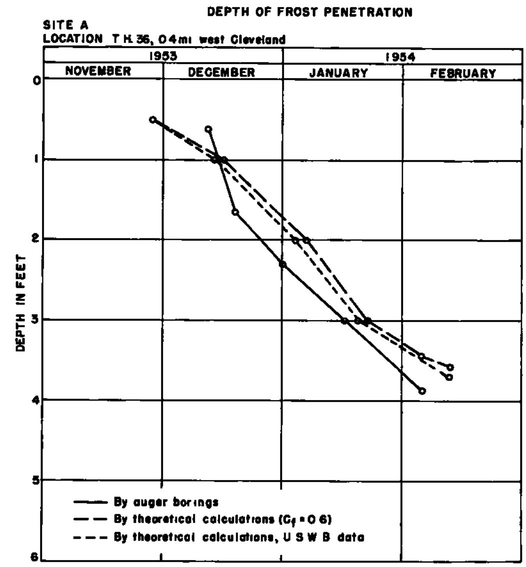


Figure 13.

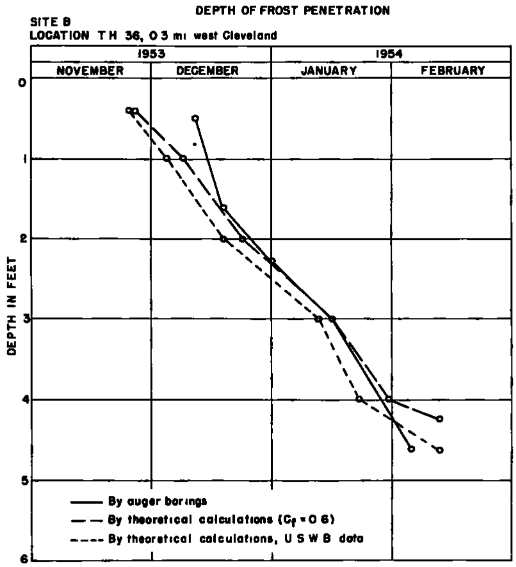


Figure 14.

CALCULATED DEPTHS OF FREEZE

The equation for calculation of depth of freeze in a layered soil has already been given as

$$F_n = \frac{L_n h_n}{24} \quad (\Sigma R_{n-1} + \frac{R_n}{2})$$

This equation was applied and depths of freeze computed. The maximum freezing index of -923 degree-days, reached on February 12, 1954, was used for all sites.

Using Site A as an example, the calculations of freeze are shown in Table 3. To calculate the degree-days to freeze the sand and gravel base, the following computation is made:

$$F = \frac{L\ h}{24} \quad (\Sigma R_{n-1} + \frac{R}{2})$$

$$= \frac{540\ (.2)}{24} \quad (.38 + \frac{.30}{2})$$

$$= 5$$

The volumetric latent heat of fusion of the sand and gravel is 540, 0.2 is the thickness of this layer, 0.38 the thermal resistance of the overlaying bituminous mat, and 0.30 the thermal resistance of the sand and gravel itself. The average resistance for the layer

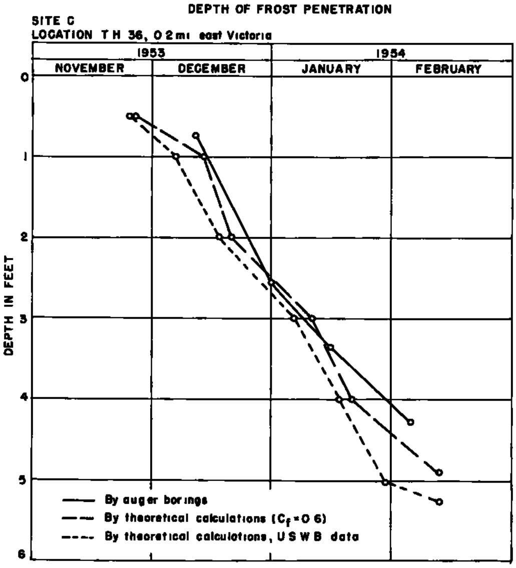


Figure 15.

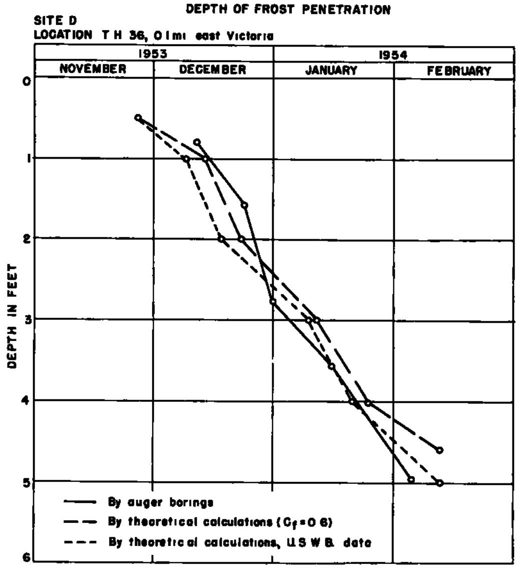


Figure 16.

during its becoming frozen is, then, 30/2. The value of 5 degree-days is a surface value. Since the relation between the surface values and air indices is given by the surface correction factor, this can be transposed into air degree-days by dividing by the value of C_f. Table 3 shows air values for C_f values of 0.6, 0.8, and 1.0.

For calculating the total number of degree-days to freeze 0.5 feet additional depth, the next calculation is made, utilizing the same equation as above, but using the L value for the loam till, a thickness of 0.5 feet, the combined result of the bituminous mat and sand and gravel base for ΣR and the average R of the 0.5 feet of loam till. The ΣF column totals the F values for all layers to that depth.

It will be noted that the maximum index of -923 degree-days produced a depth of freeze between 3 and 4 feet on February 12. In computing the depth of freeze in the final layer, it is necessary to determine the thickness of soil which can be frozen by the available degree-days of the freezing index. Since 717 degree-days are required to freeze a depth of 3 feet, (for C_f = 0.6).

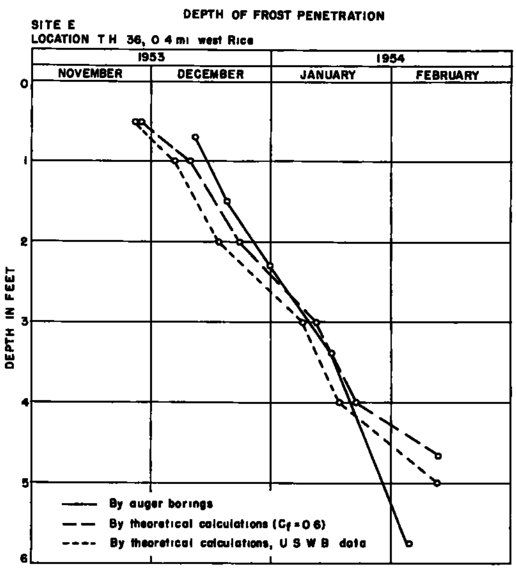


Figure 17.

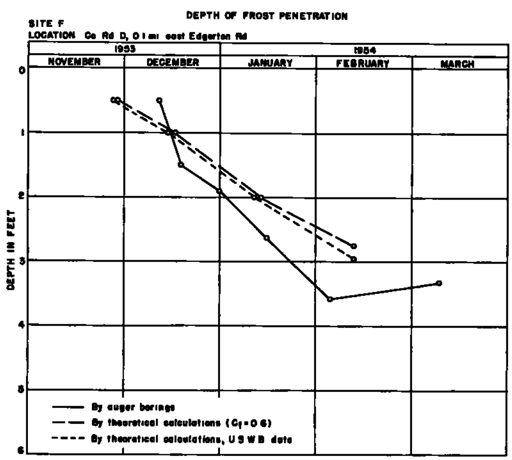


Figure 18.

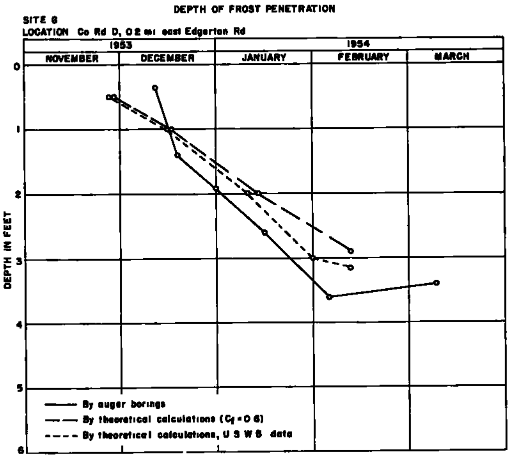


Figure 19.

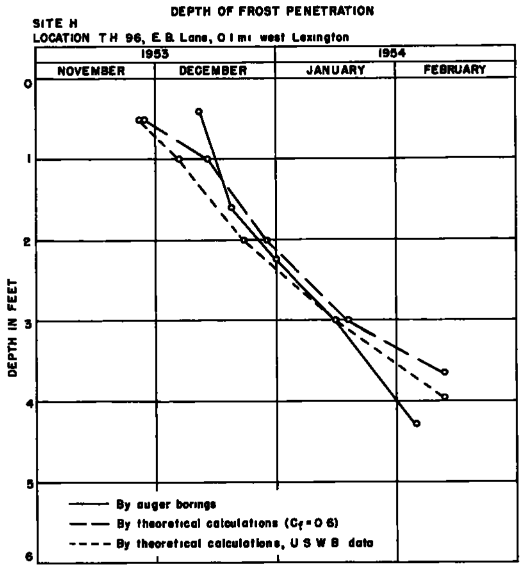


Figure 20.

$$923 - 717 = 206 \text{ degree-days.}$$

available to freeze below 3 feet.

Converting this to surface freezing index by application of the surface correction factor, C_f, gives

$$206\ (0.6) = 124 \text{ degree-days.}$$

Applying the formula again:

$$F_n = \frac{L_n h_n}{24} \left(\Sigma R_{n-1} + \frac{R_n}{2}\right)$$

$$124 = \frac{2320\ (h)}{24} \left(2.96 + \frac{h}{2(1)}\right)$$

or: $2.96\ h + .5\ h^2 = 1.28$

$$h = .40 \text{ feet.}$$

The total depth of freeze on February 12 is then 3 + 0.40, or 3.40 feet.

The same procedure was followed for all other sites in calculating the depth of freeze in the final layer. In addition, computations using the USWB freezing index of -1,064 degree-days were made and plotted on the same time-depth charts.

After the degree-days required to freeze to the various depths have been calculated, the date on which this number was reached can be taken from the curves, such as those of Figure 12. The calculated time-depth of freeze curves are then drawn. Those calculated using an air-surface correction factor of 0.6 are shown in Figures 13 to 21 inclusive.

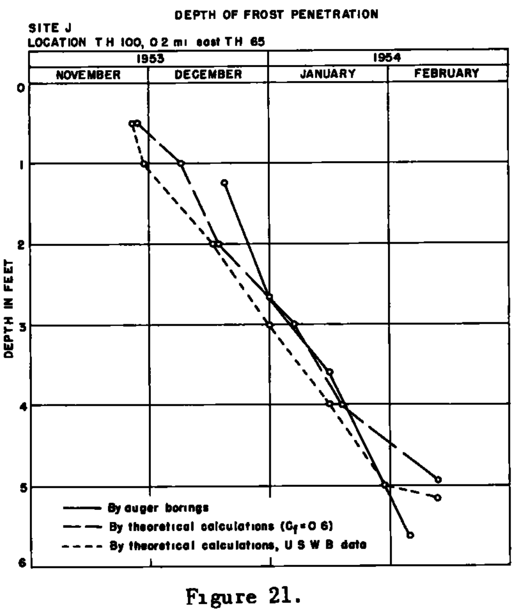


Figure 21.

EFFECT OF VARIATION IN DENSITY

Inasmuch as densities were taken at only

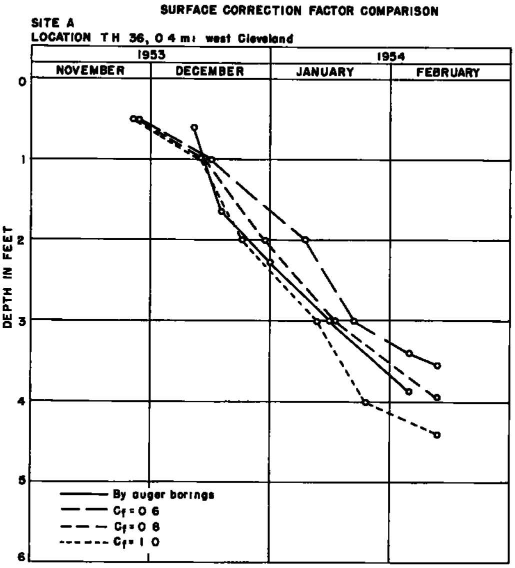


Figure 22.

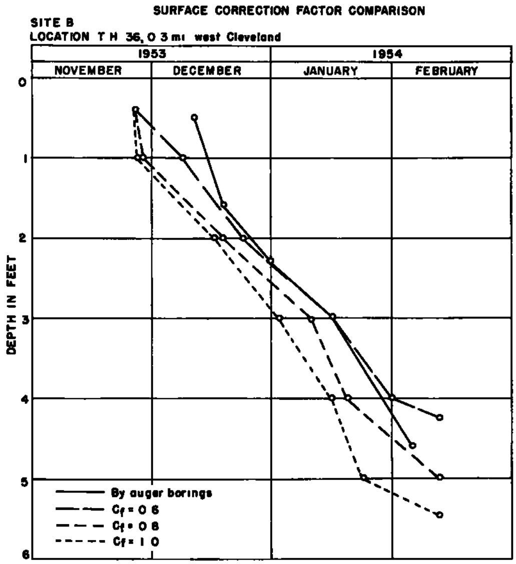


Figure 23.

two sites, A and G, to a depth of 2. 5 feet and inasmuch as this depth represents only a portion of the total frozen depth, it can be said for all practical purposes that the densities as used are assumptions which are, nevertheless, believed to be indicative of the actual field conditions.

In recognizing that they are assumptions, it is also necessary to recognize that they may be in error, some perhaps by as much as 10 pcf. plus or minus. Since the values for the other variables as used are based upon test or previously determined results, the densities used are the most significant source of error.

In the Stefan equation used here, L, the volumetric latent heat of fusion is given as

$$L = 1.434\ wd$$

where: w = moisture content in percent of dry weight
and d = dry density in pounds per cubic foot.

It is possible to determine statistically the exact effect that deviations from the assumed correct value for both moisture content and density would have upon L and hence also the

TABLE 3

CALCULATION OF FREEZING

Site A

Depth	Material	Thick. h	Density δ	w %	L	k	R	ΣR	$\frac{\Sigma R+R}{2}$	F	ΣF	ΣF/0 6	ΣF/0.8
0-0.3	Bit. Mat.	3				18	.38						
0.3-0.5	Sand and Gr Base	2	118	3.2	540	67	30	.38	53	5	5	8	6
0.5-1.0	Loam till, some gr.	.5	118	14 5	2490	1.17	.43	.68	.90	47	52	87	65
1 0-2 0	Loam till	1	118	14 5	2490	1.17	85	1.11	1.54	159	211	352	264
2 0-3.0	Loam	1	110	13 5	2130	1 0	1.0	1.96	2.46	219	430	717	538
3.0-4 0	Loam	1	110	14.7	2320	1.0	1.0	2 96	3.46	334	764	1270	955
4.0-5.0	Loam w/silty streaks	1	100	20 4	2920	1 0	1.0	3 96	4.46	543	1307	2180	1635
5 0-6.0	Loam w/silty streaks	1	100	22 0	3150	1.1	.9	4 96	5.41	710	2017	3360	2520
6.0-7.0	Loam	1	100	19.4	2780	1.0	1.0	5 86	6 36	735	2752	4580	3440

Units are as follows·
Thickness, h, in feet
Density, δ, in lb. per cu. ft (dry density)
Moisture content, w, in percent of dry weight
Volumetric latent heat of fusion, L, in BTU per cu. ft.
Thermal conductivity, k, in BTU per sq ft. per degr F, per ft. per hour
Thermal resistance, R = h/k
Degree-days of freeze, F, degree-days Fahrenheit

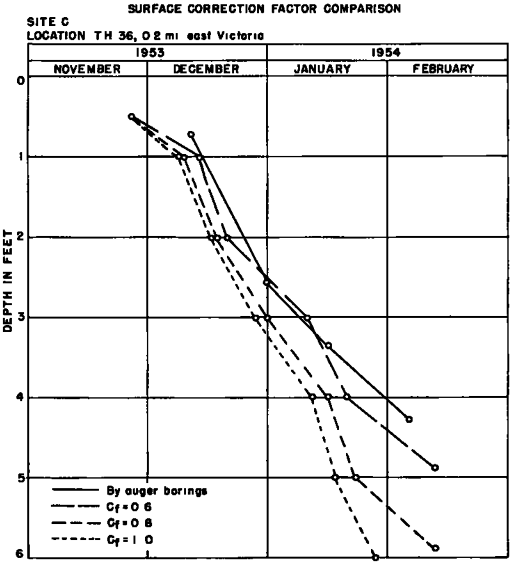


Figure 24.

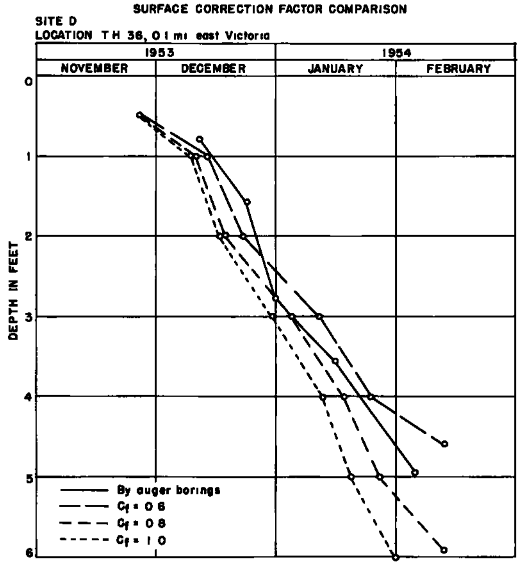


Figure 25.

effect upon the depth of freeze. The moisture contents in this study are the result of actual test and are thought to be accurate within a percentage point or two; variations in density from assumed values are more probable, and hence only these will be considered.

For this analysis the broad assumption was made that the densities could have been in error by as much as 10 pcf. plus or minus. Calculations were made for the soils at Site A representing the sandy loams, C representing the fine sands, and G representing the heavier silty clay loams. In the calculations for each site, the densities were both increased and decreased by a value of 10 pcf. and the depth of freeze calculated for the February 12 date when the air freezing index was -923 degree-days. The resultant depths are shown in Table 4.

It can be seen that a maximum variation of 0. 3 to 0. 4 feet in calculated depth of freeze would result for a variation of 20 pcf. (Plus or minus 10 pcf. from actual assumed values).

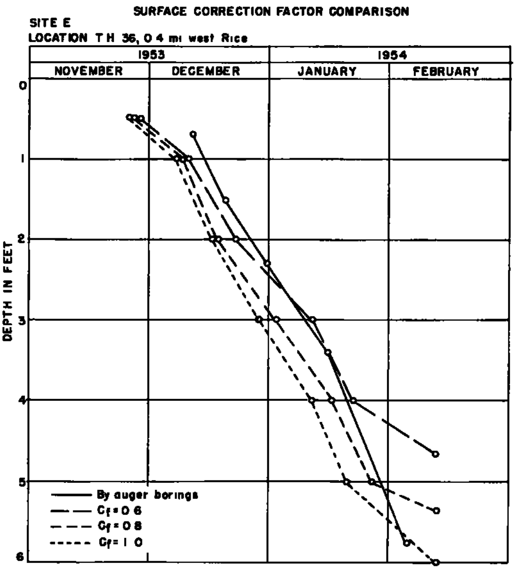


Figure 26.

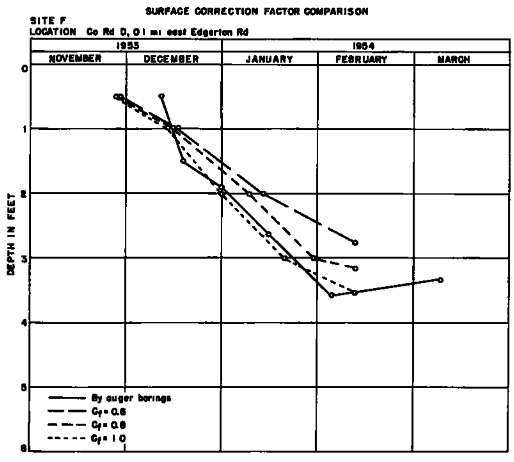


Figure 27.

COMPARISON OF CALCULATED AND OBSERVED DEPTHS

One of the important purposes of this study was to compare the actual and calculated depths of freeze and, also, to study the effect of the surface correction factor on calculated depths of freeze. The results of the former are shown by Figures 13 to 21 and of the latter by Figures 22 to 30.

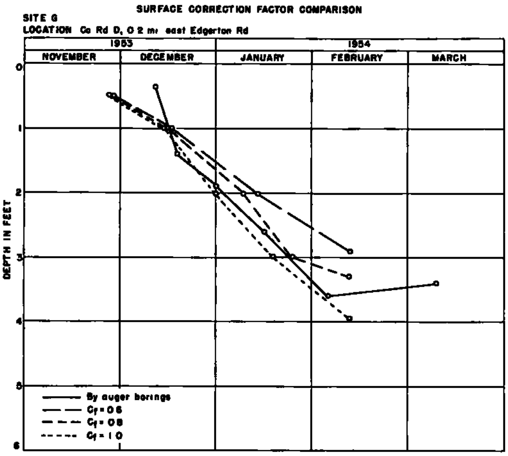


Figure 28.

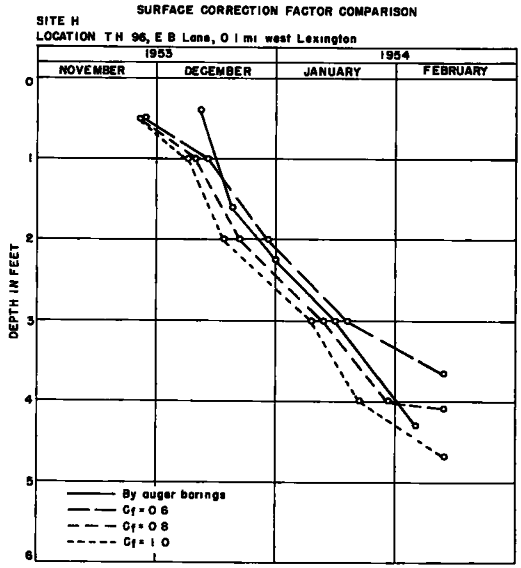


Figure 29.

