Frost-Susceptibility Ratings and Pavement Structure Performance

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A three-year study of the relations between flexible pavement performance, design methods, materials properties, and environmental factors recently completed by the Alaska Department of Transportation and Public Facilities is described. The pavement sections chosen for the investigation were 120 older sections from the various climatic regions of the state. Performance was characterized by measurements of fatigue cracking, rut depths, and peak springtime deflection levels. Pavement structures were measured and sampled to a depth of 4.5 ft. Sample testing included gradations, Atterberg limits, moisture contents, and frost-susceptibility-related factors. Additional information from previous frost-heave testing programs was also used to supplement the results of the performance study. Conclusions are presented on the relation between frost-susceptibility indicators and performance. Results of the study indicate that low contents of particle sizes smaller than 0.075 mm (no. 200 sieve) and 0.02 mm in unstabilized pavement structural layers may be the most important of the many factors that affect structural performance. Classifications and analysis of pavement-layer soils and systems by the U.S. Army Corps of Engineers frost-susceptibility system and the reduced-subgrade-strength design method showed significant relations with pavement performance in Alaska, whereas use of the stabilometer R-value method in testing and design analysis was of no value in indicating relative performance levels.

A major study was recently completed that compared the field performance of Alaska's flexible highway pavements with the physical properties of the underlying aggregate and soil layers. The primary intent of this study was to evaluate the reliability of existing design procedures and to determine the maximum contents of particles smaller than 0.075 mm and 0.02 mm that could be allowed at each depth without resulting in excessive thaw weakening and fatigue failure of the pavement system. Soil layers were investigated from the bottom of the asphalt pavement to a depth of 4.5 ft.

Highway pavement structures in Alaska are currently designed by a combination of the U.S. Army Corps of Engineers reduced-subgrade-strength design method and the Hveem R-value method (1,2). Subgrade soils are classified by the Casagrande criteria for frost-susceptibility value (FSV), based primarily on their 0.02-mm contents (see Figure 1). Minimal pavement structure thicknesses, termed the "frost overlay", are selected from design curves based on the traffic index (TI) and the FSV classifications of the subgrade soils (see Figure 2). The thicknesses of the pavement and base-course layers are then selected on the basis of R-value test data. Current specifications restrict the -0.075-mm contents of base and subbase layers to 6 percent or less, to reduce the frost susceptibility of these layers, although specifications in past years have permitted -0.075-mm contents as high as 10-12 percent.

Results obtained from this study and from prior laboratory frost-heave testing of base and subbase materials have led to a series of conclusions regarding the relations between soil properties and frost-heave test results and pavement performance in various Alaska environments.

SITE SELECTIONS AND OBSERVATIONS

A total of 120 paved highway sections with unbound granular base layers (each section 0.5-1 mile in length), located throughout the various climatic regions of Alaska, were selected for this study. The roadway sections selected were those that exhibited either good long-term performance or clear evidence of premature fatigue-type distress. Each section was intensively measured for fatigue (alligator) cracking and wheel-path rutting because these factors are considered to be the primary structural performance indicators (2). Alligator cracking was measured separately in each wheel path and is expressed here as a percentage of the full lane length. Rutting was measured at intervals by using a 5.5-ft straightedge. To determine the degree of seasonal thaw weakening, Benkelman beam rebound deflection surveys were made weekly for one month after the start of spring thawing on each section. The Asphalt Institute test procedure was used (3). Additional summer and fall deflection measurements were made to further characterize the sections. Maximum deflection levels were used in this study to indicate the structural strength of each section (4).

To determine the properties of the pavement structures, two test pits were excavated to a depth of 4.5 ft in each study section, and samples and thickness measurements were taken from each layer. Pavement cores were taken from both wheel-path and non-wheel-path areas to determine thicknesses and to correct the surface rut measurements for the effects of pavement wear and displacement.

Environmental factors were determined for each site, including climatological data such as mean annual temperature, precipitation, and mean freezing index (5), as well as age and cumulative traffic. These factors were included in the analysis to determine whether there were significant differences in performance between similar pavement structures in warmer coastal environments and the colder, dry interior regions of Alaska. Mean air temperatures ranged from 22° to 40°F, freezing indices from 300 to 6000 degree-days (Fahrenheit), and annual precipitation from 10 to 80 in. The number of equivalent 18 000-lb axle loadings at the time of site inspections varied from 20 000 to 860 000. Asphalt pavement thicknesses ranged from 1 to 4 in and averaged 1.9 in.