Inspection of Figures 13 to 21 shows that the predicted depth of frost penetration is less than actual depth, and that the use of U.S. Weather Bureau temperature data gives closer correlation between them than does use of observed temperature data. The difference between the two calculations is not especially significant. It will be noted that in each case the curves are closely parallel, differing only in magnitude.

Using the USWB curves, the differences in observed and calculated depths of freeze on February 5, 1954, are shown in Table 5. In all cases except at Site C, the actual depth was greater than the predicted depth. This leads one to conclude that Site C may have been in error regarding the final depth. This conclusion may actually be correct, since some doubt existed at the time the borings were made.

The ambiguity regarding the depth stemmed from the fact that the soil at Sites C and D was a cohesionless sand with a low moisture content; this made determination of the depth

TABLE 4

CALCULATION OF DEPTH OF FREEZE FOR ASSUMED VARIATIONS IN DENSITY

Site	Assumed Density pcf.	Calculated Depth of Freeze, Feb. 12, 1954 feet
	10 #/c.f. greater	3.3
A	As in Figure 2	3.5
	10 #/c.f. less	3.6
	10 #/c.f. greater	4.7
C	As in Figure 4	4.9
	10 #/c.f. less	5.1
	10 #/c.f. greater	2.7
G	As in Figure 8	2.9
	10 #/c.f. less	3.1

TABLE 5

DIFFERENCE BETWEEN ACTUAL AND CALCULATED DEPTHS OF FREEZE, FEBRUARY 5, 1954.

AIR-SURFACE CORRECTION FACTOR = 0.6

Site	Soil	Measured Depth of Freeze, ft.	Difference, Measured and Calculated Depths, ft.	
			Observed Temps.	U.S W.B. Temps.
A	Loam	3.9	-0.45	-0.40
B	Sandy loam	4.6	-0.50	-0.20
C	Fine sand	4.3	+0.35	+0.85
D	Fine sand	5.0	-0.65	-0.25
E[a]	Sandy loam	5.8	-1.35	-1.10
F	Silty clay loam	3.6	-1.00	-0.85
G	Silty clay loam	3.6	-0.90	-0.50
H	Sandy loam	4.3	-0.85	-0.60
J	Gravelly sand	5.7	-1.00	-0.55

[a]Results of this site in doubt.

Note: A plus sign indicates calculated depth is greater than observed.

of the frost front difficult. Therefore in the case of Site C, the borings were repeated on February 6 in order to correct what seemed to be an obvious error, with an inconclusive result.

The differences shown in Table 5 represent those which would result from the original assumptions for the theoretical equation, i.e., an air-surface correction factor of 0.6. With the exception of the dubious frost depth determinations at Site E, it will be noted that the error in such predictions did not exceed 1.0 foot for actual frost depths which varied between 3.6 and 5.8 feet. This is considered a fairly good correlation.

TABLE 6

DIFFERENCE BETWEEN ACTUAL AND CALCULATED DEPTHS OF FREEZE

U.S.W.B. TEMPERATURE DATA

AIR-SURFACE CORRECTION FACTOR = 0.8

Site	Soil	Measured Depth of Freeze, ft.		Difference, Measured and Calculated Depths, ft.	
		January 15	February 5	January 15	February 5
A	Loam	3.0	3.9	-0.2	-0.2
B	Sandy loam	3.0	4.6	+0.6	+0.2
C	Fine sand	3.4	4.3	+0.6	+1.3
D	Fine sand	3.6	5.0	+0.3	+0.6
E	Sand loam	3.4	5.8	+0.5	-0.6
F	Silty clay loam	2.7	3.6	-0.3	-0.5
G	Silty clay loam	2.6	3.6	-0.2	-0.4
H	Sandy loam	3.0	4.3	+0.2	-0.2
J	Gravelly sand	3.6	5.7	+0.5	-0.1

Note: A plus sign indicates calculated depth is greater than observed.

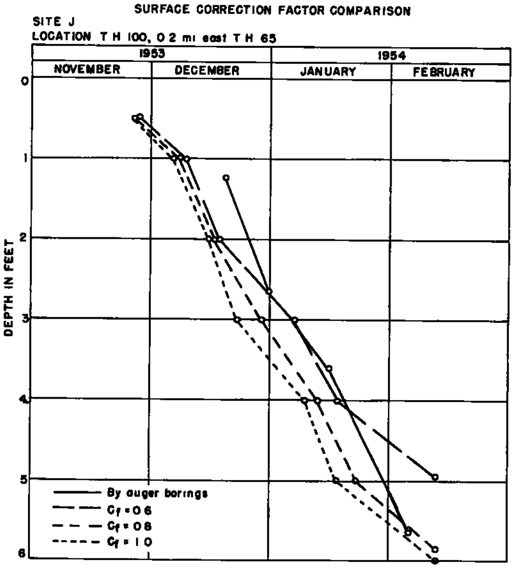


Figure 30.

At the start of the investigation it was realized that some air-surface correction factor other than 0.6 might result in a better check. As inspection of the curves of Figures 22 to 30 indicates that a value of C_f = 0.8 does give better results. Table 6 shows the differences between observed and calculated depths using C_f = 0.8 and the USWB temperature data for two different dates.

Inspection of the table shows an average difference of only about ½ foot with some calculated values being greater than the observed and others being less.

It is of interest to note that the relationship of texture and moisture content with frost penetration was the same as that shown by theoretical calculations. All other conditions being the same, the least frost penetration should occur in a fine-textured soil with a high moisture content. Sites F and G with the silty clay loam soils and moisture contents in the middle 20's had the smallest depths of freeze, about 43 inches. The greatest frost penetrations should occur for sandy soils at low moisture contents. Inspection of Table 1 shows the Sites C, D, and J had the lowest moisture contents, averaging around 5 or 6 percent.

Results show that these sites did have reasonable deep frost penetrations although Sites B, E, and H had greater depths than Site C (some doubt on accuracy of measurements at Site E has previously been mentioned). The soils with moisture contents between those of the dry sands and the silty clay loams such as the loam at Site A (15 percent) and the sandy loam at Site H (10 to 19 percent) showed intermediate depths of freeze. It is not felt that the accuracy of the frost depth determinations was sufficient to permit making more than a generalized comparison of frost depths for different textures of soils such as has been done above.

CONCLUSIONS

The study involved the collection of factual data on the soils at nine selected sites, temperature data during the freezing season, and depths of freeze at intervals. The latter were determined by means of auger borings. Utilizing the soil and temperature data, calculations of anticipated frost depths were made by a modified form of the Stefan equation. The following conclusions result from this investigation:

1. The modified Stefan equation is a useful tool for calculating frost depths. The information required for its use includes texture and moisture contents of the soil profile, density determination by tests or estimates, and average daily air temperatures during the freezing season.
2. The use of an air-surface correction factor of 0.8 gave the best agreement between the calculated depths of freeze and the observed depths. Differences in calculated and observed depths averaged about ½ foot when this factor was used.
3. Frost penetration is greatest in those soils with low moisture contents and least in those with high moisture contents. These results are obtained with the theoretical equation and were also found in the field observations.
4. For calculating the freezing index from the average of maximum and minimum 24-hour period temperatures, the time selected for readings is significant. Since midnight-to-midnight periods are commonly used, it is suggested that this be taken as standard. Further study is suggested to determine variations when another hour is selected for readings.
5. This relatively short project has indicated the desirability of other studies. It would be valuable to make measurements during a more-severe winter, with greater resultant

frost depths. The inclusion of more silt and clay soils would be helpful. Further study of the air-surface correction factor is needed. The study could be extended to include depth of thaw, since these calculations can also be made by the Stefan equation. It should also be of interest to make similar investigations in other states or areas with different climatic conditions.

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Climate in Relation to Frost Action

CARL B. CRAWFORD, Assistant Research Officer, Division of Building Research, and
DONALD W. BOYD, Climatologist, Department of Transport,
National Research Council of Canada, Ottawa

This paper attempts to illustrate the great need of correlating the main climatic factors with frost action damage. This has already been done, on a modest scale, with evidence of some success. Although adequate climatic data is available in most populated regions, much-more-concise information on actual road damage is required for a complete analysis of the effect of climate. Two specific cases of climatic influence on spring breakup are discussed.

● IN practically all of Canada and most of the United States frost action is a problem of major importance in highway and airport engineering. The term "frost action" is used here in its broad sense to include any detrimental effect on engineering works resulting from the penetration of frost below the surface of the ground.

For more than a century scientists and engineers have been concerned with the destructive effects of frost action. The advent of modern land and air transportation has added urgency to the necessary solution of the problem. This has resulted in many investigations and a voluminous literature on the subject. Fortunately this was studied, abstracted, and correlated in the admirable review of the literature on frost action by Johnson (1952).

DEVELOPMENT OF THEORIES ON FROST HEAVING

The most-significant developments in the understanding of the frost action phenomenon occurred about 25 years ago. Much good work had been done earlier but it was the publishing of Taber's treatise on "Frost Heaving" in 1929 that introduced the elements of the theories on frost action which are now widely held. Coincident with Taber's work was the equally important work of Beskow in Sweden. Although Beskow had visited America during the First International Conference on Soil Mechanics and Foundation Engineering in 1936, his work was not widely known in this country until Osterberg translated his original papers into English in 1947.

Taber postulated that a steady penetration of frost into certain types of soil would result in the formation of ice lenses which would produce heaving of the surface. In 1930 Benkelman and Olmstead, on the basis of laboratory studies and many field observations, proposed that freeze-thaw cycling of air temperature was necessary for the formation of lenses. Although Taber's theory appears to be more-widely accepted, the significance of freeze-thaw cycling has not yet been satisfactorily resolved. Heat-flow theory indicates that there is no limit to the thickness to which an ice lens may form under a slow steady extraction of heat from the ground. There is, however, some reason to believe that cycling of air temperatures, a common feature of the weather, contributes significantly to frost damage to roads (see Johnson, 1952, p. 133).

EMPIRICAL RELATIONSHIPS

There are many variables that affect the penetration of frost into the soil (Crawford 1952). There are many additional factors that will affect the frost damage to subgrades (Johnson 1952). In the solution of the problem, there are two methods of approach in which these variables have vastly different significance. In fundamental investigations into the mechanics of frost action, the investigators must attempt to consider all variables. In practical investigations or in the application of fundamental results, however, the engineer must exclude minor variables and apply averages or statistical factors to the variables of major importance.

Consider, for example, the role of the soil moisture content, a factor which is obviously important and which, to a large degree, determines the severity of frost action. In a fundamental analysis the investigator must understand the physical state of the soil water, its chemical content, its rate of movement and whether it moves as liquid or vapor under temperature gradient. In addition to its direct effects, the soil moisture will greatly influence such variables as the coefficients of thermal conductivity, specific heat, radiation, and evaporation. On the other hand, in attempts to explain frost damage in the field, average values only can be cited. Because of variability in the soil and sampling difficulties, further refinement is not possible.

This same limitation will apply to any attempt to relate climatic effects to frost action. It will be necessary to neglect completely many important factors in order to obtain workable relationships. The design of the roadway and the volume of traffic, in addition to climate and soil type, will have a considerable influence on the amount of damage which can be attributed to frost action, but these two factors are superimposed conditions and must therefore be neglected.

Casagrande, in 1931, illustrated an empirical relationship between the cumulative degree-days of below freezing air temperatures and the penetration of frost into the ground. The idea of this simple relationship had been mentioned 2 years earlier (Sourwine 1929). It is based on the premise that a reasonable measure of the magnitude and duration of cold for any one day is given by the number of degrees that the mean temperature is below freezing; the sum of these daily values for the winter season is now called the "freezing index." Approximate values of the freezing index can be calculated by using only the monthly mean temperatures or by plotting the monthly mean temperatures and finding the area between the resulting curve and the freezing line. If accurate values are needed, however, it is necessary to use daily mean temperatures for the computations.

Many subsequent investigators have noted a similar correlation. A definite empirical relationship has been established for a particular soil type under snow-free conditions (Shannon 1945).

Although it will be readily admitted that this empirical relationship is an oversimplification of complex phenomena, no improvement of the original curve has yet been possible. It is now generally referred to as the U.S. Corps of Engineers "Design Curve" (Corps of Engineers 1947). It has been suggested (Belcher, 1940; Legget and Crawford, 1952) that the rate of accumulation of degree-days of freezing air temperatures will have a significant effect on frost penetration but sufficient data are not available to establish this effect.

CLIMATIC STUDIES

In 1929 F.H. Eno presented a paper to the Ninth Annual Meeting of the Highway Research Board outlining the available climatic data which are of most importance to highway engineers (Eno 1929). Twenty-three figures were used to illustrate various features of air temperature and freezing, sunshine, wind velocity, relative humidity, evaporation, and precipitation. He emphasized the application of climatological data to drainage, subgrade and surface stability, construction operations, maintenance, snow problems, load restrictions and safety. This paper was an introduction to the importance of climate in highway engineering, but 25 years have passed since its presentation, and relatively little effort has been devoted to the promotion of its implications.

Certain features of the paper were elaborated in the discussion by J.A. Sourwine, Senior Highway Engineer, U.S. Bureau of Public Roads, who in the following year (Sourwine 1930) published results of an extensive climatic study of frost occurrence for use in highway design. His purpose was to establish a method of using recorded weather data to determine the probability of ground freezing. His method included the effects of the intensity, duration, and frequency of low air-temperature occurrences based on past records.

From the work of Bouyoucos and Petit on the required duration and lowering of air temperature to produce freezing in soil and a study of low temperature durations from meteorological records, Sourwine established 23 F. as the "critical initial air temperature" for freezing of the ground surface. He then assumed a depth of 3 inches as the "depth below which ground freezing becomes a problem for consideration in highway design." Again referring to the work of Bouyoucos and Petit, he established 26.4 F. as the "absolute minimum soil temperature coincident with the inception of ground freezing," based on the required super cooling of the soil and corrected for duration of cold period as shown by meteorological records.

From the field observations of Bouyoucos, 16 F. was found to be the "average minimum air temperature equivalent to a soil temperature of 26.4 F. at a depth of 3 inches. From the records of several stations of the U.S. Weather Bureau, Sourwine analyzed all periods during which the air temperature fell below 23 F. (the critical initial air temperature for ground freezing) and found that the absolute minimum temperature during any period was on the average 13 F. colder than the minimum temperature occurring with 5 percent frequency.[1] From this he reasoned that, since 16 F. is the air temperature at which ground freezing begins at a 3-inch depth, 3 F. represents a "critical absolute minimum air temperature" coincident with 5 percent frequency of ground freezing at a 3-inch depth.

He then compared values of monthly average daily minimum temperature and absolute monthly minimum temperature for four stations and found, by coincidence, a critical value of 23 F. for the "lowest monthly average of daily minimum temperature" which he assumed to be a critical design value for highway ground freezing.

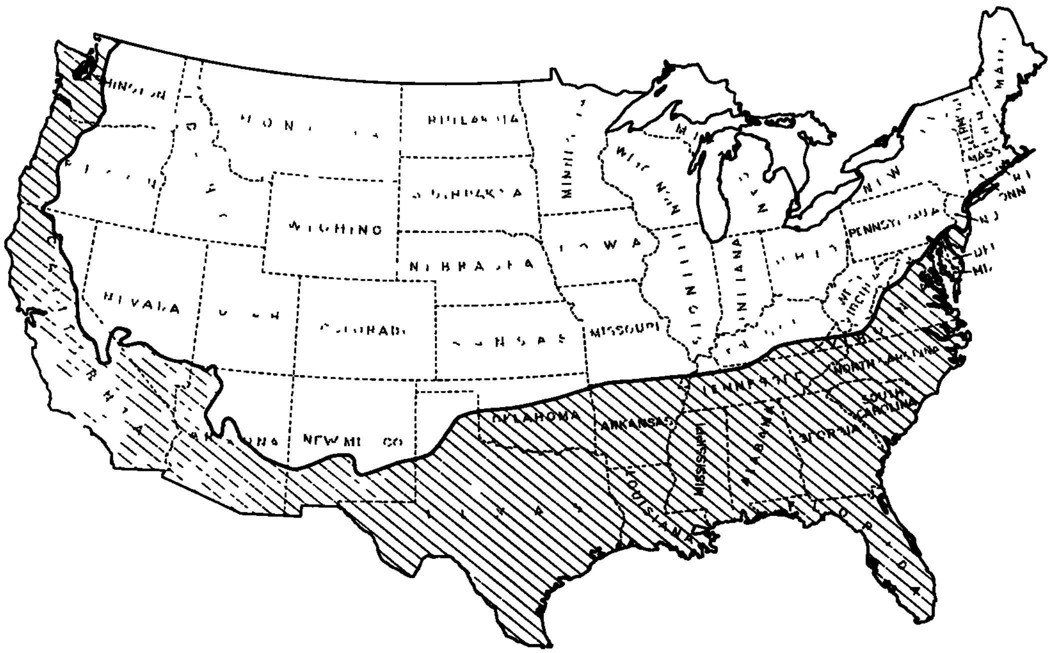

Figure 1. Critical index line for highway ground freezing (Sourwine, 1930).

Sourwine extended his initial work to a study of the effect of duration of cold periods. He studied records over a 15-year period for 35 stations near the 23 F.-minimum-air-temperature isotherm and determined in each case the cold period duration in degree-hours which occurred in 5 percent of all cold periods. After considerable study, a "degree-hour-index" of 900 for 5 percent frequency occurrence was chosen to represent the danger line for highway ground freezing.

Combining these two studies, Sourwine plotted a "critical index line for highway ground freezing" on a map of the United States, Figure 1, which, on the basis of weather records alone, separates territory relatively safe from highway ground freezing from territory in serious danger of Highway ground freezing. A portion of the territory in serious danger,

[1]This "5 percent allowable frequency" is arbitrarily defined to mean that when we consider for any locality all cold periods sufficient to cause freezing of average surface soil, ground freezing below 3 inches in depth may occur one time in 20. A frequency of more than 5 percent, or more than 1 period in 20, we designate as "objectional frequency."

according to temperature data, was modified to be a region of doubtful danger due to the nature of its winter (December to February) rainfall. This region, mainly in the south central United States, has less than 2 inches of average winter precipitation, with a lowest monthly average of daily minimum temperature between 10 F. and 23 F.

This review of Sourwine's work is presented because it is the most-extensive analysis of weather records in relation to frost action known to the authors. The purpose of the study was to determine, on the basis of meteorological data only, the approximate souther boundary of probable highway frost damage. It does not include important intrinsic variables, such as soil type, soil moisture content, density or surface cover; but, to a degree, the objective was accomplished and the approach encourages further study for relat purposes.

RECENT CLIMATIC STUDIES

In recent years many studies have been made relating climate and soil temperatures. From these studies there resulted some general relationships between climate and frost action, but few specific correlations have been evolved. It was encouraging to the authors, therefore, to review the work of Dolch (1952) who, working at Purdue University, attempted to combine temperature and precipitation data to explain variations in the sever ity of spring breakup of roads.

Using average values for about seventy stations in Indiana, he plotted monthly mean temperature, monthly total precipitation and normal precipitation in relation to time, for 16 winter seasons. Using smooth curves through these monthly average points he computed the "freezing index" (area between actual air temperature and 32 F.) and a "pre-

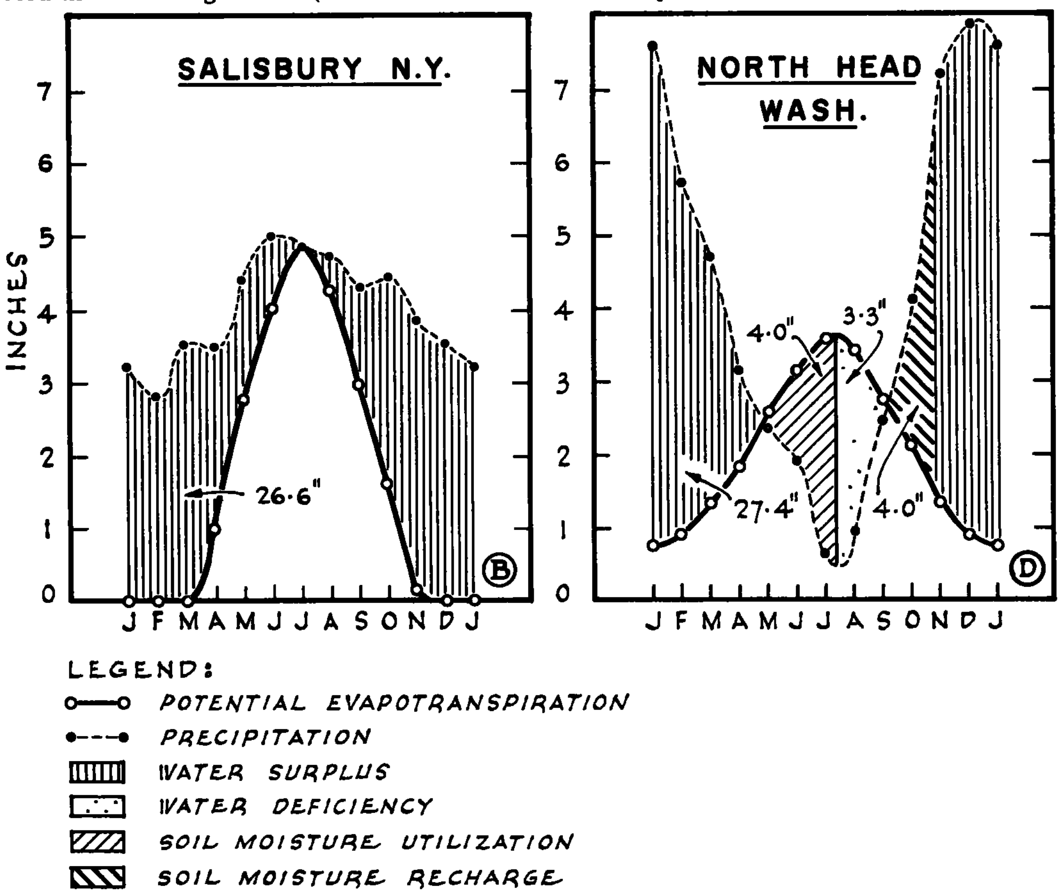

Figure 2. Extreme climatic variation (Thornthwaite, 1948).

cipitation index" (area between normal and actual precipitation during the 30-day period before freeze-up) for each winter season.

Figure 3 shows similar curves for Calgary, Alberta, during the winter of 1951-52.

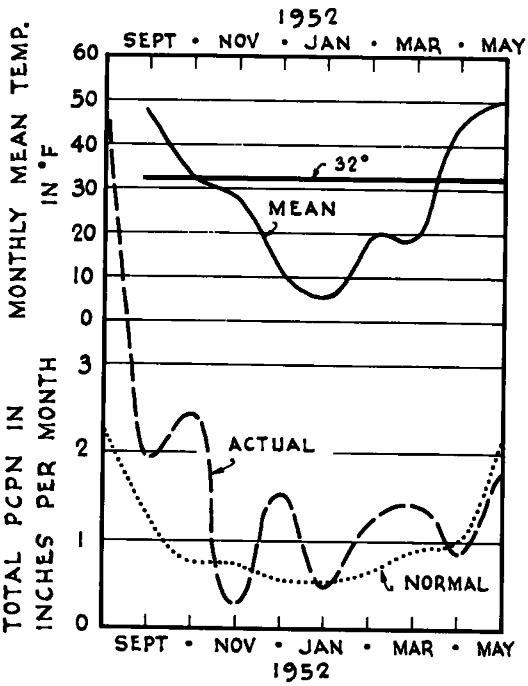

Figure 3. Calgary temperature and precipitation.

Though various combinations of temperature and precipitation were tried, the best correlation between actual spring breakup of roads and weather was found using the product of freezing-index and the 30-day precipitation index as a measure of the severity of road breakup.

Dolch recognized that freezing characteristics alone would not indicate the potential severity of spring breakup and his data certainly support this view. (One of the most-severe breakups followed a winter during which the approximate freezing index for the state was only 8 degree-days.) He also concluded, from his weather data, that snowfall and freeze-thaw cycles had no bearing on the severity of spring breakup.

There are certain disadvantages in using a precipitation index such as that used by Dolch. For example: the use of departure from normal precipitation restricts a broad application of the data, and further, the precipitation during a particular period may not be proportional to the amount of moisture in the soil at the end of the period. The amount of soil moisture depends not only on precipitation and soil type but also on other weather elements.

In a general analysis of the problem, the engineer can turn to climatology and to agricultural soil science for further assistance. Thornthwaite (1948) has pointed out that precipitation alone does not indicate whether a climate is moist or dry. It must also be known whether precipitation is greater or less than the water needed for evaporation and transpiration. Where precipitation exceeds water need, the climate is moist; and where it is less than water need, the climate is dry.

The combined evaporation from the soil surface and transpiration from plants is termed "evapotranspiration." The amount of water that would evaporate and transpire if it were available is called "potential evapotranspiration." Thornthwaite points out that evapotranspiration and precipitation, representing flow of moisture to and from the atmosphere respectively, are equally important climatic factors. Evapotranspiration can be measured only with considerable difficulty, and potential evapotranspiration must be determined experimentally.

Since the determination of potential evapotranspiration is so difficult, it was necessary to establish a relationship between potential evapotranspiration and other weather elements. This was done by Thornthwaite, and although the relationship is entirely empirical, it has been found to be satisfactory. Computed values can be obtained from temperature records and latitude.

The annual potential evapotranspiration ranges from more than 60 inches in the southern part of the United States to less than 18 inches in the western mountains; it varies greatly from summer to winter. Along most of the Canadian border it is less than 21 inches. In southern Ontario it ranges from 20 to 26 inches. (Sanderson 1950).

Monthly or daily values of potential evapotranspiration and precipitation can be used to keep an account of the amount of moisture stored in the soil, and of any surplus or deficit.

It is assumed that a certain amount of rainfall can be stored in the soil near the surface and any amount in excess of this quantity will run off or be used in recharging the ground-water table. On the other hand, when the storage has been exhausted, a deficit will occur due to the potential evapotranspiration.

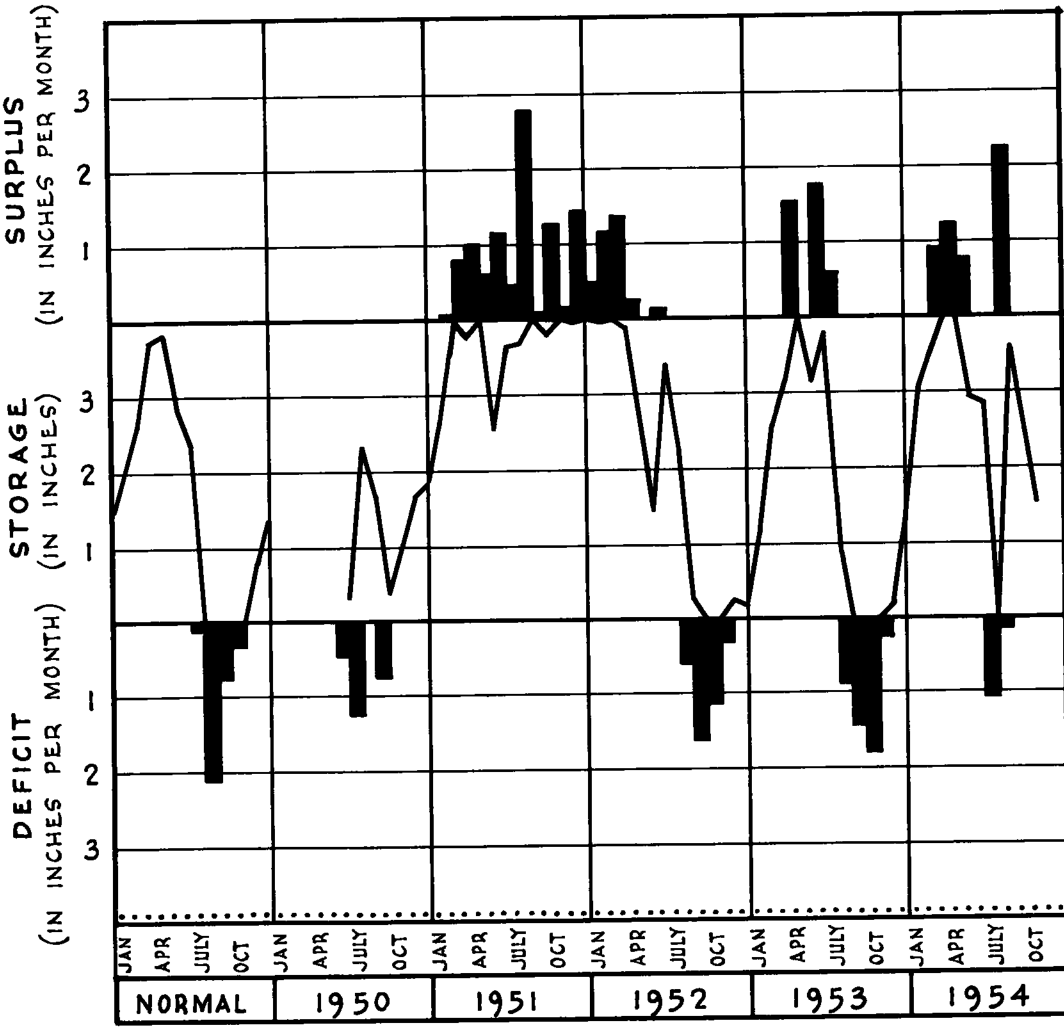

Figure 4. Soil moisture at Calgary.

TABLE 1
WEATHER DATA, CALGARY, ALBERTA

Winter	Freeze Thaw Cycles	Date of Freeze-up	Freezing Index (degree-days)	30-day Precip (Inches)	Dolch Breakup Index	Soil Moist Storage (Inches)	Modified Breakup Index	Condition of Road During Spring
1948-49	13	Nov 16	2421	0 37	896	0 30	726	
1949-50	17	Dec 2	2889	0 01	20	0	0	
1950-51	14	Nov 5	2493	1 22	3041	0 94	2343	Worse than normal
1951-52	15	Oct 15	2313	2.16	4996	3 50	8789	Worst on record
1952-53	18	Nov. 14	1143	Tr	6	0	0	Worse than normal
1953-54	15	Nov 17	1824	0 31	565	0.24	438	Better than normal
6-Year Av	15	Nov. 11	2180	0 68	1589	0 88	2049	

It is recognized that the storage capacity will vary with soil type and distribution of root systems. Thornthwaite states the "except in areas of shallow soil the water storage capacity available to mature plants with fully developed root systems varies around a mean that is the equivalent of about 10 centimeters, or 4 inches, of rainfall."

For purposes of illustration two extreme climatic cases are shown in Figure 2 (from Thornthwaite 1948). It is seen that at Salisbury, New York, no water deficiencies occur during the average year. At North Head, Washington, on the other hand, where annual rainfall is slightly greater, a serious deficiency lasting more than 4 months may be expected during the average year. It seems obvious that groundwater and soil moisture conditions at these two locations will be entirely different. Furthermore, it is probable that there exists a relationship between the combination of precipitation and potential evapotranspiration and the soil moisture conditions at any location.

a) April 2, 1953.

b) April 1, 1954.

c) April 2, 1953.

d) April 1, 1954.

Figure 5. Contrast in spring breakup during 1953 and 1954.

The importance of climate to practical aspects of building research led, in 1949, to a useful arrangement between the Division of Building Research of the National Research Council and the Meteorological Division of the Department of Transport. By this special agreement the full time services of a trained meteorologist are seconded to the Division of Building Research for cooperative work on building problems while he retains the associations and facilities of the Meteorological Service. One of the first joint studies was to analyze the weather conditions which accompanied an unusual spring breakup of roads in the Province of Alberta.

EXAMPLES OF SPRING BREAKUP OF ROADS

Calgary, Alberta

During the spring season of 1952, roads in the Calgary district of Alberta suffered unusual deterioration, while roads in the Edmonton area, to the north, experienced a normal breakup. It is known that frost action in the Calgary area, a region of silty soil mantle, is always more severe than in the predominantly clay soils around Edmonton, but in this particular year the breakup was so severe in the southern part of the province that a special investigation of the weather was thought to be warranted.

TABLE 2

WEATHER DATA, OTTAWA, ONTARIO

Winter	Freeze Thaw Cycles	Date of Freeze-up	Freezing Index (degree-days)	30-day Precip (Inches)	Dolch Breakup Index	Soil Moist Storage (Inches)	Modified Breakup Index	Condition of Roads During Spring
1948-49	11	Nov 28	1269	4 17	5292	4 00	5076	
1949-50	18	Nov 17	1719	2 04	3507	2 76	4744	
1950-51	13	Nov 21	1491	3 76	5606	4.00	5964	Very Bad
1951-52	13	Nov 1	1557	1 50	2336	0 51	794	Very Good
1952-53	12	Nov 28	933	2 13	1987	4 00	3732	Bad
1953-54	14	Dec. 15	1449	2 56	3709	3.41	4941	Good
6-Year Av	13	Nov 23	1403	2 69	3740	3 11	4208	

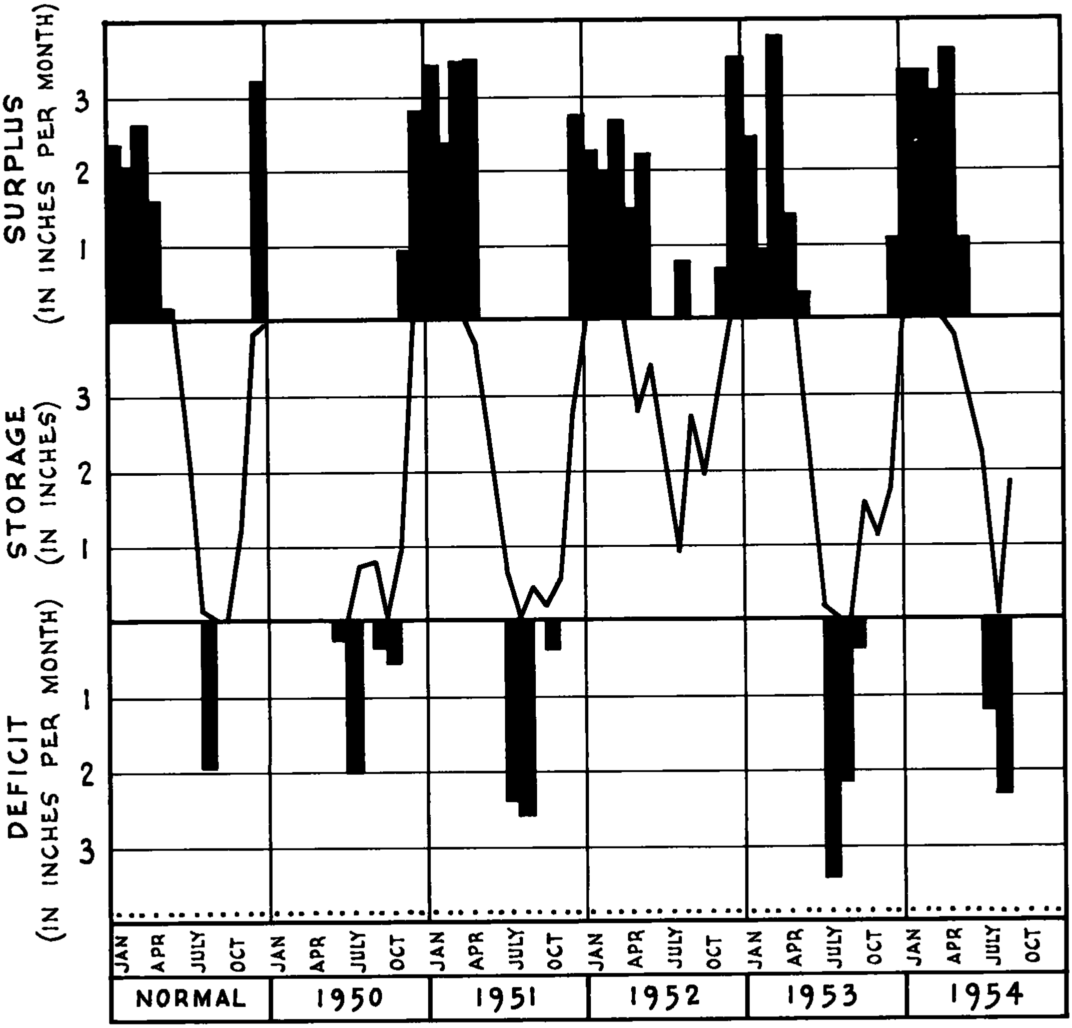

Figure 6. Soil moisture at Ottawa.

The most-difficult problem encountered in an investigation of this type is the evaluation of relative road damage, both in its variation over a wide area and in disparity from year to year. It was possible to obtain only descriptive terms, based on experience, in order to differentiate the degree of damage from year to year. Significantly, the road breakup near Calgary during the spring of 1952 is termed "the worst on record." This was attributed, in a general way, to a comparatively wet cycle in the weather which caused a general rise in ground-water levels. Study of the records does, in fact, show that this was probably the case.

In the climatic investigations, a large volume of data was assembled and analyzed in an attempt to establish a relationship between road damage and the weather. Weather records for 6 years at Calgary were studied; the results are shown in Table 1. Freeze-thaw cycles were based on daily mean air temperatures; each figure indicates the number of times during the winter that the mean temperature fell below and rose above the freezeing point. The freezing index was computed using monthly mean temperatures. This procedure is an approximation, adequate for comparative purposes but resulting in an index which is usually low.

In the correlation of road damage to weather it was considered desirable to refine the index which was originated by Dolch. Accordingly the "30-day Precipitation" is the actual total precipitation which occurred during the 30-day period before freeze-up, rather than the "departure from normal" based on graphical analysis. The Dolch breakup index, reported in this paper, is the product of the freezing index and the 30-day precipitation as outlined above. Values of soil-moisture storage were computed using Thornthwaite's theories; the value represents storage in inches at the time of freeze-up. The modified index is the product of freezing index and soil moisture storage. In the opinion of the authors this index provides a more accurate evaluation of soil moisture conditions than is possible using precipitation data alone.

For a comparison of the winter of 1951-52 with that of other years, reference may be made to Table 1. During this particular winter, the occurrence of freeze-thaw cycles was equal to the 6-year average. The freezing index was about 6 percent greater than average, but the freeze-up occurred much earlier than usual. The most-significant variations occurred in the precipitation which preceded freeze-up and in the soil moisture storage. Precipitation during the 30-day period before freeze-up was more than three times the 6-year average and soil moisture storage was more than four times the average.

Reference to Figure 4, showing soil moisture conditions at Calgary from 1950 to 1954, supports the general opinion that the Calgary area was experiencing a wet cycle. This diagram illustrates the variation of soil moisture storage on a monthly basis. The bar graphs indicate the total amounts of surplus and deficiency which occurred during each month.

Unforturately, it was possible to obtain comparative opinions of the severity of road breakup for the last four seasons only. The breakup indices for three of these years are in the appropriate order. The indices for the winter of 1952-53, however, indicate a light breakup when in fact it was worse than normal. Significantly, the freezing index for this year was only about half of the 6-year average; it was one of the mildest winters on record. Further comments on this feature will be made later.

A study of weather records for Edmonton during the period from 1948 to 1954 showed much less variation in the Dolch index (535 to 1,944) than at Calgary (6 to 4,996) during the same period. This climatic feature, together with the great difference in soil type, previously mentioned, results in greatly different average breakup conditions.

It is noted in a private communication from R.M. Hardy of the University of Alberta: "The effect of frost action is usually much slower in becoming apparent as damage to pavements than is the case in the Calgary area. In the Edmonton area, damage to pavements may still be developing as late as July by subbase failure while in contrast, in the Calgary area, pavement failure occurs almost within hours after the breakup."

An extensive survey of thirty blocks of newly paved streets (Hardy, 1950) showed that pavement damage occurred only where ice segregation took place within a depth of 18 or 20 inches of the surface. Redesign of these streets using a mechanically stabilized pit-run gravel base with minimum thickness of 24 inches has proved adequate, even during the spring of 1952.

Ottawa, Ontario

Observations of road breakup at Carleton County, near Ottawa, Ontario, during the past two spring seasons have shown the remarkable effect of weather variation on frost damage. During the spring of 1952-53, the county experienced a severe deterioration of its road system. The following year damage was confined to a few isolated frost boils.

Figure 5 shows typical road conditions at two locations during the two spring seasons. Figure 5 (a) shows a section of road that was barely passable on April 2, 1953. A few days earlier it had been completely closed to traffic. The same road is shown in Figure

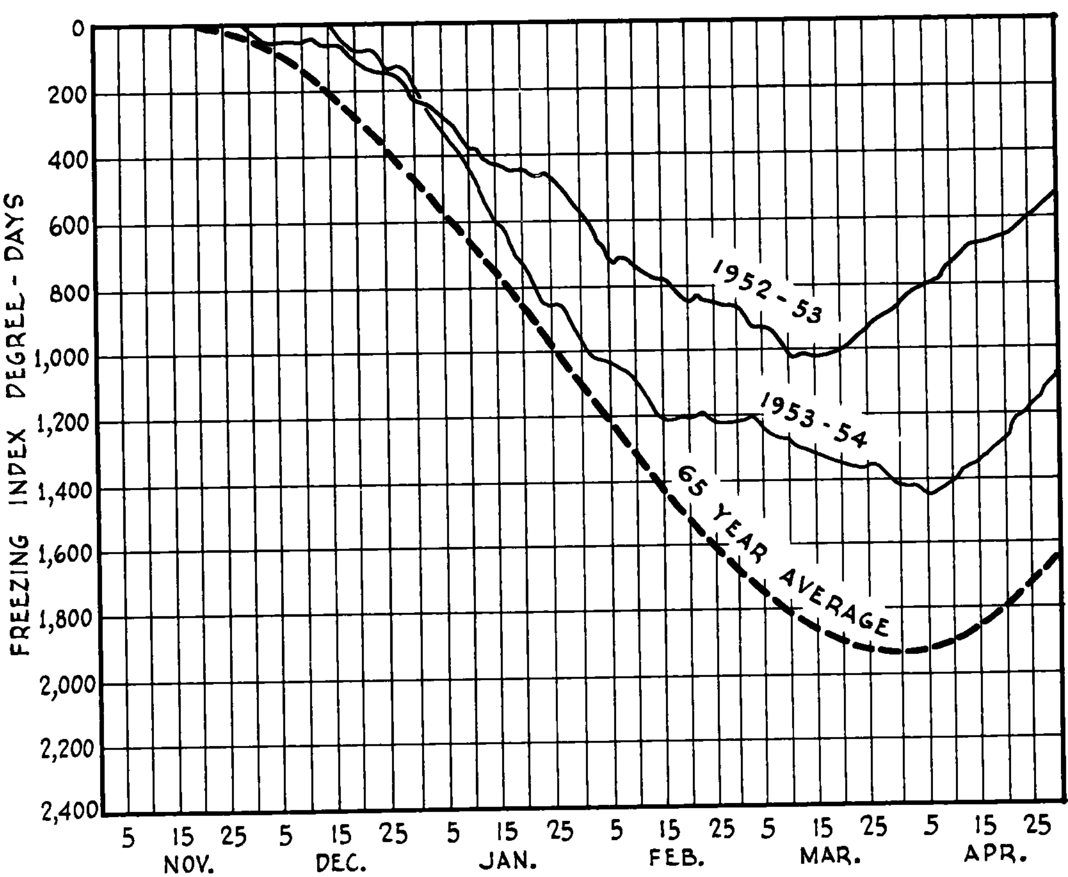

Figure 7. Freezing index at Ottawa, 1952-54.

5 (b) a year later. Although no change had been made in the road structure during the previous year, it remained in good condition. Figure 5 (c) shows a similar piece of roa that was impassable on April 2, 1953, but quite satisfactory during the spring of 1954, Figure 5 (d). This road had been treated with about a foot of pit-run gravel following th spring breakup of 1953 but is shown as a typical contrast between the two seasons.

Table 2 shows weather data for Ottawa similar to those shown previously for Calgary It was again possible to obtain an assessment of road breakup during only the last four years. During this period freeze-thaw cycles did not differ significantly. Freezing indexes were near normal, except during 1952-53, which was the mildest winter on recor Both breakup indices indicate a bad road breakup during the spring of 1951, and the mod ified index indicates a negligible breakup during the spring of 1952 and a bad breakup in 1953. To this extent the index is a satisfactory guide. But the indices intimated a bad road breakup during the spring of 1954, when in fact breakup was light. A more-detaile analysis of weather data illustrates at least part of the reason for the varying usefulnes of the indices.

Figure 6 shows soil-moisture conditions at Ottawa (according to the Thornthwaite criterion) from 1950 to 1954. Reference to Figure 6 and Table 2 will show that the soil was saturated at the time of freeze-up in 1950 and 1952, the years followed by bad breakup. During 1951-52 there was no surplus and only about ½ inch of storage during freeze-up. Although the freezing index was about 10 percent above the average, no road breakup occurred.

During 1953 a water deficiency existed early in September and storage remained less than 2 inches until early December and was still not saturated at the time of freeze-up in mid-December. These observations show substantial differences in the amount of water in the soil during the fall seasons which appear to be related to the degree of breakup during the following spring.

The seasonal variation in water content of soil is not well understood. There is no question but that in most regions it is dependent on the weather, but much correlation is necessary to establish the relationship between weather and the ground-water table. Measurements of ground-water levels near the Building Research Centre in Ottawa show values which are probably typical of the region. At the time of freeze-up in 1952, the ground-water table was less than a foot below the surface; in 1953 it was about 6 feet below the surface. Although no other actual records are available, the Water Resources Division of the Geological Survey of Canada confirms this trend throughout the Ottawa area.