LABORATORY TESTING

All pavement-structure soil samples were tested for particle size gradation, moisture content, and Atterberg limits and were classified for FSV by using Figure 1. Additional tests were performed on selected samples to determine their stabilometer R-values and heave rates.

Heave Rate

Heave-rate testing was performed on 41 base-course and subbase samples selected from study sections that ranged from excellent to poor in performance. Heave test procedures used in this study were developed by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) (6,7) and modified by the Alaska Department of Transportation and Public Facilities over the past 10 years, for evaluating base and subbase aggregates at differing gradations and fines contents. The procedure involves the removal of particles larger than 0.75 in and conditioning to optimum moisture followed by compaction

Figure 1. FSV soil classification chart based on Corps of Engineers system.

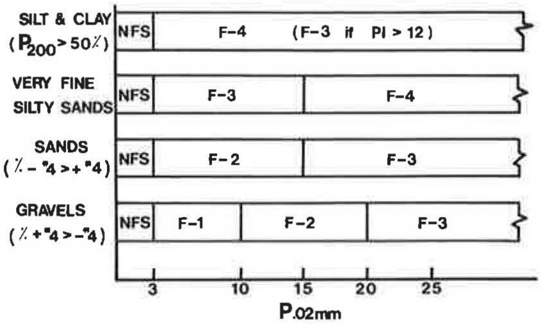
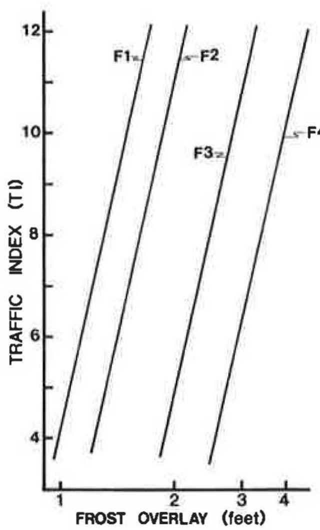


Figure 2. Alaska's pavement design chart for frost-susceptible soils based on soil FSV class and TI.



of the samples with a vibratory hammer in 6-in-diameter by 5.5-in-high segmented ring molds. Samples are then saturated by overnight soaking before freezing. Heave measurements are recorded while applying a fixed +15°F air temperature above the samples for 72 h and maintaining a +40°F temperature and an open water supply beneath the samples (see Figure 3). Samples are classified on the basis of the heave rate occurring between 48 and 72 h after the start of freezing, during which time the freezing rate approximates 0.5 in/day.

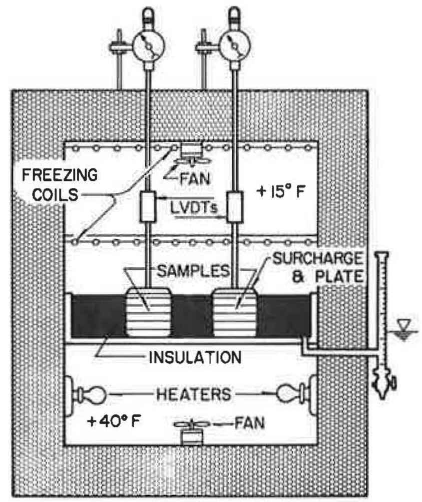
Stabilometer R-Value

For evaluation of the Hveem R-value design method, 25 road sections were selected to cover the full range of distress severity, and samples from each layer were tested to determine the stabilometer R-values in accordance with ASTM D2844.

PERFORMANCE ANALYSIS

Initial analyses to relate performance levels to soil properties at various depths were made by using a computerized multiple regression procedure developed by Kansas University as part of the Statistical Package for the Social Sciences (SPSS) program. The major dependent variables used to describe the performance of each section were percentage of alligator cracking, maximum deflection level, and average rut depth. Although all possible soil property, layer thickness, and environmental param-

Figure 3. Frost-heave test cabinet.



eters were included in stepwise multiple regression analyses, no reasonable correlations were obtained between performance and any combination of variables, in spite of the large number of sections (120) and the large number of independent variables considered for each section (175). As a result, this approach was abandoned in favor of grouping pavement study sections on the basis of similar levels of performance. Average performance levels of these groups were then compared with the average material properties at selected depths beneath the pavement of 3, 9, 18, 30, 42, and 54 in.