The accumulation and decrease of degree-days below freezing during the 1952-53 and 1953-54 winters are compared with the normal in Figure 7. Several anomalous features are evident from these curves. During the first month of the 1952-53 winter season, degree-day accumulation was extremely slow; a situation thought to favor ice lensing. Later, two significant periods of slow freezing occurred and the winter ended about 2 weeks earlier than usual.

The winter of 1953-54 began quite late but the air temperatures during January and the first two weeks of February were exceptionally cold, causing rapid accumulation of degree-days of freezing. This was followed by about 3 weeks of very mild weather, with mean daily temperatures near freezing. Ground temperatures, during this mild period, were very near the freezing point in the upper few feet indicating a softening of ice formations and possibly some redistribution of the soil water. It is thought that the rapid accumulation of degree-days of freezing early in the winter and the sustained mild weather which followed were responsible for the absence of road damage and for the failure of the break-up indices to assess the situation adequately.

COMMENTS

In considering the general problem of frost action as it affects the performance and maintenance of highways, two problems are paramount: pavement heaving and loss of strength during spring thaw. Obviously freezing air temperatures, a frost-susceptible soil, and water are prerequisites for road damage. The soil can be eliminated from this discussion of the effect of climate; since, in general, it is affected by climate only in the pedological sense and so can be assumed to remain in the same physical and mechanical state from year to year. Air temperature, however, needs close attention.

In regions of doubtful freezing danger, climatic studies, such as those of Sourwine, are helpful in predicting the possible frequency and degree of damage. In colder regions, the rate of accumulation of freezing temperatures probably has great effect on the depth of frost penetration and on the degree of ice lensing in the soil. Consequently, it affects the amount of heave and the potential loss of strength during break-up.

The rate at which the frost "leaves the ground" must also be of prime importance, insofar as loss of strength is concerned. A rapid thaw is usually followed by severe breakup, due to the quick release of excess water within the upper part of the soil which cannot escape quickly, due to the frozen layers below. The quantity of water released during breakup is, of course, dependent on the amount of heaving.

Although in this study the freezing index is incorporated in the indices of road breakup, it is evident that factors other than cold weather are extremely important in causing road failures during spring thaw, except in regions where the freezing index is occasionally equal to zero. It is submitted that two features in particular greatly influenced this notable difference in breakup. These two features are the amount of water available and rate of

accumulation and decrease of degree-days of freezing. The data quoted support this vie
Dolch cited cases of severe breakup following very mild winters. At both Calgary an Ottawa following the exceptionally mild winter of 1952-53, a "worse-than-normal" road breakup occurred. Therefore, a wide collection of case histories must be made if the fu effect of air temperature is to be understood.

The last, and probably most-important, variable is water. It is a much-more-compl variable than temperature, since it occures in three phases and comes in variable amou the utilization of which depends largely on temperature, drainage, vegetation, and soil t Since the amount of water available affects the degree of frost lensing, the position of th ground-water table is important. The ground-water table, in turn, depends on the above variables.

Although Thornthwaite's method of dealing with water utilization was developed for cl mate classification, it can probably be used, in a general way, to assess soil moisture and ground-water levels for engineering purposes. The quantitative method of separatin from precipitation the amount of water used by evaporation and transpiration is an impo tant development.

As previously mentioned, the problem of assessing road breakup is difficult. Adjectives, as used in this report, are not adequate. Perhaps a breakup classification can be developed which would allow more-direct comparisons with weather. Other possible me ods of evaluation include plate-bearing tests, the cost of repairs, or a combination of m and duration of road breakup. Plate-bearing tests are expensive and, therefore, not ma extensively. But it has been suggested by N. W. McLeod that this type of evaluation may best be carried out using the Benkelman Beam Pavement-Deflection Indicator. Recent tests with this instrument (Carey 1954) have shown much promise in this regard. An example of road breakup evaluation based on pavement maintenance costs has been given b Otis (1952), but in the case of the Carleton County study this method was quite unreliabl

CONCLUSIONS

This paper is an attempt to draw attention to the necessity of studying climate to gain more-complete understanding of frost action in soils. Owing to the fact that an entire ye is necessary for the collection of each piece of evidence, relatively little field data have been presented. The following tentative conclusions may, however, be stated:

1. The partial success of Dolch's index in assessing spring breakup at Calgary and Ottawa indicates that a combination of air temperature and precipitation can be used as a index of spring breakup of roads.

2. The somewhat-greater success of the modified index indicates that Thornthwaite's formulas lead to a better evaluation of the moisture available for frost action than does p cipitation alone.

3. There is no evidence that very cold winters result in severe breakup. In fact, in northern regions where annual freezing always occurs, the slope of the freezing index cu probably has more effect than the absolute value of the index on frost damage to roads.

4. This investigation has revealed no evidence that the number of freeze-thaw cycles air temperature affect the degree of frost damage to roads.

In the general approach to understanding frost action, the phenomena must be studied in the laboratory as well as by studies in the field. Problems such as the role of moistu movement and the criteria for frost-susceptible soils require fundamental studies. But even if these phenomena were completely understood, highway engineers would still be baffled by the weather. To understand the phenomenon of frost action in the field consideration of the effect of climate is essential. With such an understanding, it can be hoped that accurate forecasting of the amount of heave, the duration and the degree of spring breakup and thus improved roadway design will follow.

ACKNOWLEDGMENTS

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Theoretical Basis for Frost-Action Research

A. R. JUMIKIS, Associate Professor of Civil Engineering,
Rutgers University

The purposes of this article are to (1) direct the attention of the highway designer, builder, owner and user to the adverse effect of the freeze-thaw phenomena on the performance of street, highway and runway pavements; (2) indicate the need for effective frost action research; and (3) stimulate interest among research workers, highway engineers and others in the importance of frost action research in highway and airport engineering, with a view to improving technical service of the nation's transportation system.

For these purposes, a theoretical basis as an imperative and indispensable tool to work with is here briefly reviewed, the physical concepts involved are mentioned and the principal factors to be studied outlined.

The evaluation of the geotechnical and thermal properties of the materials in question, and their application to a practical problem on a scientific basis, with or without modifications, may, it is believed, help to suggest to the investigator an approach to a satisfactory solution of almost any frost action research problem.

The theories displayed (which are based on the discipline of heat transfer) are by no means the ne plus ultra for this field of research. However, based on the present state of knowledge, they can be considered as representing a theoretical basis for frost action research in soil mechanics (géotechnique) and highway engineering. As our knowledge increases, theories can then be modified and corrected.

● THIS article is a summary of thoughts resulting from the writer's reading of the pertinent literature, from his observations in practice through many years, from experienc gained in research in the fields of soil mechanics, foundation engineering, highway engineering and the building of airports, and from teaching undergraduate and graduate courses in these and other related fields, as well as from practical consulting work.

For several years now highway owners and engineers have become increasingly conscious of the kind and amount of damage to highways caused by frost action. Both contir uous (seasonal) freezing followed by thawing (in northern regions), and periodic (cyclic) freezing and thawing (in more temperate regions) of the pavement-supporting soil, parti ularly in areas where roads are kept free of snow in winter, create difficulties for traff and maintenance. These difficulties consist of differential frost heaves, development of boils, and damage or destruction of pavements, making driving conditions dangerous. Repairs of such damage usually cost huge sums.

Observations made of existing roads in the Atlantic states showed that many miles of roads of all types, which under normal climatic conditions have served well for many years, were damaged seriously in the severe winter of 1947-48. From experience with road performance in this particular winter, one concludes that engineers are really wor ried as to how to combat effectively frost on highways and airports.

FACTORS CONTRIBUTING TO DAMAGE

Some of the main factors contributing to the damage to roads by frost action include: (1) climatic conditions, for example, freezing temperatures and their duration; (2) the soil itself, particularly silts; (3) precipitation; (4) drainage conditions; (5) soil moisture; (6) position of ground-water table; and (7) vehicular loads.

Great damage occurs particularly in thawing periods, when improper soil below a pavement under unfavorable conditions becomes soft from saturation by melting ice water without proper drainage. The consequence is the loss of bearing capacity of the soil, resulting in damage to pavements and causing so-called spring breakup. The inte relationship of the various factors involved in damaging roads is complex. A variation any one of the factors involved influences to a greater or lesser extent the others, the properties of the soil, as well as the whole thermal system, soil-moisture-temperature.

These facts revive and focus more sharply the two well-known fundamental requirements for a good road: (1) a solid foundation and (2) good drainage. These requirements were understood by the early Roman and Incan road builders, and still are to be honored. The idea could be somewhat extended by saying, "A road must be dry even in wet weather."

By drainage is here understood taking care not merely of the water on the road surface, from floods and groundwater, but also of the melting water from the ice in the thawed pavement-supporting soil. In practice, drainage is often overlooked. One can appreciate its importance simply by remembering that excess moisture in the pavement-supporting soil reduces the bearing capacity of the soil.

NEED FOR FROST ACTION RESEARCH

Although the development in highway transportation in the past 30 years has made considerable progress, relatively little progress was made in arriving at more effective methods for evaluating the thermal system of soil-moisture-temperature in connection with the suitability and performance of highways under freeze-thaw conditions. Because of the complex nature of this thermal system, engineers in their efforts to solve a problem involving temperature differences, heat transfer and moisture migration in soils, relied for the most part upon a practical approach or depended upon mechanics alone. Relatively little attention in this matter has been given to a theoretical basis or approach.

Now, we often realize that a satisfactory explanation of a thermal problem cannot be found merely by experimentation or by way of mechanics alone. We must admit the fact that we have not arrived at a point where so-called practical methods in such researches have developed to a sufficiently refined point to give satisfactory solutions pertaining to pavement performance under freeze-thaw conditions.

If we hope to get from research all the answers we desire and to understand soil and pavement performance under freeze-thaw conditions, any approach or method of research must be based on a theoretical foundation. In addition to geology, mechanics, soil mechanics, hydraulics, and hydrology, we should also consult with increasing frequency the disciplines of mathematics and heat transfer. This is because none of the complex frost problems in soils can be undertaken without some knowledge of the thermal conductivity and diffusivity of the soil-moisture medium and the basic laws of heat transfer. In other words, we should utilize every theoretical tool available to our present state of knowledge and technology; because: (1) the system "soil-moisture-heat" is a complex thermal system requiring appreciation; (2) there is a need for better understanding of the physical factors and their mutual relationship in this thermal system and the laws governing it; and (3) large sums and great effort usually are spent in investigations pertaining to frost action in highway soils.

All these points, without reservation, manifest the great need for frost action research in highway engineering on a solid theoretical basis. The value of frost action research within the scope of highway research is now generally recognized, and therefore it is indisputable. Like all other phases of highway research, frost action research is vital, and is of national importance.

Experience indicates that conclusions drawn from observations on the effects of geophysical, climatic and soil conditions on highway behavior and performance in one area are not, in general, valid for other areas, however similar those areas may be. Therefore, it is believed that frost action research should be carried out under conditions as they prevail in each particular locality.

PURPOSE OF RESEARCH

The purpose of frost action research is to: (1) study the various factors interacting within the thermal system of soil-moisture-temperature and to try to find the fundamental relationships between factors entering into highway design and construction (of particular interest are thermal and thermoösmotic processes in soils in winter and summer, the climatic influence on the physical properties of soils, loss of bearing capacity of pavement-supporting soils under thawing conditions, spring breakup, study of materials to be used in lieu of rapidly diminishing supplies of gravel material in certain areas, and how to us substitute materials to advantage in highway construction); (2) investigate soils in orde to know which ones are particularly susceptible to heaving under freezing conditions; and (3) provide observation and test data to establish a method or criteria index for the eval- uation of frost danger to subgrade soils and to base and subbase courses where condition are conducive to frost action. All this, in turn, serves to obtain quantitative and qualita- tive knowledge for design purposes.

The ultimate goal of frost-action research is to provide the engineer with knowledge concerning design and construction of better roads, airports, and other earthworks and thus to contribute to the improvement in the service of our nation's transportation syster It is necessary because highways may be said to be the backbone of the nation's life and are considered to be the most-important factor in civilization, for progress and for na- tional defense. Of course, the magnitude of our efforts in research should be great enou to match the magnitude of the important highway performance problem under freeze-thaw conditions.

PLACE OF FROST-ACTION RESEARCH

Having familiarized ourselves with the important frost action problem in highway en- gineering, accepting the thought that frost action research of the thermal system of soil- moisture-temperature must be based on a theoretical basis and that among other discipli heat transfer should also be consulted and having formulated the need and purpose of such research, one might ask what is the place of frost action research in engineering?

The answer, fortunately, is not too difficult to be found. The place is géotechnique (s mechanics). This discipline permits us to subdivide it in phases paralleling the disciplin of general mechanics (see Table 1). Hence, it seems that the place of frost action resea

TABLE 1

SOIL MECHANICS (GÉOTECHNIQUE)

General Mechanics	Statics
Statics	Statics (all static soil tests, bearing tests, stability of slopes and foundations, static stabilization of soils, earth pressures)
Dynamics	Dynamics (all dynamic soil tests, seismic soil investigation, dynamic compaction of soils, pile driving, earthquakes and vibrations in soils induced by vibrating machinery)
Hydrodynamics	Hydrodynamics (consolidation, suction force, moisture migration in soils, permeability, lowering of groundwater table, injections [grouting])
Heat Transfer with Thermodynamics	Thermal Soil Mechanics (formation of ice, freezing and thawing, frost penetration, heaving, moisture migration in soils in cold and hot regions, heat transfer in soils, thermoösmosis, artificial freezing operations, permafrost)
	Geo-Electric Soil Investigation
	Electro-Ösmosis (de-watering of fine grained soils)
	Chemical Stabilization of Soils
	Nuclear Soil Mechanics (non-destructive tests for the determination of soil density and moisture content by means of radioactive isotropes)

in soils is in thermal soil mechanics under the aegis of geotechnique.

SCOPE OF RESEARCH

Having assigned the frost action research a place, we can now proceed with a brief discussion of the general scope of research. To start anything in research, there must be a theoretical basis for it. Using a theoretical basis, it is possible to treat the frost penetration problem analytically, provided the necessary quantities and soil constants, characterizing soil geotechnical and thermal properties, are readily furnished by tests. It is believed that the results of frost-action research based on sound theoretical considerations can be made available, with modification and adjustment, for practical application to highway and airport engineering.

The general scope of a frost research program should comprehend: (1) formulation of the problem; (2) library studies; (3) mathematical analyses; (4) building and construction of research facilities; (5) laboratory and field research; (6) studies of research material and testing of the hypotheses and theories; (7) scientific travel for gathering and exchanging information; and (8) correlation work and adjustment of findings to practice.

By mathematical analyses is here understood the treatment of the thermal system of soil-moisture-temperature by means of heat transfer theories. These theories have the advantage of (1) having a broad bearing and (2) the generality of their methods of analysis, which permits applying them in the theoretical and practical approach to the solution of frost problems in soils.

Of course, in any domain of scientific knowledge, some basic principles are treated by means of mathematics. Engineering has always been founded on a base of mathematics, and the trend today is toward more extensive use of it. As the Committee on Adequacy and Standards of Engineering Education of the American Society for Engineering Education recently concluded, "The greatest potential for future development in science and technology is to be found in mathematics."

In an article entitled "Have We Lost Control of Our Profession?" in "American Engineer" for May, 1954, Major General Samuel D. Sturgis, Jr., says:

"What is of real concern to me, however, is the gradual acceptance on the part of the average engineer in the average firm of an attitude geared to just getting by and no more. A continuance of such an attitude in any large segment of engineering firms will lead inevitably to a deterioration of the over-all quality of American engineering and a decrease in the confidence and esteem of those we serve, which it has always been our good fortune to enjoy . . . . One insidious influence on both the quality and quantity of engineers, the full impact of which is scarcely yet being felt, is the general drift in public schools away from mathematics and science towards the so-called social studies. The mental discipline of the three R's has been thrown overboard in favor of happy, well-adjusted children."

Hence, we conclude that the solutions pertaining to the problems of our thermal system are governed to a great extent by mathematics.

THEORETICAL BASIS

Having established the need for a theoretical basis in frost action research, it is pertinent to review briefly the following principal theories: (1) heat flow in the steady state, (2) heat flow in the unsteady state, (3) temperature oscillation theory, (4) sudden changes in temperature, (5) cold snaps, (6) Neumann's theory, (7) Stefan's theory, and (8) Ruckli's suction force theory.

Frost penetration in soils and thawing is simultaneously a geotechnical and a heat transfer problem. As a theoretical basis for analysis, the following laws apply: (1) the amount of heat in a differential soil element is proportional to its mass and to its temperature; (2) heat flows from a higher to a lower temperature; (3) the rate of heat flow across an area is proportional to it and to the temperature gradient at a point of that area; and (4) upon freezing, the migration of soil moisture takes place from a warmer medium toward the cold front (thermoösmosis).

1. Depending upon field conditions, heat conduction in the steady state or unsteady state is to be considered.[1] Where the temperature gradient is linear and constant, and the soil can be considered homogeneous, isotropic material, with constant geotechnical and thermal properties, the law of steady-state flow of heat can be applied:

$$q = K \cdot A \cdot \frac{dT}{dx}, \tag{1}$$

where

q = rate of heat flow;

K = thermal conductivity;

A = the area measured normal to the direction of flow, and

$\frac{dT}{dx} = i$ = temperature gradient.

2. (a) Analysis furnishes the following general partial differential equation for unsteady-state heat conduction in rectangular coordinates:

$$\frac{\partial T}{\partial t} = \alpha \left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial^2 y} + \frac{\partial^2 T}{\partial^2 z} \right), \tag{2}$$

where

T = temperature in the soil;

t = time;

α = coefficient of diffusivity of a soil;

x, y, and z = space coordinates.

This equation expresses the conditions that govern the flow of heat in soil. The soil itself is considered to be homogeneous. (b) Assuming that on highways cold penetrates the soil in one direction only (vertically), i. e., normal to the upper road surface ($y = O$, $z = O$), we have a special case of Equation (2), which can be rewritten as follows:

$$\frac{\partial T}{\partial t} = \alpha \cdot \frac{\partial^2 T}{\partial x^2} \tag{3}$$

This type of equation has wide application in the theory of heat transfer as well as in the unidimensional consolidation theory of soils.

3. Observations show that the highway surface- and air-temperatures oscillate diurnally sinusoidally around a mean temperature, T_O. Hence, the solution of the partial differential equation (3) must also fit the surface boundary condition (at $x = O$)

$$T = T_O \cdot \sin(w \cdot t), \tag{4}$$

where

$w = \frac{2 \cdot \pi}{P}$, P being the period, and

x = depth below the ground (road) surface.

The solution of equation (3) is:

$$T = T_O \cdot \left(e^{-x \cdot \sqrt{\frac{w}{2 \cdot \alpha}}} \right) \cdot \sin\left(w \cdot t - x \cdot \sqrt{\frac{w}{2 \cdot \alpha}} \right), \tag{5}$$

where

$e = 2.71 \ldots$ = the base of natural logarithms.

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[1]A full treatment of the various theories can be found in A. R. Jumikis, "The Frost Penetration Problem in Highway Engineering," 1955, Rutgers University Press, (in preparation).

Respective maximum and minimum points on the temperature oscillation wave are delayed progressively by $x \cdot \sqrt{\frac{w}{2 \cdot \alpha}}$ as depth x increases. The amplitude of the oscillating temperature wave is damped rapidly by

$$\left(e^{-x \cdot \sqrt{\frac{w}{2 \cdot \alpha}}} \right)$$

as depth, x, increases.

This solution is valid for a homogeneous and uniform soil. No consideration to formation of ice lenses in the subgrade is taken into account. Nevertheless, it gives a fairly good idea as to how the temperature fluctuations in the soil take place.

The quantities α (thermal diffusivity) and x (depth) are to be furnished by tests and investigations, respectively. Besides, this theory allows determining the thermal diffusivity, α, of a soil or pavement in place by means of temperature measurements at different depths. Tests in the laboratory can also be performed.

4. Sudden Changes of Temperature. For the solution of the problem as to how deep will the freezing temperature (say 0 C. = 32 F.) penetrate a soil in a certain period of time (α must be furnished by tests), if the surface temperature is lowered to T_S^0, the following equation, which is also a particular solution of the general partial differential Equation 3, can be used;

$$\frac{T - T_S}{T_0 - T_S} = \frac{2}{\sqrt{\pi}} \cdot \int_0^{x \cdot \eta} \left(e^{-\beta^2} \right) \cdot d\beta = G\left(x \cdot \eta\right), \tag{6}$$

where

T = temperature at depth x (for example, T = 0 C. = 32 F.),

T_0 = average initial temperature of the soil,

T_S = surface temperature,

t = time,

$\eta = \dfrac{1}{2 \sqrt{\alpha \cdot t}}$,

$\beta = \dfrac{x}{2 \sqrt{\alpha \cdot \left(t - \tau\right)}}$

τ = time variable (limits 0 and t),

$G\left(x \cdot \eta\right)$ = Gauss's probability integral (or error function).

For example, assuming T = 0 C. = 32 F., T_0 = 5 C. = 41 F., T_S = 20 C. = -4 F., and with α = 0.005 ft.²/hr., and t = 24 hours, x = 1.25 ft.

5. The following expression

$$T = \frac{2}{\sqrt{\pi}} \cdot \int_{x \cdot \eta}^{\infty} f\left(t - \frac{x^2}{4 \cdot \alpha \cdot \beta^2}\right) \cdot \left(e^{-\beta^2}\right) \cdot d\beta , \tag{7}$$

which is also a particular solution of the general partial differential Equation 3, enables a more accurate evaluation of the effect of surface temperature fluctuation (so-called cold snaps) to be made than is possible if one assumes that they are simple periodic sine variations.

This expression is particularly well suited for problems where it is necessary to find soil temperatures at a certain depth, when a period of uniform soil temperature, say 0 C. = 32 F., is broken by a cold snap lasting several days.

6. Neumann's Solution. Taking into consideration ice and water, Neumann gives two equations for a moist, isotropic, semiinfinite body whose temperature to begin with is positive and constant. By suddenly lowering the surface temperature to a new constant but freezing value, the thermal process is started. The two equations are:

$$\frac{\partial T_1}{\partial t} = \alpha_1 \cdot \frac{\partial^2 T_1}{\partial x^2} \quad \text{for ice,} \tag{8}$$

and

$$\frac{\partial T_2}{\partial t} = \alpha_2 \cdot \frac{\partial^2 T_2}{\partial x^2} \quad \text{for water,} \tag{9}$$

where ξ is the thickness of the advancing ice layer, which is a function of time, thermal and other physical properties of the soil.