Comparisons among the three primary performance factors--namely, fatigue or alligator cracking, rut depths, and peak spring levels of Benkelman beam deflections--indicated very good relations among these variables, as shown by Figures 4 and 5. For this reason, generalized comparisons between materials properties and performance could be based on either maximum spring deflection levels or percentages of alligator cracking. All references in this paper to deflection levels refer to the maximum or "peak" springtime Benkelman beam rebound deflection, calculated as the average of 11 tests on each section plus twice the standard deviation of those tests (3).

Performance Versus Contents Smaller than 0.075 and 0.02 mm

Particles smaller than 0.075 mm [particles passing a no. 200 mesh sieve (P_{200} or fines)] and fractions smaller than 0.02 mm are generally used as the primary indicators of frost susceptibility. Fines-content specifications are most often used to control frost susceptibility in pavement layers due to the ease of testing. To evaluate the effects of P_{200} contents on deflection levels and alligator cracking, the study sections were grouped into six deflection-level ranges and four alligator-cracking ranges. Average P_{200} contents for each group were then determined at each of the six depths mentioned above. Plots of these relations at the 3-in depth are shown in Figures 6 and 7. The best-fit lines in these plots were then used to determine the average fines contents at each depth that corresponded to selected deflection and alligator cracking levels, summarized in Figures 8 and 9.

As a second indicator of the effects of increasing fines contents on reduced roadway strength during the thaw-weakening period, five roadway

Figure 4. Maximum rebound deflection versus alligator cracking: average pavement age =14.7 years and average total EALs = 155 000.

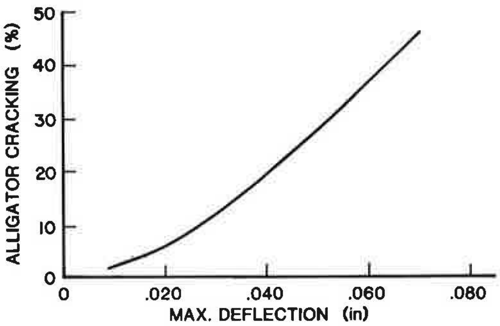


Figure 5. Rut depth versus alligator cracking.

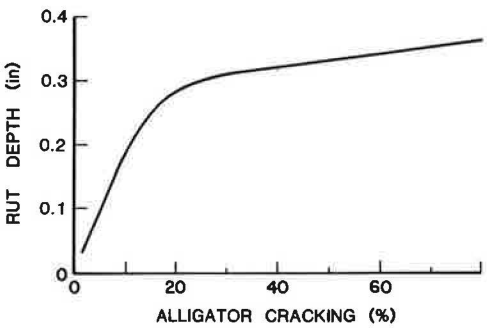


Figure 6. Average P_{200} contents in base course for 120 roadway sections by levels of alligator cracking.

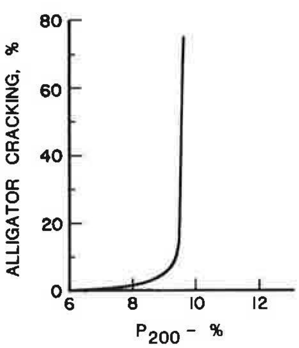


Figure 7. Average P_{200} contents in base course for 120 roadway sections by pavement deflection levels.

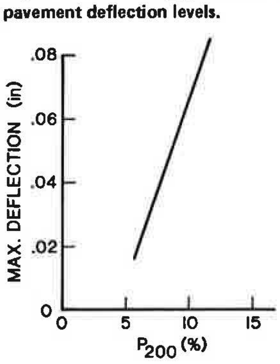


Figure 8. Average P_{200} contents at various depths related to increasing deflection levels.