Considering latent heat of fusion, and setting up boundary conditions, the solutions of Equations (8) and (9) are:

$$T_1 = B_1 + D_1 \cdot G(x \cdot n_1), \tag{10}$$

and

$$T_2 = B_2 + D_2 \cdot G(x \cdot n_2), \tag{11}$$

where

$$G(x \cdot n) = \frac{2}{\sqrt{\pi}} \cdot \int_0^{x \cdot n} \left(e^{-\beta^2}\right) \cdot d\beta$$

is the Gauss's probability integral. One of the boundary conditions is:

$$\xi = b \cdot \sqrt{t}\,, \tag{12}$$

i.e., the frost penetration depth ξ is proportional to $\sqrt{t}$. The coefficient m is to be determined from the following transcendental function:

$$b_1 \cdot \left(T_f - T_s\right) \cdot \frac{e^{-\frac{m^2}{4 \cdot \alpha_1}}}{G\left(\frac{m}{2 \cdot \alpha_1}\right)} - b_2 \cdot \left(T_o - T_f\right) \cdot \frac{e^{-\frac{m^2}{4 \cdot \alpha_2}}}{I - G\left(\frac{m}{2 \cdot \sqrt{\alpha_2}}\right)} = \frac{Q_L \cdot \delta_s \cdot \sqrt{\pi} \cdot W \cdot m}{2}, \tag{13}$$

where

$$b_1 = \frac{K_1}{\sqrt{a_1}} = \sqrt{K_1 \cdot c_1 \cdot \delta_1}\,,$$

$$b_2 = \frac{K_2}{a_2} = K_2 \cdot c_2 \cdot \delta_2\,,$$

a_1 and a_2 are coefficients of thermal diffusivity,

Q_L = latent heat (when water is converted into ice),

δ_S = density of ice, viz., frozen soil,

δ_1 = density of ice,

δ_2 = density of water,

c_1 and c_2 are specific heats,

T_s = surface temperature,

T_o = initial temperature,

w = moisture content in soil,

and T_f = 32 F. = freezing temperature.

7. Stefan's Solution - Stefan simplified the foregoing theory assuming that the temperature gradient in the ice layer is linear, and the temperature of the water is 0 C. = 32 F. The frost penetration depth, ξ, according to Stefan, is expressed:

$$\xi = \sqrt{\frac{2 \cdot K_1}{Q_L \cdot \gamma_1} \cdot T_s \cdot t} \qquad (14)$$

The first derivative of ξ with respect to time t gives the rate of frost penetration.

8. Suction Force. Neumann's and Stefan's theories are valid only for soils where the moisture present is motionless. However, on formation of ice lenses, moisture in soils is subject to migration or flow towards a cold boundary (ice lenses). The upward flow of moisture in soils can be considered as being caused by the so-called suction force, P_s, the magnitude of which, according to Ruckli, can be expressed as follows:

$$P_s = \frac{(\gamma_w) \cdot (\Delta h) \cdot (H - \xi)}{(1.09) \cdot (t) \cdot (k_s)}, \qquad (15)$$

where

P_s = subpressure (real or fictive),

γw = unit weight of water,

Δh = amount of permissible frost heave,

H = depth of the ground-water table below ground surface,

δ = frost penetration depth,

t = time required to obtain an upward flow (in laboratory or actual freezing period),

k_s = coefficient (experimental) of proportionality of vertical flow of moisture through the soil concerned.

It seems that the suction force, inaugurated on freezing, would be an important soil constant in predicting frost penetration depths and calculating the amount of lowering the ground water table in order to interrupt moisture supply to the growing ice lenses.

9. Ruckli's Theory. The theory by R. Ruckli furnished a differential equation for frost penetration depth, ξ, taking into consideration the vertical flow of soil moisture toward a cold front upon freezing, suction force, porosity of soil, moisture content in soil, unit weight of the soil, depth of the ground-water table, diffusivity of the soil, duration of the cold period and its intensity, and other thermal properties and soil mechanical characteristics of the soil:

$$\frac{d\xi}{dt} = \frac{A}{\xi} - B \cdot v - \frac{C}{\sqrt{\xi}} \qquad (16)$$

in which A, B and C are constants to be determined for any given case, and v = velocity of the upward flow of soil moisture toward the ice lenses. The general solution of this equation can be found by plotting the integral curve of Equation (16), or analytically.

It seems to be obvious that basic theories contribute to the understanding of natural phenomena and provide an objective guide in theoretical as well as applied research. Particularly are they helpful when we have to face new and unfamiliar situations. In such cases they facilitate handling the problem in question with real competence. Therefore, heat transfer theories can be considered as great untapped sources of knowledge for the solution of the complex frost problems.

OFFICE, LABORATORY, AND FIELD RESEARCH

From the foregoing discussion it can be seen that for the evaluation of frost susceptibility of soils used in highway engineering, the following general studies, characterizing the properties of the particular soil, should be made; their results, as well as other information relative to the particular problem, should be studied and correlated: (1) climatologic data (duration and intensities of frost periods); (2) soil temperatures (in the field and laboratory); (3) ground-water table fluctuations and temperatures; (4) frost heaves on highways; (5) frost penetration depths; (6) physical and mechanical properties of soils entering into the evaluation of their frost susceptibility; (7) moisture content variation in unfrozen and frozen soils; (8) permeability of soils; (9) suction force studie in soils; (10) thermal properties of soils; (11) thermal properties of concrete pavements (12) thermal properties of bituminous pavements; and (13) correlation of findings.

Whatever our frost action research program should be, and whatever the definition of the practical problem might be, two kinds of research and studies should be done anyway: (1) research on thermal diffusivity and conductivity of soils and highway pavements and (2) research on suction force in soils upon freezing. If we have this information, we may apply any one of the aforementioned theories, with or without modifications, to a practical problem to be investigated.

Of course, important elements in any research are research-minded men, time, space to do research, basic equipment, a good library and financial support. Needless to menti that, above all, enthusiasm for research is a contributory factor of research workers, sp sors and those who direct research, as well as good cooperation and interest of all parties involved. Besides, as in any other field of scientific and technical endeavor, new facts, their better understanding and their better application can be expected from frost action re search. This contributes to the advancement of knowledge.

What has been learned by some research workers today might result in new, practical applications tomorrow which would be directly useful in highway engineering.

ACKNOWLEDGMENT

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Drainage Index in Correlation of Agricultural Soils with Frost Action and Pavement Performance

WILBUR M. HAAS, Assistant Professor of Civil Engineering,
Michigan College of Mining and Technology

This paper deals with the application of selected concepts of pedology to highway soil-engineering problems. Particular emphasis is placed on the natural drainage index of a soil type as an indicator of its probable performance as a highway subgrade. The report is based on a study of representative soils of the glaciated portion of Wisconsin. Because of the severe climate, frost action in soils is a major consideration in the design of highways in this state.

The natural drainage index is primarily a function of the topographic position of the soil type and of the texture or textures of its profile. The soil association is a group of soils that commonly occur throughout a soil region. The individual soil types within this association may vary with respect to topographic position, texture, or both. Therefore each soil within this association has its particular drainage index. For agricultural purposes, these indices range from "excessively drained" to "very-poorly drained." The highway engineer probably will interpret these terms differently, but the drainage relationships of the soils are useful, nevertheless.

Soil samples were taken from selected horizons of 19 soil types. Each profile was assigned a generalized frost-susceptibility rating on the basis of the rating of each of its horizons. When these ratings are compared with the drainage index, it is shown that the soils that are least susceptible to frost action usually occupy the better drained positions, and the soils that are most susceptible to frost action are in the intermediate to poorly drained positions.

Another phase of the study was concerned with observations of pavement performance over several types of soil. On one study section, the actual amount of longitudinal and transverse cracking was determined for each of the several soil types traversed by the pavement. This pavement broke up the most where laid over poorly drained soils and performed better on the better-drained soils. On another study section, certain segments of the pavement had been resurfaced. When these segments are plotted on the soil map, it is shown that resurfacing was not generally required over the well-drained soils, that resurfacing alone was required over the intermediate soils, and that resurfacing plus a new base was required over the poorly-drained soils.

The results of this investigation indicate that the drainage index of the soil type may be of considerable value in soil engineering. This is borne out by the correlations with accepted soil classification systems and with the actual performance of soils as a subgrade.

● THE use of the pedological system of soil classification has attracted considerable interest, with several states using it to a greater or lesser extent in their soil engineering problems. Evidence of this interest is demonstrated by the series of bulletins published by the Highway Research Board dealing with the use of soil maps, aerial photos, and other means for determining soil conditions (6, 11, 12, 15). Some states have published detailed manuals as an aid to the interpretation of the significance of the various soil series and types (13).

This paper reports some of the results of a research project, the object of which was to develop general relationships between the pedological classification and recognized engineering classification systems. The project was divided into two major phases. The first phase consisted of determining the engineering classification of 65 samples taken from 19 selected soil profiles. The second phase consisted of observing the actual performance of pavements constructed over each of several soil series.

A number of significant relationships were developed or reaffirmed, but the most-important relationships appear to be those between the natural drainage index of the soil series and the Highway Research Board (now American Association of State Highway Officials) classification (1), the Corps of Engineers frost-susceptibility rating (10), and the actual performance of the pavements. The natural drainage index is a general expression of the natural soil moisture content and is a function of both the topographic position and slope of the soil body and of the texture of the soil profile.

Other relationships that appear to br significant include those between the textural classification and the susceptibility to frost action, and to pumping action under a rigid slab.

DRAINAGE INDEX ILLUSTRATED BY LANDSCAPE DIAGRAMS

The natural drainage index has been described as a measure of the natural moisture condition of the soil profile. It is influenced by both the texture or relative permeability of the soil profile and the landscape position and slope, which governs both surface and subsurface drainage. There are, of course, many variations and combinations of these factors, but a few illustrations will serve to point out the possible range. For instance, a sand could be "excessively drained" if in a relatively high position, or "moderately well drained" if in a depression and if underlain by a less-permeable layer. A silt or till soil on a hill would be "well drained" or possibly better, but in lower lying positions, especially in depressions, the same texture would be "poorly drained."

It should be pointed out here that the engineer and the agronomist probably will not interpret these word descriptions in the same manner. While the highway engineer usually would think of a sand as being "excellently drained," the agronomist would consider this same material to be "excessively drained." Many silt loam soils would be rated as "well-drained" by the agronomist, but the engineer would probably consider the drainage only "fair." Recognizing the difference in interpretation, this report will use the soil mapping terminology, rather than the terminology that might be suggested by engineering considerations.

With the objective of condensing and clarifying the many items that enter into a complete pedologic description of soil bodies, F. D. Hole has suggested certain terms, and corresponding numerical scales to permit the writing of the soil description in a numerical shorthand (7). One of the descriptive items for which he proposes a numerical index is the natural drainage of the soil.

In the system he proposes, well-drained soils are arbitrarily assigned the value of +1. An organic soil or bog soil is rated as +10, very-poorly drained, while a soil that is very-excessively drained is rated as -10. Soils of drainage characteristics between these extremes are assigned appropriate numbers to indicate their relative position in the sequenc This system will be further illustrated by a definite example.

If all the possible combinations of texture and topographic position that condition the drainage of the soil are considered, the range at first seems too great to comprehend and organize into workable units. However, the number of soil series in a given landscape association or major soil region is not large. By studying the relationships between the members of one association, a good idea of their probable nature can be developed. The soils belonging to an association are related because of similarities in climate and geological history and typically occur together in fairly consistent patterns. Furthermore, soil associations occur in rather well-defined zones or belts, which fact facilitates their study considerably. This is illustrated by Figure 1, a map of the major soil regions as recognized in Wisconsin at the time of the study.

Within a given association or soil region, the general nature of the various soil series can be shown rather well by means of landscape diagrams (14b). Figure 2 illustrates a fe of the important considerations. The Rodman soil, formed from gravel and sand, is "excessively drained" (natural drainage index of -10). The Bellefontaine soil is " somewhat excessively drained," -2. The difference in drainage, compared to the Rodman soil, is a

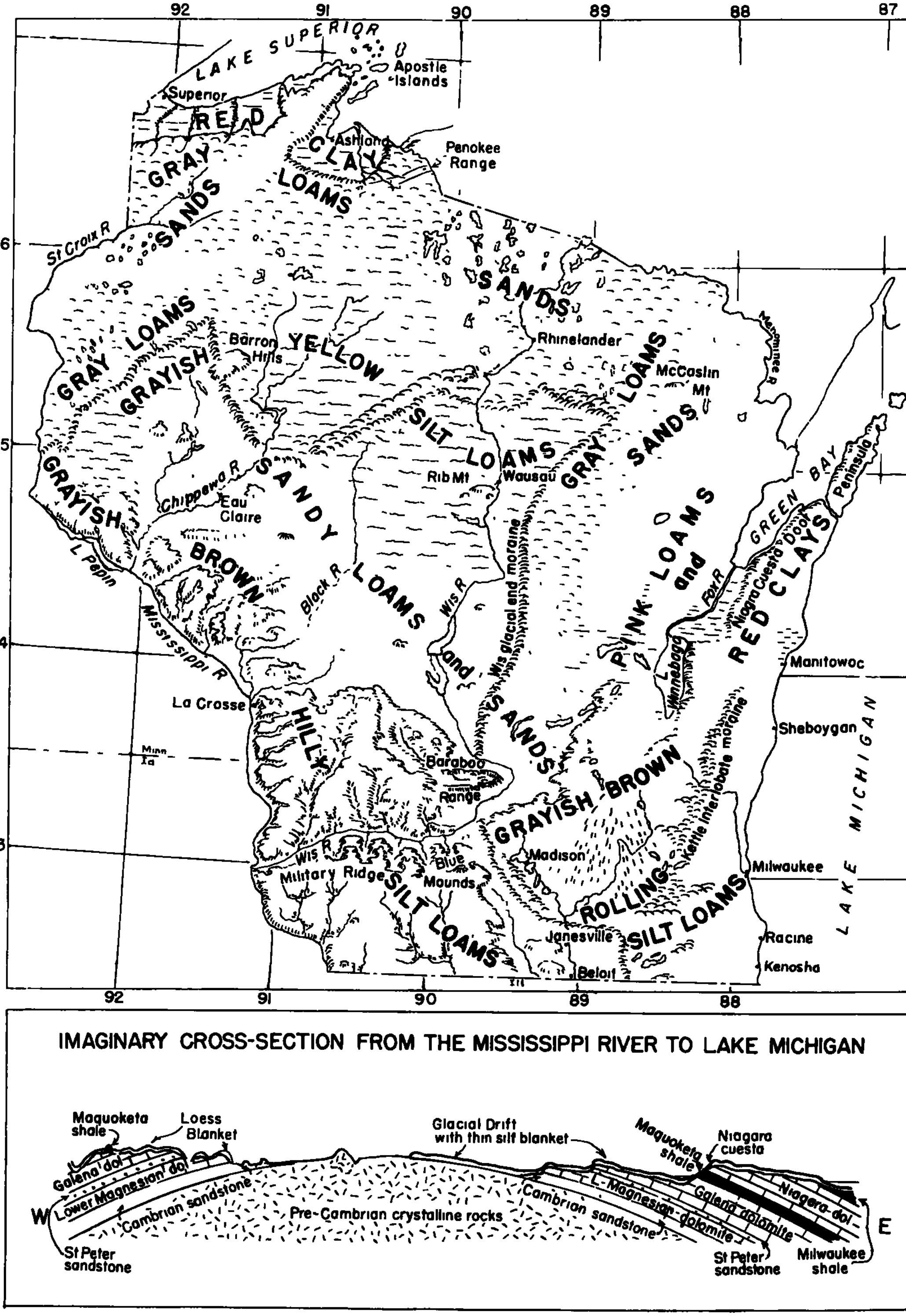


Figure 1. Major soil regions of Wisconsin. (The Soil Survey Division, Wisconsin Geological and Natural History Survey.)

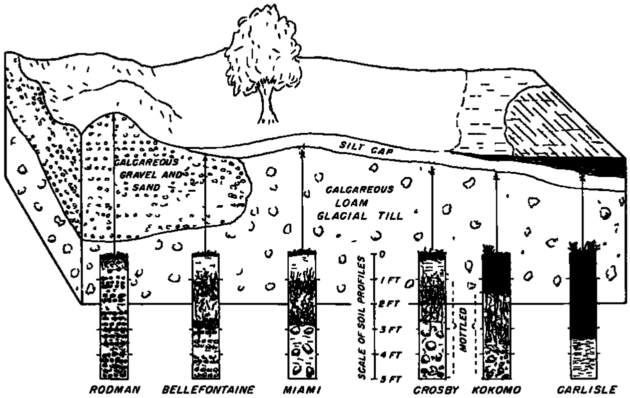


Figure 2. Typical soils of the grayish-brown rolling silt-loam region. (Soil Survey Division, Wisconsin Geological and Natural History Survey.)

result of its lower position, and to a minor extent, of the thin silt cap covering the gravel and sand. Although the Miami soil may be actually somewhat higher in the landscape than the Bellefontaine, it is rated as "well-drained," +1, because of the loam till underlayer, with a silt cap of moderate thickness.

The only difference between the Miami and the Crosby is the position, yet the Crosby soil is sufficiently lower to be rated as imperfectly drained, +4. The Kokomo soil is a marsh border soil, only slightly lower than the Crosby, but very-poorly drained, +8.5. The Carlisle soil is a bog soil, very-poorly drained, +10.

The gravel and sand are of glacial origin, as is the loam till. The silt cap is a wind-blown deposit that may be as thick as 4 feet.

Other diagrams have been prepared by Hole to illustrate other soil association areas (14c).

STUDY OF SELECTED SOIL PROFILES

As stated in the introduction, the objective of this phase was to determine the engineering classification of the significant horizons of several soil profiles and to see what general relationships or correlations could be determined, if any. Soil profiles were selected from a wide range of soil series and major soil regions in Wisconsin's glaciated area. These ranged from "excessively drained" soils to "very-poorly drained soils," to use the terminology of the soil surveyor.

The test program consisted of determining the liquid limit, the plasticity index, and the mechanical analysis. The results of these tests permitted the AASHO classification and the Corps of Engineers frost-susceptibility rating to be determined for each of the horizons studied.

The relationship between AASHO subgroup and the group index number is shown in Tabl 1. This is of interest because tentative thicknesses of flexible pavement were suggested for given ranges of the group index number (1), and the Washington State Highway Department has tabulated thicknesses required for each subgroup (8). When actual soils are pro erly located in this table, it is possible to check the extent of agreement between the two systems.

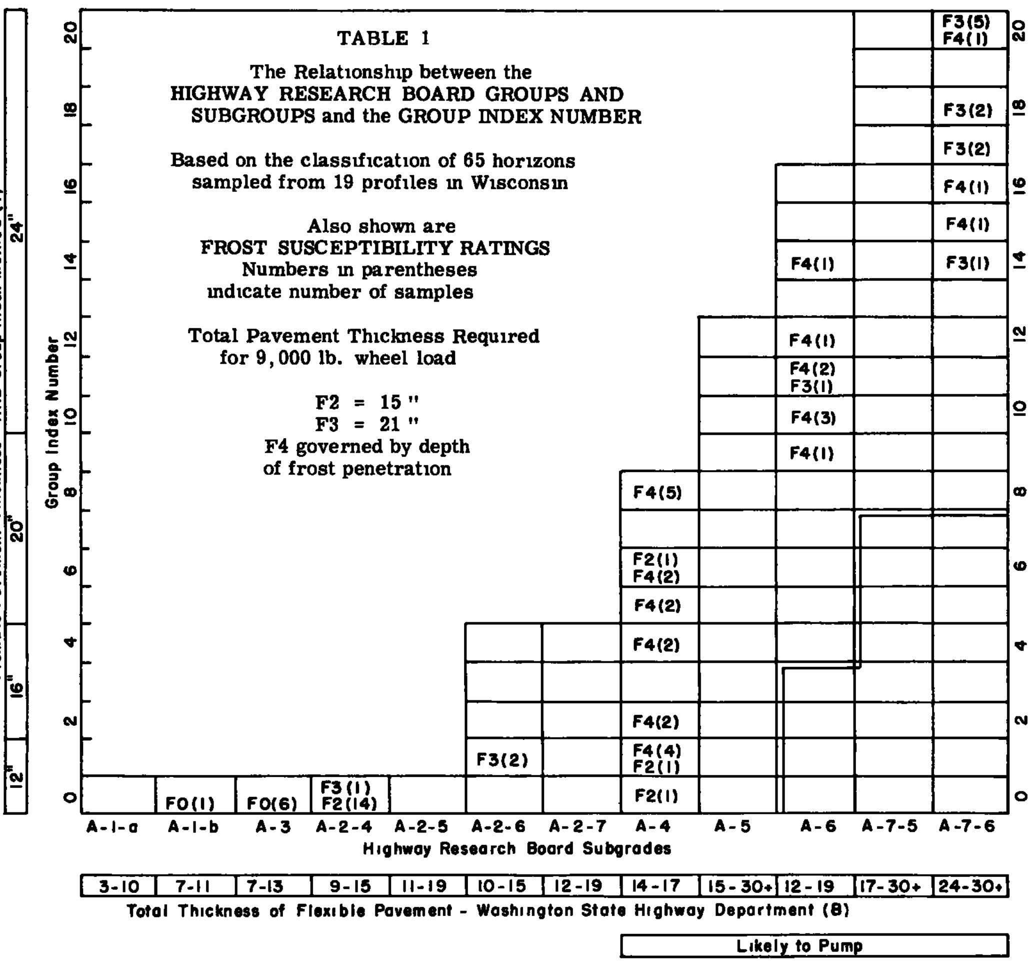
This table also shows the frost-susceptibility ratings of the soils tested, located in the table according to group and group index number. On the basis of this table, it may be concluded that there is no significant relationship between the group index number and the frost-susceptibility rating. There is a general correlation between frost action and the su groups, however. A-1 and A-3 soils probably are not susceptible to frost action, A-2 soi are moderately frost-susceptible, A-4 soils are moderately to very-highly frost susceptib and A-6 and A-7 soils are highly to very-highly frost susceptible. The differences in pave ment thickness required are due primarily to differences in the duration and magnitude of subfreezing temperatures.

The criteria for the frost-susceptibility ratings referred to in the preceding paragraph may be briefly summarized as follows: F-0, generally not frost-susceptible; F-1, gravelly soils containing between 3 and 20 percent finer than 0.02 millimeter by weight; F-2, snads containing between 3 and 15 percent finer than 0.02 millimeter by weight; F-3, (a) gravelly soils containing more than 20 percent finer than 0.02 millimeter by weight, and sands, except fine silty sands, containing more than 15 percent finer than 0.02 millimeter by weight and (b) clays with plasticity indices of more than 12, except varved clays; F-4, (a) all silts including sandy silts, (b) fine silty sands containing more than 15 percent finer than 0.02 millimeter by weight, (c) lean clays with plasticity indices of less than 12, (d) varved clays.

The rating F-0 is not included in the original system, but the author has taken the liberty of adopting that designation for soils not susceptible to frost action.

From the above criteria it can be seen that the relative frost susceptibility of soil is largely a function of its mechanical analysis or texture. This suggests plotting the several samples on the triangular textural chart and indicating the frost-susceptibility rating. Figure 3 is such a chart, based on the chart currently used by the agricultural soil survey agencies and the 2-micron definition of maximum-size clay particles.