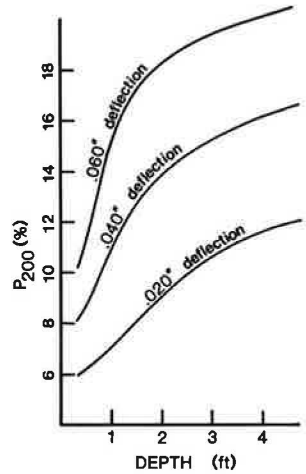
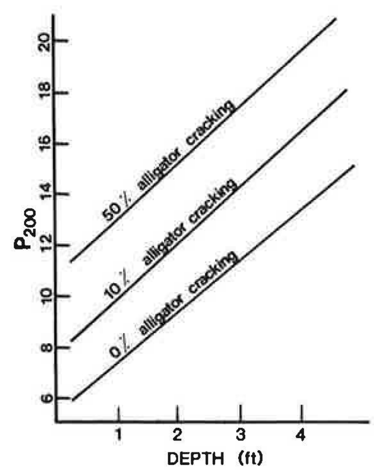


Figure 9. Average P_{200} contents at various depths related to increasing levels of alligator cracking.



sections were selected that had progressively increasing fines contents in the base-course layer and cleaner soils underlying the base to at least 4.5 ft. In these cases, the base course obviously represents the most frost-susceptible layer in the pavement system. All sections of this series showed seasonal deflection histories that had very rapid thaw weakening and rapid strength recovery followed by low summertime deflection levels. Figure 10 shows the peak spring deflection levels for these sections versus the fines contents of the base course and again demonstrates the detrimental effects of increased fines contents on pavement structural performance.

Correlations between performance factors and -0.02-mm contents were similar to, but not quite as significant as, those for the P_{200} contents in spite of the fact that the -0.02-mm particle size is a stronger predictor of frost-heave rates under laboratory conditions (8). The ratio of percentage 0.075-mm to percentage 0.425-mm particle sizes, termed the dust ratio, also had a significant relation with performance. No other particle size or shape factors were observed to significantly affect performance.

The average soil properties associated with those pavement sections that had no alligator cracking were very similar to those associated with maximum deflection levels of 0.020 in and with best overall performance. For the soil samples obtained in this study, the -0.02-mm particle contents averaged approximately 60 percent of the -0.075-mm contents whereas soils with clayey fines were rarely encountered. Therefore, no conclusions could be made on the effects of plasticity index (PI) on performance.

From these observations, the detrimental effects on performance of increasing fines and -0.02-mm contents is obvious. However, the significance of soil properties at the greater depths listed above could not be determined from this study, since this was not a controlled experiment in which single-layer properties could be varied while all other factors were held constant. Inspection of the soils data indicated that, due to geological factors controlling soil distribution, high fines contents at depths of 2-4 ft related to a higher probability that base and subbase layers would also be high in fines.

Performance Versus Soil FSV Classifications

After the soils at the six depths listed above were grouped by similar FSV values, the average maximum deflection values for each FSV group and depth were calculated. The results are shown in Figure 11. The following observations were made from inspection of these relations:

1. Non-frost-susceptible (NFS) (F = 0) soils showed slight performance advantages over classes that have greater frost susceptibility.
2. No significant performance differences are apparent between F-1 and F-2 classes. This may be due primarily to the organization of the frost class chart (Figure 1), where, for instance, nearly identical gravels with a content of 4 percent finer than the -0.02-mm size may be assigned to an F-1 class (gravel) or to an F-2 class (sandy gravel).
3. FSV classes F-3 and F-4 at depths of up to 42 in relate to significantly higher deflection levels than the lower FSV classes. Performance differences between the F-3 and F-4 classes appear insignificant.
4. Overall, the FSV classification system appears only weakly related to pavement deflection levels.

Similar plots of FSV groups by percentage of alligator cracking show even more erratic relations. In the plot, shown in Figure 12, NFS soil layers show major benefits in reduced alligator cracking to a depth of approximately 3 ft, whereas differences in alligator cracking among classes F-1 to F-4 are erratic and appear nonsignificant. In contrast to the intent of the classification system that higher values represent the worst frost susceptibility, Figure 12 appears to indicate that the F-2 soils group may represent the worst FSV classification.

The effects of the various FSV classes on rut depth were similar to those observed for alligator cracking except that NFS soils did not exhibit such a decisive performance advantage.

In general, it was not demonstrated that the existence of a particular FSV class at a given depth can serve as a basis for confident performance prediction because all layers contribute to performance. NFS material is, however, most strongly indicated for use in the base and subbase layers. In view of the favorable performance expectations attached to low deflection levels, this study indicates the use of NFS materials to be of benefit even at depths of 30 in or greater.

Figure 10. Deflections versus P_{200} in base course for selected roadway sections: data from five roads with lower fines in subbase and subgrade.