The results of this correlation may not be surprising to those persons well acquainted with the nature of frost action. However, there are a few rather clear-cut patterns that



may be of interest. These patterns suggest the following conclusions: (1) It is emphasized again that the silts are the worst offenders with respect to frost action. (2) Within proper limitations, it appears that the frost-susceptibility rating could be determined from the textural classification with reasonable accuracy. (3) Clays, clay loams, silty clays, and silty clay loams are likely to have an F-3 rating. (4) Silts, silt loams, and loams are likely to have F-4 ratings. (5) Sands may range from F-0 to F-2. (6) All the loamy sands had an F-2 rating, but inspection of the pattern in the area of loamy sands and sandy loams near the line of zero clay indicates that there is a very narrow boundary between F-2 and F-4 ratings. This arises from the criteria used in establishing the frost susceptibility ratings originally. (7) Sandy loams are likely to be either F-3 or F-4, depending upon the precise amount of silt present. (8) To the extent that this sampling is representative of Wisconsin soils, most of them are frost-susceptible. Of the 65 samples analyzed, 7 rated F-0, 17 rated F-2, 15 rated F-3, and 26 rated F-4. (9) In the zones on the chart where slight differences are critical, an abbreviated mechanical analysis by

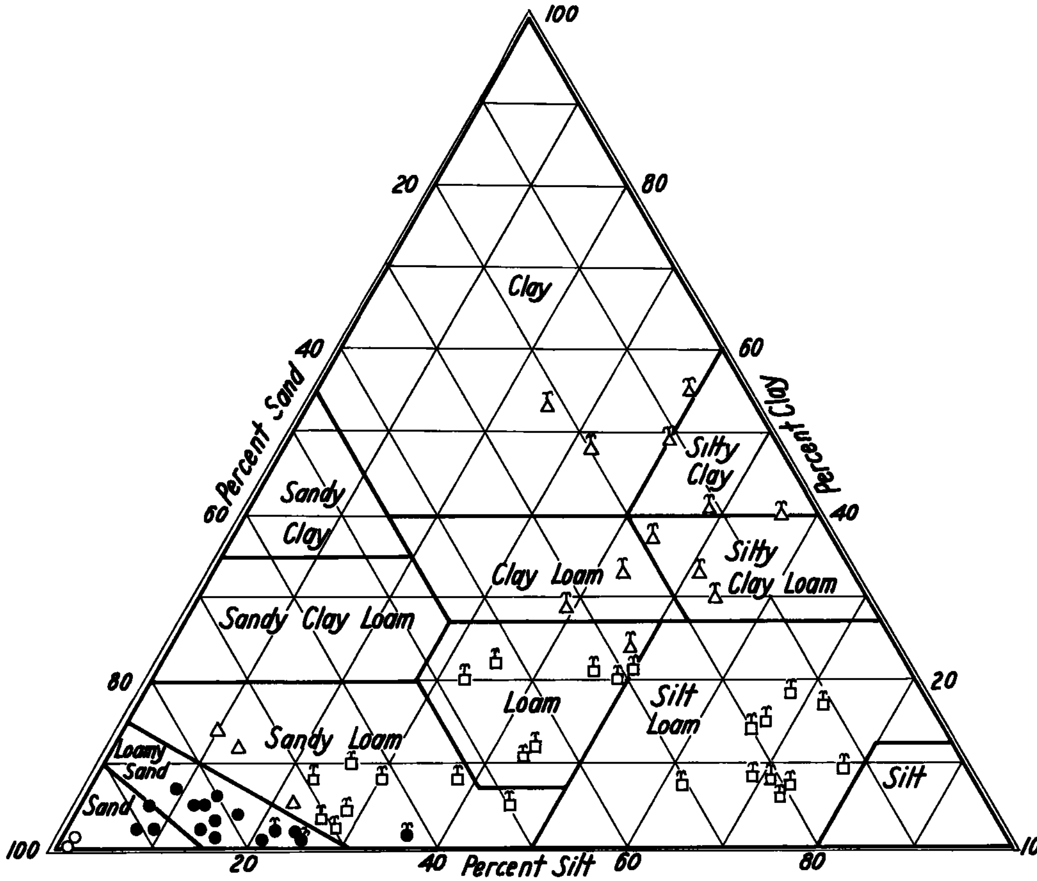


Figure 3. Relationship of frost-susceptibility rating and pumping to textural classification O = F0 ● = F2 Δ = F3 □ = F4 τ added to any of above symbols indicates that the soil is susceptible to pumping action under rigid slabs.

by sedimentation methods (determining the percent finer than 0.02 millimeter) supplementing the textural classification would establish the correct frost-susceptibility rating.

The same figure indicates some possible correlations with susceptibility to pumping action under rigid slabe. These conclusions are as follows: (1) Clays, silty clays, silty clay loams, clay loams, silt loams, and loams are very likely to pump. (2) Sands are not likely to pump. (3) The correlation is not good for sandy loams and loamy sands, but it appears that about 20 percent silt is the maximum for a nonpumping soil. This would require further analysis to make a final determination. (4) With few exceptions, soils with frost-susceptibility ratings of F-3 and F-4 are also susceptible to pumping. (5) Soils that are rated F-2 generally are not susceptible to pumping, with a few exceptions where the silt content is rather high.

For the above correlations of pumping with the textural chart, the gradation of each sample was compared with criteria used in North Carolina (6), and in Ohio (12). In Ohio, it was discovered that pumping was confined principally to soils having more than 45 percent of the particles passing the No. 200 sieve. Pumping was also related to the frequency of loading of heavy axles.

North Carolina has established rather detailed specifications for "blotter" courses to be used under rigid pavements. These are provided specifically to prevent pumping action. The minimum specification for this material is as follows:

100 percent passing	No.	10	sieve
40 - 100 " "	No.	40	"
12 - 35 " "	No.	200	"

"The percentage passing the No. 200 sieve shall not exceed two thirds of the percentage passing the No. 40 sieve. The liquid limit shall not exceed 25, and the plasticity index shall not exceed 6. Material which does not meet this specification is considered to be susceptible to pumping."

In figure 3, any sample that is susceptible to pumping according to either or both of the criteria is designated as pumping.

While such studies of individual horizons indicate general relationships and are the key to profile studies, it is nevertheless true that soils occur in definite profile relationships. Therefore the actual profile must be considered to gain a fuller appreciation of the probable behavior of the soil under a pavement. For example, it has been previously recognized that the contrast between horizons may account for the concentration of pavement breakup where the grade line shifts from cut to fill.

This contrast is shown in graphical form for the several types of soil parent material in Figure 4. On the two charts the AASHO subgroups are arranged from left to right, with the group index number as a secondary variable plotted along the same axis, where applicable. The horizons for each soil series are arranged downward from the top, and designated by letter and subscript. Trace lines have been drawn to facilitate the comparison of horizons. From these charts the extent of contrast can be visualized. If the trace line is vertical, there is no contrast. If the trace line is sloping or nearly horizontal, varying extremes of contrast are indicated. Also, the general or average position of the horizons indicates the nature of the profile as a whole. Positions at the left indicate the best soils, while positions at the right indicate the poorer soils.

From Figure 4, it may be seen that soils formed from silt over till range from rather good quality with minor contrast to poor quality with considerable contrast.

Soils formed from sands and lake clays show little or no contrast between horizons, although the two groups are at opposite ends of the quality scale. Soils formed from silt over stratified sands show extreme contrast, and are difficult to generalize as their relative quality.

Examination of these diagrams also leads to two other interesting observations. In many cases, while adjacent horizons fall in different subgroups, these subgroups bear some relation to each other. For example, the Milaca, Iron River, and Eldron profiles are composed of an A-4 horizon over an A-2-4 horizon, while in the Shiocton and Elba profiles, this relationship is reversed. These two subgrades are related in the classification system because of si, ilar plastic properties.

In addition to this relationship, it should also be noted that as a general rule the soils plotting to the left, or good quality range, are those that are among the better drained soils, while those falling to the right, or in the poorer subgroups, are also those that are inadequately drained.

To compare the profiles of several soil series, however, some generalization must be attempted. Charts of this nature offer a means of rating profiles according to the probable governing horizon of each. Furthermore, several profiles fall into a given pattern of B-to-C horizon relationship, as already demonstrated. Other profiles have no significant contrast. In the case of still other profiles, the probable position of the grade line

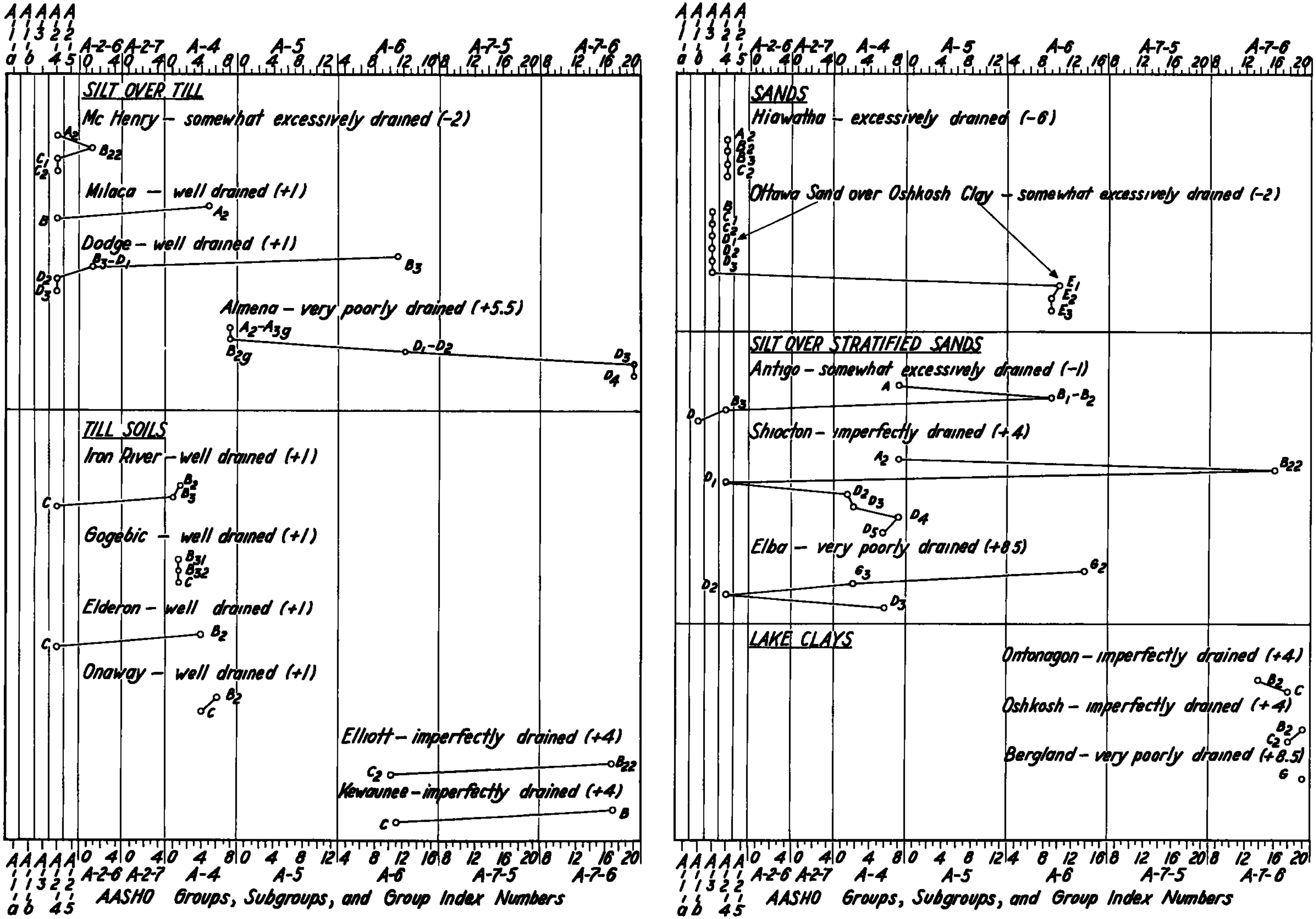


Figure 4. Profile contrast and rating.

relative to the horizons determines which AASHO group the profile as a whole shall be placed in.

Such a generalized rating of the profile as a whole is necessary if the drainage index concept is to be used, as the drainage index refers to the entire profile, not individual horizons. In a similar manner, the profile can be generalized with respect to its frost-susceptibility rating.

The frost-susceptibility rating does not mean that the intensity of frost action will vary with the rating regardless of other conditions. It does mean that if all soils are located similarly with respect to a moisture supply, those with the highest rating will exhibit the greatest frost action. This in turn suggests that if all soils or soil profiles of a given frost-susceptibility rating be arranged with respect to their drainage index, a truer picture might be indicated of the actual amount of frost action to be expected. For example, the actual distress due to frost action would be much less in the Antigo series (somewhat excessively drained) than in the Almena series (poorly drained) although both are in the F-4 category.

Table 2 has been prepared for this purpose. The several profiles are arranged according to their respective drainage indices on the vertical scale, and their generalized frost-susceptibility rating on the horizontal scale. The soil profiles themselves are

TABLE 2

DRAINAGE INDEX RELATED TO FROST-SUSCEPTIBILITY RATING

Drainage Index	F-O	F-1	F-2	F-3	F-4
-6 Excessive			Hiawatha, A-2		
-2 Somewhat Excessive	Ottawa, A-3		McHenry, A-2		
-1 Somewhat Excessive					Antigo, A-6 & A-2
+1 Well Drained			Milaca, A-2 Elderon, A-2 Iron River, A-2 Dodge, A-2		Gogebic, A-4 Onaway, A-4
+2.5 Moderately Well Drained				Dubuque, A-7-6	
+4 Imperfectly Drained				Ontonagon, A-7-6 Oshkosh, A-7-6 Kewaunee, A-7-6	Shiocton, A-7-6 Elliott, A-6
+5.5 Very Poorly Drained					Almena, A-4, A-6, & A-7-6
+8.5 Very Poorly Drained				Bergland, A-7-6	Elba, A-6

represented by their series names and generalized AASHO classifications. This table shows that the soils that are least susceptible to frost action generally occupy the most favorable drainage positions, and that the soils most susceptible to frost action generally occupy the least favorable positions, although there is some overlapping. The chart also shows a similar pattern for the generalized AASHO classification. In those cases where it is difficult to express a generalized rating, more than one group is indicated.

STUDY OF PAVEMENT PERFORMANCE

The objective of this phase was to determine differences in pavement performance ove soil types of various drainage indices. The general procedure was to determine the amount of pavement cracking or to otherwise rate the performance over individual soil bodies, then to correlate this rating with the natural drainage indices of the several soil bodies. On the basis of three types of studies, there appears to be a definite relationship between pavement performance and the drainage index.

The first of these studies consisted of determining the amount of transverse and longitudinal cracking on a concrete road about 10 miles in length. The results are tabulated in two sections because of two different jointing arrangements. On one section, there were no transverse joints, except for construction joints, while on the other section transverse expansion joints were spaced at approximately 50-foot intervals. There was no longitudinal joint on either of the sections. With a detailed soil map in hand, the amount of cracking over each soil body traversed by the highway was determined by walking over the entire length.

When these data are reduced to averages and expressed as the amount of cracking per unit of length, a definite relationship can be noted between the drainage index of the soil bodies and the amount of longitudinal cracking in the pavement. Also, a reasonably good

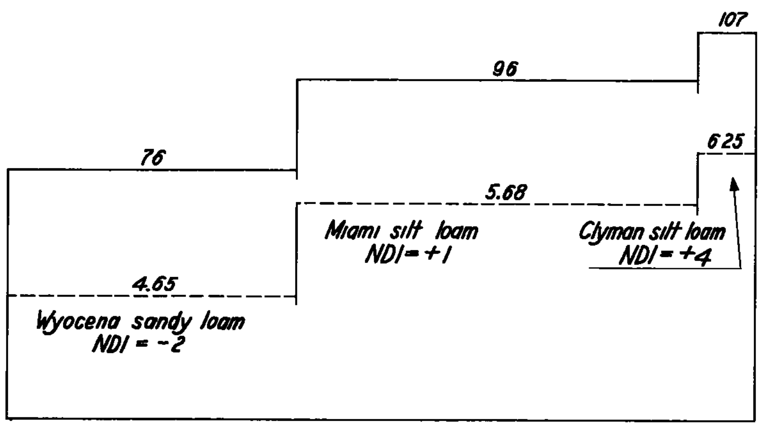


Figure 5. Intensity of pavement cracking related to soil series and drainage, unjointed section. Solid lines and corresponding numbers indicate longitudinal cracking in percentage of total length; broken lines and corresponding numbers indicate transverse cracking expressed as average number of cracks per 100 feet along centerline, NDI is the natural drainage index, expressed numerically.

correlation is shown for the intensity of transverse cracking, although there are some discrepancies here. These relationships are shown in graphical form as Figures 5 and 6, with intensity of cracking increasing upward. The relative distance over each soil type (and therefore over each drainage index) traversed by the highway is shown by the horizontal length of the appropriate lines. The soils are arranged so that the most-poorly drained soils generally are to the right. The greatest intensity of cracking and therefore the poorest performance occurs on these soils.

The second of these studies considered the relationship of the resurfacing requirement to the soil series along a 16-mile section of highway. During 1952 it was found necessary to resurface this highway in certain sections because of excessive breakage. In some cases, a granular base course was required in addition to the bituminous mat, and in

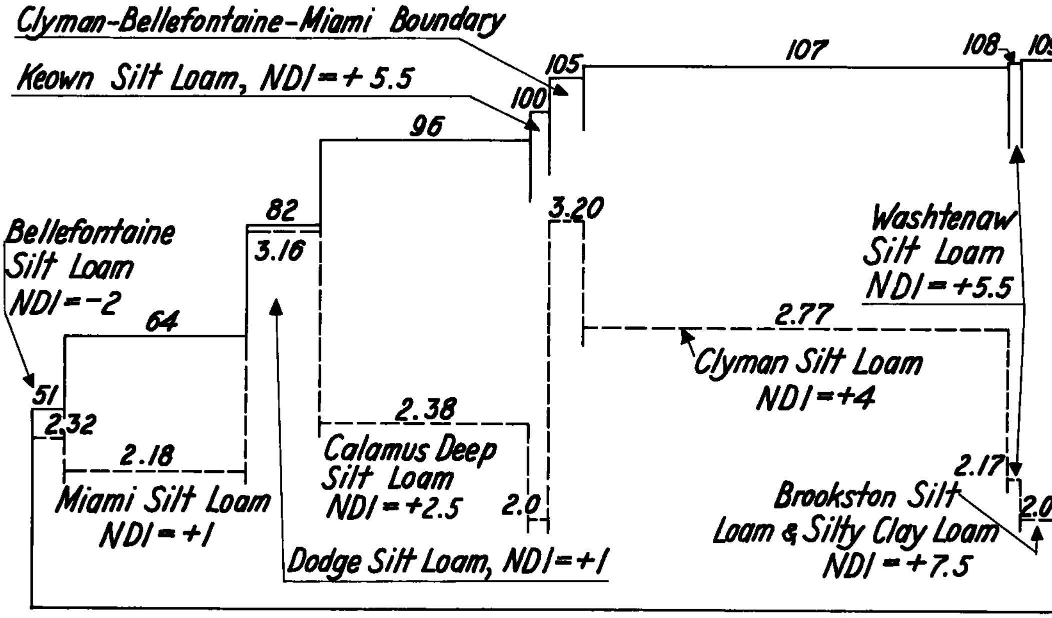


Figure 6. Intensity of pavement cracking related to soil series and drainage - section with transverse joints at 50-foot intervals. Solid lines and corresponding numbers indicate longitudinal cracking in percentage of total length; broken lines and corresponding numbers indicate transverse cracking expressed as average number of cracks per 50-foot slab length; NDI is the natural drainage index, expressed numerically.

other sections, no resurfacing was required at all. Therefore three classes of pavement performance are suggested: (1) no resurfacing, (2) bituminous mat only, and (3) gravel lift course plus bituminous mat.

When the locations of these classes of pavement performance are compared with the soils map (Figure 7), it can be seen that with few exceptions, the portions requiring no resurfacing are on "well-drained" soil. The portions requiring only the bituminous mat were resting on soil types that were "moderately well drained," while the portions requiring the base course as well as the mat were on soil types ranging from "imperfectly" to "very-poorly drained."

The third of these studies was based on a comparison of present pavement conditions with those observed 10 years ago. In 1944, A. T. Bleck of the Wisconsin Highway Commission observed pavement conditions at many places over the state, noting the type of soil as well as general and quantitative information about the pavement (4, 5). These observations and accompanying photographs were made available to the author through the courtesy of Bleck. Several sites were selected for inspection in the spring of 1954, covering a considerable range of soil types. The amount of pavement cracking as well as other evidence of distress was noted, and another photograph taken for comparison.

Because of variations in pavement design, traffic volumes, and local climatic conditions, it is not possible to make such clear-cut comparisons of performance as in the two studies mentioned previously. However, the general trend is similar, with pavements over sand showing the least increase in cracking compared with 1944, and those over silts and clays, poorly drained, showing the most cracking, or even the need for resurfacing, as indicated in the second study. Conditions at two of the seven observation sites are described in the following paragraphs to illustrate this trend.

Figures 8 and 9 illustrate the conditions in 1944 and 1954, respectively, on Plainfield sand (excessively drained, -10). From the photos and recorded observations, it can be seen that cracking has progressed only moderately in the past 10 years. Commercial traffic volume doubled in the same period.

Figures 10 and 11 demonstrate the differences between "well-drained" soils and "poorly drained" soils. The 1944 photo includes Clyde series soil in the foreground (very-poorly drained), and Fox series soil in the background (well drained). In 1944, there was no appreciable difference in the observed behavior of the pavement over the two soils. In 1954, however, quite a change was apparent. The pavement over the Clyde soil has been covered with both a gravel lift and bituminous mat, while the pavement over the Fox soil has not required covering, although there was more cracking in 1954 than in 1944. Again, traffic volumes had about doubled in the intervening period.

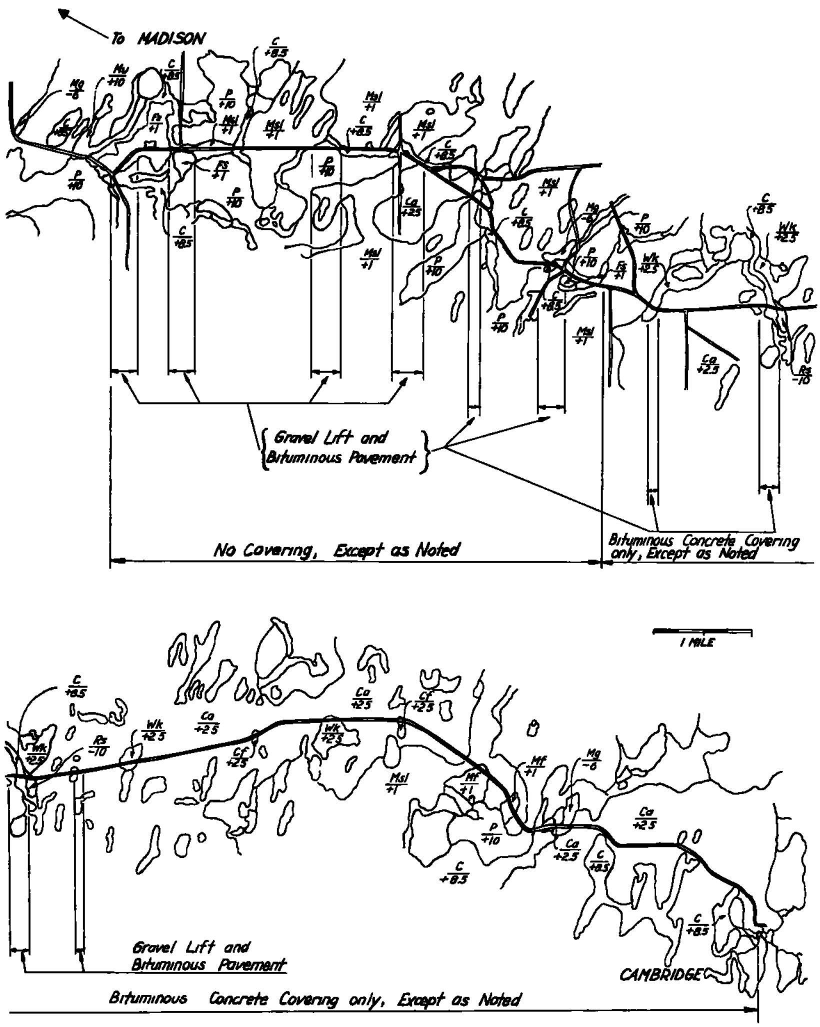


Figure 7. Relationship of soil series and drainage to resurfacing requirements. Original portland cement paving about 1926 resurfacing as indicated in 1952, base map adapted from Dane County Soil Report and Map, Wisconsin Geological and Natural History Survey. Soil-mapping legend: C, Clyde silt loam; Ca, Carrington silt loam; Cf, Carrington fine sandy loam, Fs, Fox silt loam; Mf, Miami fine sandy loam; Mg, Miami gravelly sandy loam; Msl, Miami silt loam; Mu, Muck; P, Peat; Rs, Rodman gravelly sandy loam; and Wk, Waukesha silt loam. The number beneath the mapping symbol is the natural drainage index, expressed numerically.

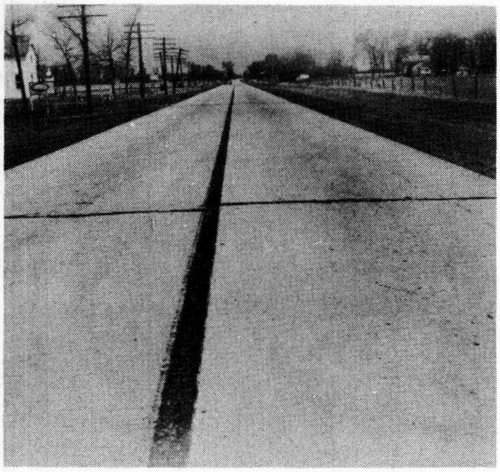


Figure 8. Pavement constructed on Plainfield sand in 1927 without transverse joints - photo taken in 1944 by A. T. Bleck; some transverse cracking at intervals of 120 and 66 feet.

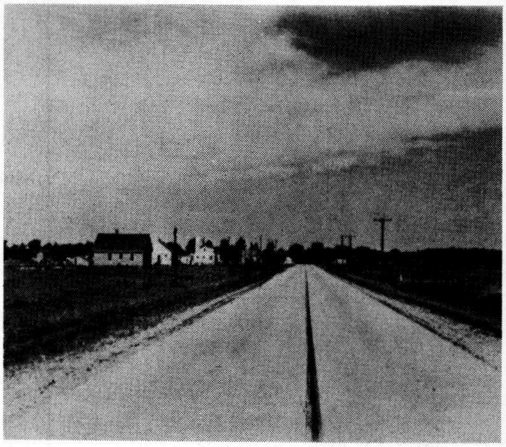


Figure 10. Pavement constructed in 1927 without transverse joints, Clyde soil in foreground, Fox soil in background; crack interval ranges from 15 to 50 feet or more. Photo taken in 1944 by A. T. Bleck.

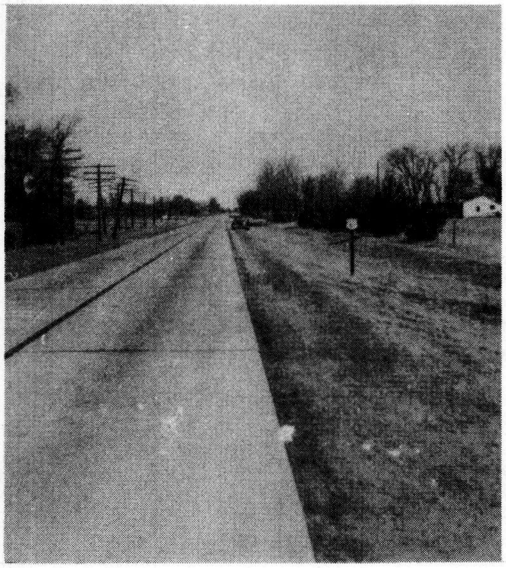


Figure 9. Same pavement as pictured in Figure 8. Pavement rode well in 1954, appeared to be in good condition; in some places, new cracks formed 10 to 15 feet back from older cracks.

Figure 11. Same pavement as pictured in Figure 10. Pavement in immediate foreground is over Miami soil; bituminous pavement and lift course has been placed over pavement constructed on Clyde soil; pavement has not been covered where it rests on Fox soil. Photo taken in 1954.

The traffic volume was somewhat greater than in the case of the pavement constructed over the Plainfield sand.

Figures 10 and 11 also supplement Figure 7 in illustrating the second study.

CONCLUSIONS

The studies upon which this paper is based indicate: (1) there is a reasonably good correlation between the natural drainage index of a soil series and recognized engineering classifications, such as the AASHO classification, the Corps of Engineers frost-susceptibility rating, and pumping criteria; (2) there is a good correlation between the drainage index and actual pavement performance as measured by the observed intensity of cracking of a rigid pavement over several soil types, by resurfacing requirements, and by the observable changes over a period of 10 years; and (3) the drainage-index concept, supplemented by the soil-association concept, may offer an expedient means of organizing the many soil series into workable classifications for engineering purposes.

It is further indicated that the soil textural class is a fairly reliable indicator of the relative susceptibility of a given soil to frost action and pumping action. In this respect, it is important to consider each horizon separately, and not to use the class indicated by the soil mapping unit, which refers to the texture of the surface soil only. Careful study of the soil profile description will indicate the texture of each horizon, as well as other significant information about the soil profile.

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Plate-Bearing Study of Loss of Pavement Supporting Capacity Due to Frost

WILLIAM C. SAYMAN, Engineer, Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers

This paper summarizes field studies of the effect of frost action on the magnitude and duration of loss in pavement supporting capacity, measured by plate-bearing tests, as part of a comprehensive frost-investigation program initiated by the Corps of Engineers in 1944. The studies represent one phase of continuing field investigations initiated in the fall of 1950 at the Frost Test Area, Loring Air Force Base, Limestone, Maine, to improve methods for the prediction of the effects of frost action. The Frost Test Area, 30 by 40 feet in plan, consists of four test sections with various combined thicknesses of pavement and base course constructed on a natural gravelly sandy clay subgrade.

Plate-bearing tests at the Frost Test Area indicate the following results for the reported investigational period: (1) succeeding years of freezing and thawing decreased the normal period (fall) pavement supporting capacity progressively during successive years; (2) the quantitative loss of pavement supporting capacity is the same for each test section during frost-melting period when compared to the normal period supporting capacity for the specific test sections; (3) the duration of loss in pavement supporting capacity is approximately four months as measured by static loading test and about three months when measured by repeating load tests; (4) all repeating-load tests show good agreement, whereas static-load tests at locations where they are preceded by repeating-load tests indicate considerably less loss in pavement supporting capacity when compared with the test results from the static load test locations.

● IN northern latitudes, as well as in the Arctic and Subarctic regions, freezing and thawing of soils cause considerable damage to roads and airfields. Constantly increasing wheel loads and speeds of trucks and aircraft on roads and airfields, together with the demand that these facilities be kept in usable condition at all times, have brought about the need for improved design and construction of pavements for both airfields and roads. Freezing of the ground, especially where frost-susceptible soil types are present and water is readily available, will cause ice segregation resulting in possible non-uniform heave which in turn may cause damage to the pavement surface. The most severe damage associated with frost action occurs during frost-melting periods when the supporting soil loses a large proportion of its strength and may become practically liquid due to the melting of the segregated ice in the soil. With the strength of the supporting soil reduced, frost boils, pumping and pavement breakup may occur necessitating either halting of all traffic or restricting the traffic to lighter wheel loads.

To develop pavement design and evaluation criteria for such frost conditions, the Arctic Construction and Frost Effects Laboratory of the New England Division was established in 1944 by authority of the Chief of Engineers, Department of the Army, and was assigned the responsibility of carrying out the investigations under the supervision of the Airfields Branch, Engineering Division, Military Construction, Office of the Chief of Engineers. Extensive field investigations consisting of traffic tests, plate-bearing tests, California Bearing Ratio tests and supplementary observations have been conducted at various airfields in the northern part of the United States and extensive field data have been assembled to aid in the development of criteria.

The available field data, however, indicated the need for further field investigations under controlled conditions to determine the magnitude and duration of loss of pavement supporting capacity due to frost action as measured by plate bearing tests and to obtain temperature, ground water and heave data with the objective of improving methods for the prediction of the effects of frost action. To attain these objectives, a frost test area was constructed at the Loring Air Force Base, Limestone, Maine, during

September 1950 and a testing program, which was initiated upon completion of the test area, has been carried out continuously to the present time.

Studies, which are being conducted under controlled conditions, are intended to give a clearer understanding of frost action under field conditions and should result in the development of improved criteria for design and evaluation of pavements for airfields and highways. With the facilities available, a comparison of test results at selected points may be made on a year-round basis and the continuing study should encompass a range of climatic conditions.

The present paper is devoted mainly to a summary of the results of the plate-bearing tests which were performed to measure the magnitude and duration of reduction in pavement supporting capacity due to frost action.

DESCRIPTION OF TEST AREA

The Frost Test Area, shown in Figure 1, is located at Loring Air Force Base, Limestone, Maine. The 40- by 30-ft. test area consists of four test sections, fourteen by eighteen feet, with base courses of 7, 12, 18 and 24 inches, respectively, over a natural glacial till subgrade. The base course consists of a lower layer of sandy gravel and an upper layer of crushed rock which was choked, rolled, and paved with a double surface treatment of sand and tar. In the fall of 1952, the area was resurfaced with approximately one inch of hot-mix asphaltic concrete to eliminate depressions and irregularities caused by previous test operations. Drainage swales are provided around the test area to take care of surface runoff water. Subsurface drainage facilities were also incorporated around the area and bisecting the area in the north-south direction. This system is built of perforated corrugated metal pipe installed in the trenches and connected to a water supply well in which the water level may be controlled to either

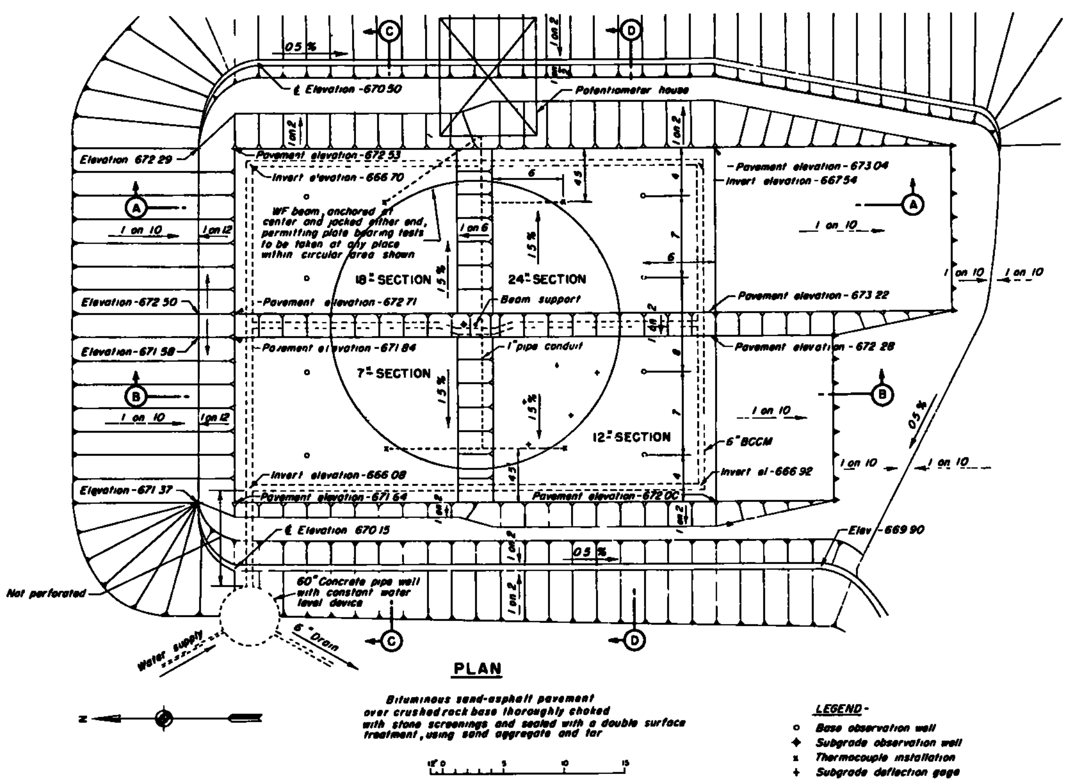

Figure 1.

drain the test area or maintain the ground water elevation at any height in the subgrade or the lower part of the base.

The subgrade material at the test area exposed after removal of topsoil and badly weathered subgrade soil was a gravelly sandy clay (CL) glacial till with an average liquid limit of 21 and a plasticity index of 6. This material is classified as of high frost-susceptibility in accordance with the classification system adopted as one factor to aid in in the comparison of the relative frost-susceptibility of soils subjected to standard laboratory freezing tests.[1]

The thickness of upper and lower base course materials in each of the four test sections are as follows:

Total Base Thickness Inches	Upper Base Crushed Rock Inches	Lower Base Sandy Gravel Inches
7	4	3
12	6	6
18	6	12
24	6	18

Copper-constantan thermocouples were installed in each test section to observe subsurface temperatures at depths of 0.25, 0.50, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0 and 7.0 feet beneath the pavement surface. Each group of thermocouples was encased in water tight plastic tubing and the measuring tip of each thermocouple was hermatically sealed to insure a water tight unit. The thermocouple leads terminated at temperature measuring units located in an instrument house adjacent to the test area.

Observation wells were installed in each section to observe fluctuation in the ground water level of both the base course and the subgrade.

Subgrade deflection gages were installed in each section to measure the subgrade deflection at five plate loading test locations. These gages, consisting of steel rods attached to four square inch square plates, rest on a sand cushion on the subgrade surface and the steel rod is encased in a pipe sleeve extending from approximately ¾ inch above the steel plate to the pavement surface. The space between the gage rod and the pipe sleeve was packed with grease to permit free movement of the rod and to prevent water from entering, freezing, and hampering the action of the gage. The gages were located in the same relative location and similarly numbered in each section.

Reaction for the plate loading tests is provided by a 27-inch WF steel beam anchored at the center and free to rotate horizontally over the areas of the test sections where the plate bearing tests are conducted. Reaction is obtained by cribbing one end of the beam and jacking against the other end. The beam is supported by a yoke which in turn is fastened to a 3¼-inch-diameter steel rod grouted in a hole extending 20 feet into bedrock. The surface of the bedrock is 20 feet beneath the surface of the test area.

Additional installations at the test area consist of five heave reference points installed in each section and a bench mark installed adjacent to the area for use as a reference in heave measurements.

FIELD PLATE BEARING TESTS

Test Procedures

Upon completion of the construction of the test area in September 1950, the subgrade material was saturated by adjusting the water level in the water supply well to control the ground water level slightly above the subgrade surface. The water level was maintained at this position until freezing started and was then lowered to and maintained at an elevation slightly below the subgrade surface, until the middle of August 1952. Thereafter, the ground water was allowed to stabilize at its natural level which fluctuates between 11 and 14 feet below the subgrade surface.

Field investigations were initiated in the latter part of October 1950, which included

[1] Haley, James F., "Cold Studies of Frost Action in Soils, A Progress Report," Soil Temperature and Ground Freezing, Highway Research Bulletin 71, Washington, D. C., 1953, pp, 1-18.

an initial series of plate bearing tests to determine normal period values of pavement supporting capacity. Plate-bearing tests were discontinued during the freezing season; however, observations were continued periodically of ground water, temperature and heave. Plate-bearing tests were resumed at the start of the frost melting period and continued periodically until the next freezing season. Prior to the freezing and during the frost melting season of each year, test pits were excavated to observe ice segregation, obtain moisture and density data and to perform CBR tests.

The plate-bearing tests were performed on the pavement surface using a 30-inch diameter plate. A thin layer of Ottawa sand was used to seat the bearing plate on the pavement. Plate deformations were measured by three dial extensometers placed 120 deg. apart on the outer edge of the plate. Subgrade deflection was measured by a single dial extensometer placed on the subgrade deflection gage through a hole in the center of the plate. All four extensometers were attached to a steel beam supported approximately eight feet from each side of the bearing plate. A jack stand was used to distribute the load over the plate and to provide a space for the extensometer placed over the subgrade deflection gage. The load was applied by jacking against the 27-inch WF beam with a 50-ton hydraulic jack controlled by an electrically driven hydraulic pump. A ball and socket joint, between the jack and beam, reduced eccentricity of loading on the bearing plate. A 500-lb. seating load was used at the start of each plate-bearing test.

Two types of plate-loading tests were conducted at each test section, namely: static load tests at two locations, repeating load tests at two locations and a repeating load test followed by a static load test at a fifth location.

The static load tests were performed by loading the plate in approximately five equal increments with each load increment held constant and the deflection of the plate and

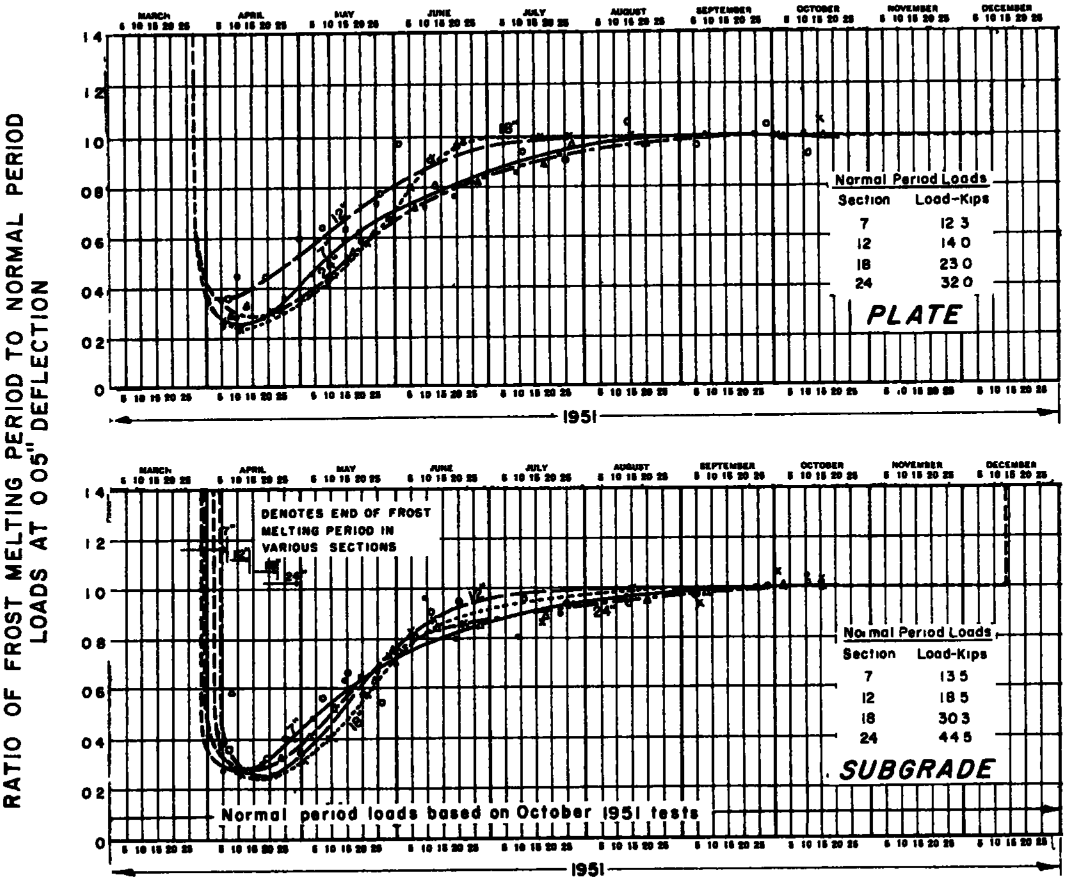

Figure 2.

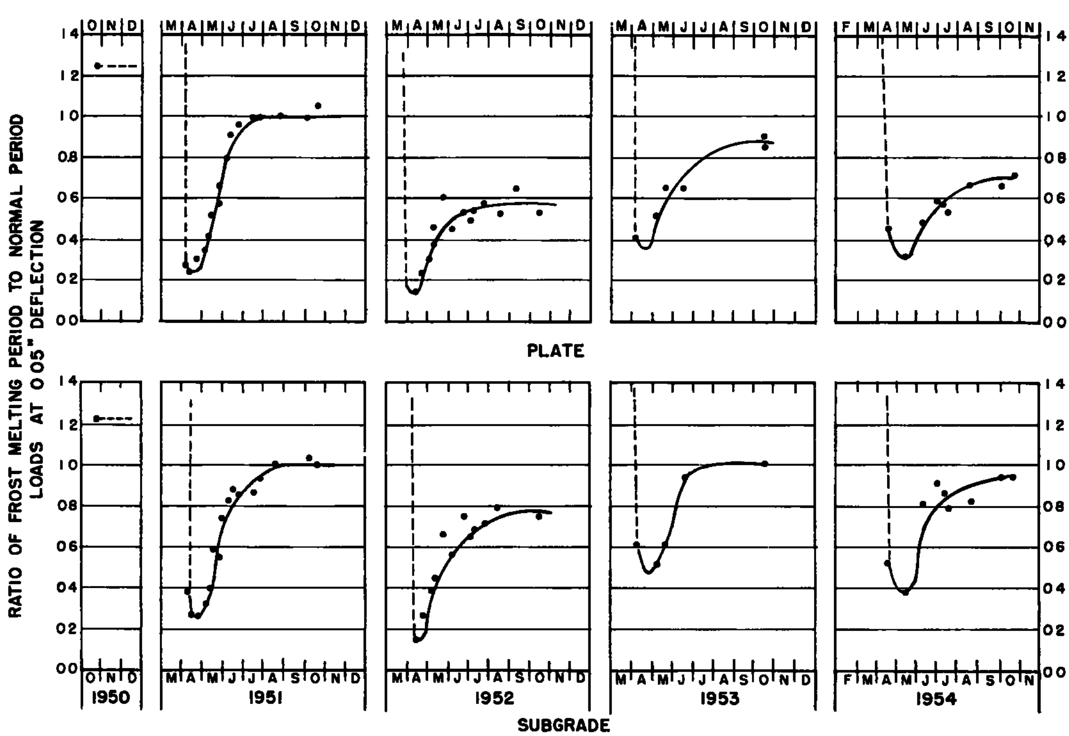

Figure 3. Static-load tests, 18-inch section. Note: all ratios based on 1951 normal period loads.

subgrade recorded when the rate of plate deflection became less than 0.004 inches per minute. A maximum load of 50,000 lb. was used regardless of deflection for the initial normal period static load tests of October 1950 due to difficulties with the anchorage of the loading beam. For all tests, thereafter, the plate was loaded to either a maximum of 60,000 lb. or to a deflection of 0.200 inches, whichever was attained first.

The procedure for conducting the repeating load tests was to subject the 30-inch-diameter plate to 30 loading cycles in a period of 15 minutes. A specified load was rapidly applied in one increment, held constant for a period of approximately 20 seconds and then rapidly released. The deflection of the plate and subgrade was determined after the 1st, 5th, 10th, 20th, and 30th repetition of load. The permanent deflection was determined 10 minutes after the release of the repetition of loading. The following constant loads were selected for the various test sections:

Test Section Inches	Test Load per Repetition Pounds
7	10,000
12	15,000
18	30,000
24	50,000

Climatological Factors

During the investigational years, complete weather data were obtained from the Loring Air Force Base Weather Station at Limestone, Maine. Freezing indexes for the years 1950 through 1953 were 1,529, 1,926, 1,647 and 1,737 degree-days as compared to the ten year normal of 2,417 degree-days at the Caribou, Maine Weather Station located approximately 10 miles southwest of the test area. Differences in the seasonal average values of the other factors for the periods of test were so small that evaluation of each factor was not possible.

Results and Conclusions

In order to depict the seasonal changes in pavement supporting capacity, the ratios of frost melting period to normal (fall) period static loads required to deflect the bearing plate 0.05 inches are plotted against time. The repeating load tests are compared by plotting deflections under the 30th load repetition for each test series. A typical example of static load ratios for the four test sections is shown in Figure 2. The top plot illustrates the ratios based on deflections of the bearing plate (or pavement) and the lower plot shows the ratios based on deflections of the subgrade during 1951.

The ratios of frost melting period to normal period static load tests at the 18-inch base thickness section are summarized in Figure 3 for the investigational years from 1950 through 1954. These plots are typical of the ratios obtained at the other sections of the test area. The ratios for the various years shown thereon are based on the normal period loads required for 0.05 inches deflection during the fall of 1951.

It may be seen in Figure 3 that the load required to cause 0.05 deflection in the fall season progressively decreased from 1950 to 1952. The ground-water table in this period was controlled slightly below the subgrade surface. Allowing the ground water to seek its natural elevation and adding approximately an inch of pavement in the latter part of 1952 appears to have caused an increase in the 1953 normal static-period load ratios, which thereupon progressively decreased with succeeding years of study.

The plots also indicate that the loads required to cause 0.05 inch pavement or subgrade deflection at the time of maximum weakening ranged from 15 to 30 percent of loads required to cause 0.05 inch deflection during the normal period following a particular frost melting period. Furthermore, for the soil types and conditions existing at the test area, it appears that approximately 4 months are required for complete pavement recovery to normal supporting capacity. After reaching the maximum degree of weakening during the frost melting periods, strength recovery took place at a fairly rapid rate until the degree of pavement supporting capacity was approximately 10 percent below the normal value, then the recovery was quite gradual.

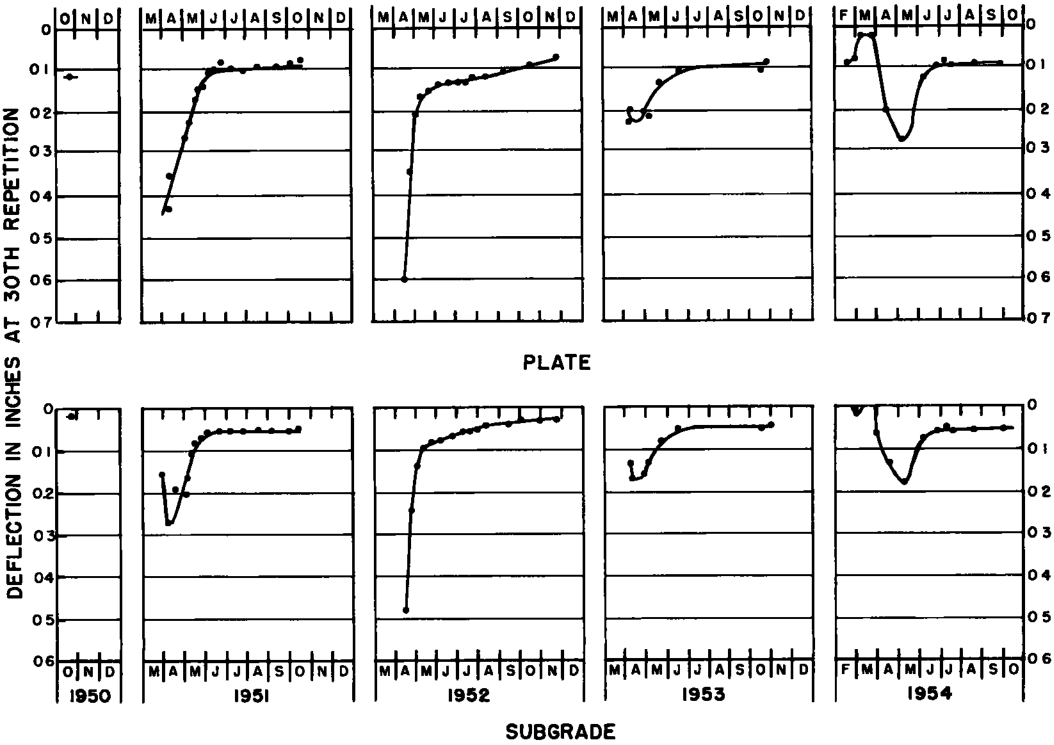

Figure 4. Repeating load tests, 18-inch section.

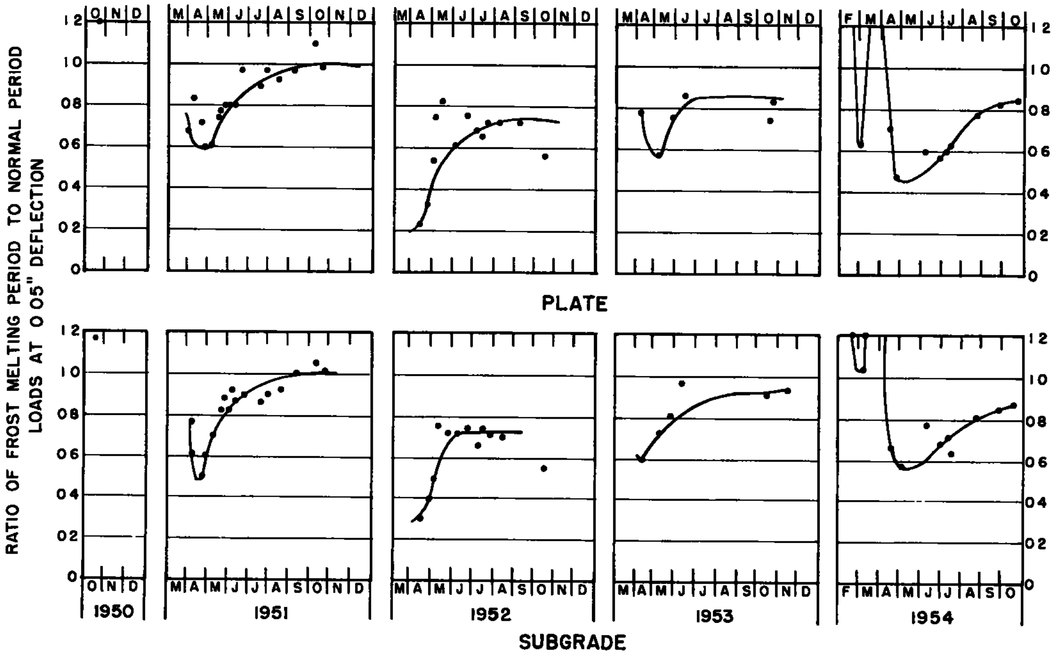

Figure 5. Static load tests preceded by repeating load tests, 18-inch section. Note: all ratios based on 1951 normal period loads.

Plots of the results of the repeating load tests at the 18-inch base thickness section shown on Figure 4 are typical of the results at the other sections. It was found that the pavement or subgrade deflections increase with increased number of repetitions up to 30 during the frost melting period but during the normal period increase in deflections was practically negligible after the first repetition of loading. The recovery of pavement supporting capacity is observed to be more rapid at repeating load test locations than at static load locations.

At locations where repeating load tests were followed by static load tests, it was found that the repeating load tests tended to consolidate the subgrade as shown on Figure 5 by the lesser deflections measured by static load tests at the 18-inch section for this test series as compared with the deflections obtained at the static load locations shown on Figure 3. The load required to cause 0.05 inch deflection in the normal period, however, similarly decreased progressively with succeeding years of tests. Approximately 50 to 60 percent reduction in the pavement supporting capacity of the static load test locations was noted at locations where static load tests were preceded by repeating load tests.

The results of the repeating load tests at the combined repeating load test and static load test locations at the 18-inch section, shown on Figure 6, indicate good agreement with the test results on Figure 4 for repeating load tests alone. These plots are typical of the results measured at the other sections of the test area.

The static and repeating-load plate-bearing test results indicate the magnitude and duration of subgrade weakening for the particular soil and loading conditions but do not necessarily provide a reliable measure of the reduction in the wheel load supporting capacity of pavements.

The consolidation of the subgrade soil by the repetitive loads in the repeating load tests apparently accelerated the rate of regain of subgrade strength as compared with the static load tests in which there was only one load application. Traffic, because of frequency of repetitions would also tend to consolidate and speed up the regain of subgrade strength. However, the traffic would possibly have a more adverse effect on the subgrade in its weakened condition during the frost melting period. The suddenly applied and short duration of the traffic load at a specific location would, for a saturated subgrade, result in a large increase in pore water stress with little or no increase in effective stress.

At the frost-test area, the major effect of repetitive loading appears to be consolidation as illustrated by the appreciably smaller loss of supporting capacity determined from the static-load tests at the combined repeating and static load test locations as compared to the static load test locations. The duration of loss in pavement supporting capacity determined by static load tests appears to be of the same length of time regardless of whether or not repeating load tests are performed at the location. All of the repeating load tests indicated a shorter duration or loss in pavement supporting capacity.

Reduction of normal period loads through succeeding years of test indicate that each year's freezing and thawing cycle has altered the structure of the glacial till subgrade at the froct-test area.

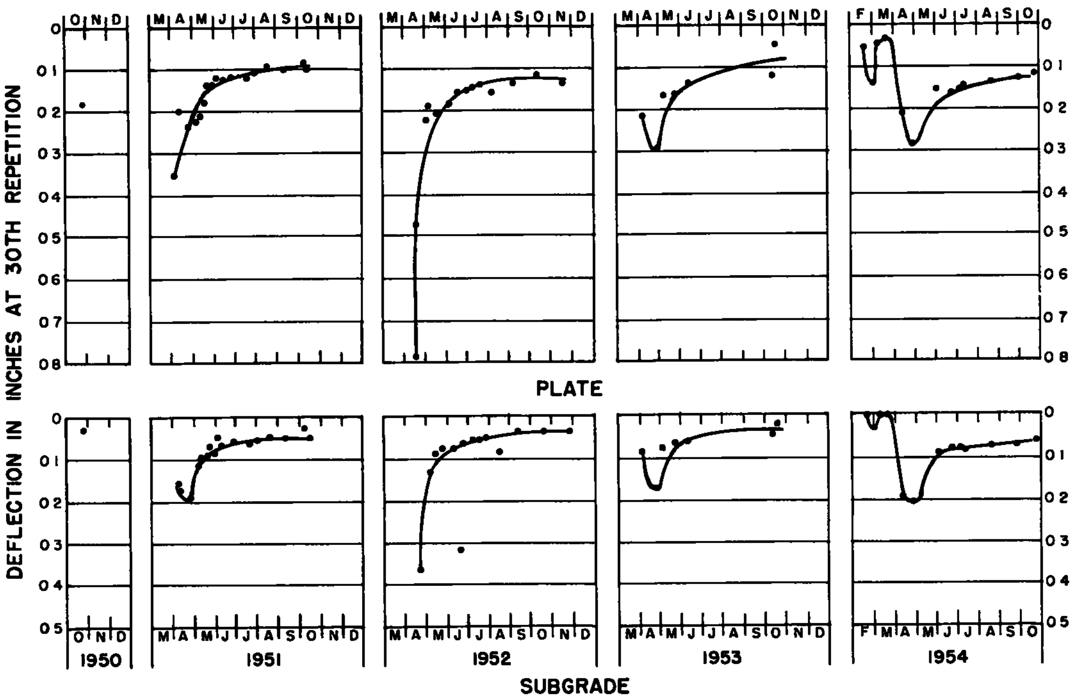

Figure 6. Repeating load tests followed by static load tests, 18-inch section.

Correlation of these data from the field studies with other frost investigational studies will result in more positive methods of determining the effects of frost action and the magnitude and duration of loss of pavement supporting capacity due to frost action. Also improved design and evaluation criteria will be developed resulting in improved and more dependable highway and airfield pavements.

Frost and Permafrost Definitions

The following list of terms used in current literature on Frost and Permafrost has been prepared and approved by the Highway Research Board Committee on Frost Heave and Frost Action in Soil. Special credit is due Frank Hennion, chairman of the subcommittee which prepared the list for approval by the main committee.

GENERAL

Arctic. The northern region in which the mean temperature for the warmest month is less than 50 F. and the mean annual temperature is below 32 F. In general, arctic land areas coincide with the tundra region north of the limit of trees.

Subarctic. The region adjacent to the Arctic in which the mean temperature for the coldest month is below 32 F., the mean temperature for the warmest month is above 50 F., and where there are less than four months having a mean temperature above 50 F. In general, subarctic land areas coincide with the circumpolar belt of dominant coniferous forest.

Break-up period. The period of the spring thaw during which the ground surface is excessively wet and soft, and ice is disappearing from streams and lakes. Duration of the break-up period varies from one to six weeks, depending on regional and local climatic conditions.

Freeze-up period. The period during which the ground surface freezes, and during which ice cover is forming on streams and lakes. The duration of the freeze-up period varies from one to three months, depending on regional and local climatic conditions.

Frost-melting period. An interval of the year during which the ice in the foundation materials is returning to a liquid state. It ends when all the ice in the ground has melted or when freezing is resumed. Although in the generalized case there is visualized only one frost melting period, beginning during the general rise of air temperatures in the spring, one or more significant frost melting intervals may occur during a winter season.

Normal period. The time of the year when there is no reduction in strength of foundation materials due to frost action.

Period of weakening. An interval of the year which starts at the beginning of the frost-melting period and ends when the subgrade has begun to regain its strength.

SOIL-AND-FROST

Permafrost. Perennially frozen ground.

Suprapermafrost. The entire layer of ground above the permafrost table.

Permafrost table. An irregular surface which represents the upper limit of permafrost.

Annual frost zone. The top layer of ground subject to annual freezing and thawing. In arctic and subarctic regions where annual freezing penetrates to the permafrost table, suprapermafrost and the annual frost zone are identical. (Sometimes referred to as active layer or active zone).

Residual thaw zone. A layer of unfrozen ground between the permafrost and the annual frost zone. This layer does not exist where annual frost extends to permafrost.

Frost action. A general term for freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are part or with which they are in contact.

Frost thrust. A lateral displacement due to frost action.

Frost heave. The raising of a surface due to the formation of ice in the underlying soil.

Percent heave. The ratio, expressed as a percentage, of the amount of heave to the depth of frozen soil before freezing.

Frost boil. The breaking of a localized section of a highway or airfield pavement under traffic and ejection of subgrade soil in a soft and soupy condition caused by the melting of the segregated ice formed by frost action.

Frost-susceptible soil. Soil in which significant (detrimental) ice segregation will occur when the requisite moisture and freezing conditions are present.

Non-frost susceptible materials. Cohesionless materials; such as, crushed rock, gravel, sand, slag and cinders in which significant (detrimental) ice segregation does not occur under normal freezing conditions.

Homogeneously frozen soil. A soil in which water is frozen within the material voids without macroscopic segregation of ice.

Heterogeneously frozen soil. A soil in which a part of the water is frozen in the form of macroscopic ice occupying a space in excess of the original voids in the soil.

Ice segregation. The growth of ice as distinct lenses, layers, veins and masses in soils commonly, but not always, oriented normal to the direction of heat loss.

Ice wedge. A vertical wedge-shaped ice mass in permafrost usually associated with fissure polygons.

Ice lenses. Ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss, and commonly in repeated layers.

Ice content. The ratio, expressed as a percentage, of the weight of ice phase to the dry weight of soil.

Tangential adfreezing strength. Unit bond strength between frozen ground or ice and another material.

Pavement pumping. The ejection of water and subgrade soil through joints, cracks, and along edges of pavements caused bv downward slab movement actuated by the passage of heavy axle load over the pavement after the accumulation of free water on the subgrade.

Open system. A condition in which free water in excess of that contained originally in the voids of the soil is available to be moved to the surface of freezing, to form segregated ice in frost-susceptible soil.

Closed system. A condition in which no source of free water is available during the freezing process beyond that contained originallv in the voids of soil.

TEMPERATURE

Average daily temperature. The average of the maximum and minimum temperatures for one day or the average of several temperature readings taken at equal time intervals during one day, generally hourly.

Mean daily temperature. The average of the average daily temperatures for a given day for several years.

Average monthly temperature. The average of the average daily temperatures for a particular month.

Mean monthly temperature. The average of the average monthly temperatures for a given month for several years.

Average annual temperature. The average of the average daily temperature for a particular year.

Mean annual temperature. The average of the average annual temperatures for several years.

Degree-day. The degree-days for any one day equal the difference between the average daily air temperature and 32 F. The degree-days are minus when the average daily temperature is below 32 F. (freezing degree-days) and plus when above (thawing degree-days).

Degree-hour. A variation of one degree Fahrenheit from 32 F. for a period of one hour. The degree-hour is negative if below 32 F. and positive if above 32 F.

Freezing season. That period of time during which the average daily temperature is generally below 32 F.

Thawing season. That period of time during which the average daily temperature is generally above 32 F. Note: The definitions for "freezing season" and "thawing season" are applicable to conditions in arctic and subarctic regions where frequent oscillations about the freezing point are uncommon.

Freezing index. The number of degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season. The index determined for air temperatures at 4.5 feet above the ground is commonly designated as the air freezing index, while that determined for temperatures immediately below a surface is known as the surface freezing index.

Mean freezing index. The freezing index determined on the basis of mean temperatures. The period of record over which temperatures are averaged is usually a minimum of 10 years and preferably 30.

Thawing index. The number of degree days between the lowest and highest points on the curve for cumulative degree-days versus time for one thawing season. It is used as a measure of the combined duration and magnitude of above-freezing temperatures occurring during any given thawing season. The index determined for air temperatures at 4.5 feet above the ground is commonly designated as the air thawing index, while that determined for temperatures immediately below a surface is known as the surface thawing index.

Mean thawing index. The thawing index determined on the basis of mean temperatures.

Correction factor. The ratio between the surface index and air index for either freezing or thawing.

HEAT TRANSFER

Thermal regime. The temperature pattern existing in a body.

Thermal conductivity. The time rate of heat flow through unit area of a substance under a unit temperature gradient. Common units are Btu. per hour per square foot per degree F. per inch or foot of thickness.

Thermal resistivity. The reciprocal of thermal conductivity.

Thermal conductance. The time rate of heat flow through a substance for an area of 1 square foot and a difference of temperature of 1 F. between surfaces.

Thermal resistance. The reciprocal of thermal conductance.

Volumetric heat capacity. The number of Btu. necessary to raise the temperature of 1 cubic foot of a material 1 F.

For dry soils it is cd; for wet soils it is $d\left(c+1.0\,\frac{w}{100}\right)$,

and for wet, frozen soils it is $d\left(c+0.5\,\frac{w}{100}\right)$,

where c = specific heat of the dry material
d = dry density of a soil in lb. per cu. ft.
and w = water content of a soil in percent of dry weight.

Diffusivity. An index of the facility with which a material will undergo temperature change. It is numerically equal to the quotient of the thermal conductivity and the volumetric heat. The diffusivity of a soil is increased by freezing, by an increase of moisture, and by an increase in density.

Latent heat of fusion. The number of Btu. necessary to melt one pound of ice without a change in temperature.

Volumetric latent heat of fusion. The number of Btu. necessary to melt the ice in 1 cubic foot of soil without a change in temperature.

Specific heat of soil. The number of Btu. necessary to raise the temperature of one pound of dry soil 1 F.

TERRAIN

Patterned ground. A general term describing ground patterns resulting from frost action such as soil polygons, stone polygons, stone circles, stone stripes, and solifluction stripes. The most common type of soil polygon is known as a fissure polygon.

Creep. Extremely slow downslope movement of superficial soil or rock debris usually imperceptible except to observation of long duration.

Solifluction. The perceptible slow downslope flow of saturated non-frozen soil over a base of impervious or frozen material. Movement occurs primarily when melting of segregated ice or infiltration of surface runoff results in concentration of excess water in the surface soils.

Frost mound. A localized upwarp of land surface caused by frost action or hydrostatic pressure.

Pingo (hydrolaccolith). A large frost mount not uncommonly a hundred feet high or more containing a core of ice.

Icing. A surface ice mass formed by freezing of successive sheets of water.

Tundra. A treeless region of grasses and shrubs characteristic of the Arctic.

Muskeg. A shallow, poorly drained, peat-filled depression supporting bog vegetation.

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