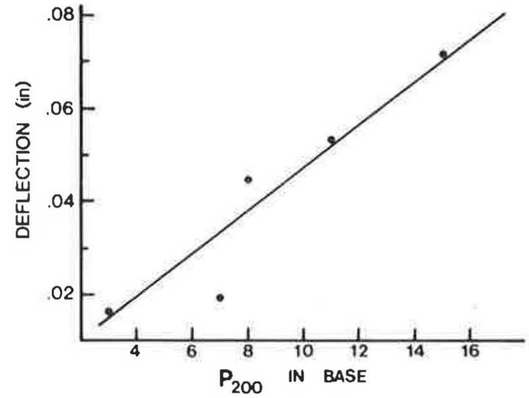


Figure 11. Average deflections related to different soil frost classes at various depths.

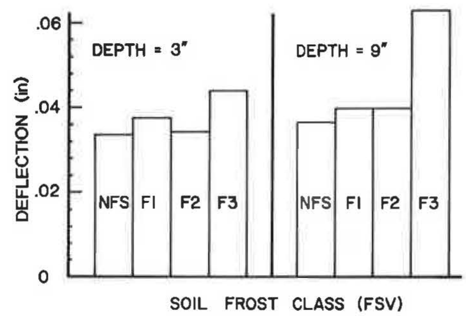
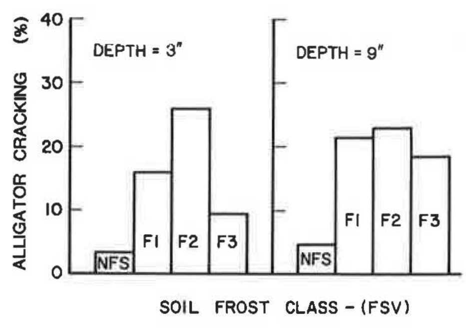


Figure 12. Percentage of alligator cracking related to different soil frost classes at various depths.



Evaluations of FSV Design Method

A second approach to evaluating the frost-susceptibility classification system involved the use of Alaska's current pavement-structure design procedure (Figure 2) to study the adequacy or deficiencies of the existing study-section pavement structures. The TI for each section was first determined from the number of equivalent 18-kip axle loadings (EALs) by the following formula:

$$TI = 9(EAL/10^6)^{0.119} \quad (1)$$

Additional frost overlay (FSV) thickness requirements were then derived by examining the design overlay thickness required by Figure 2 for each soil layer to a maximum depth of 54 in. The actual thickness of the overlying layers was subtracted from the design requirement for the layer being considered, and the remainder, if any, was termed the frost-overlay deficit for that layer. The largest net deficit for all layers in a given pavement structure was then defined as the additional FSV overlay requirement. Of all 120 road sections studied, only 8 were found to require no additional overlay according to the reduced-subgrade-strength design method. By far the most common overlay deficit was in the range of 10-15 in and was required over an F-1 or F-2 soil type. The practice observed prior to 1977 of allowing fines contents as high as 10-12 percent in base and subbase materials accounts for many of the indicated overlay deficiencies.

Following the determinations of the additional FSV overlay requirements, the relations between this factor and the pavement distress factors were investigated by grouping and averaging data, as shown in Figure 13. Significant correlations are apparent between poor performance and FSV overlay deficiencies.

In summary, analysis of the FSV design method indicated generally significant relations between performance and overlay requirements. The currently used FSV overlay design chart may be overly conservative by 15-18 in of overlay for average conditions, since performance did not significantly deteriorate until pavement structures became deficient by more than this amount. However, some conservatism is necessary to allow for worse conditions.

Heave Rates Versus FSV Classifications and Fines Contents

Relations between FSV and laboratory heave rates for 41 tests on samples in the NFS, F-1, and F-2 classes are shown as "quartile plots" in Figure 14. Direct increases in the median heave rates were observed with increased FSV classification number, but the range of heave rates also increases with class, which makes the FSV class system a relatively poor predictor of laboratory heave rate.

The relation of soil fines contents to laboratory heave rate was also investigated. Predictive equations derived by using least-squares linear regression analysis are as follows:

$$\text{Heave rate}_{\text{mm/day}} = (0.36)(\% - 200) + 0.38 \quad (2)$$

$$\text{Heave rate}_{\text{mm/day}} = (1.02)(\% - 0.02\text{ mm}) - 1.01 \quad (3)$$

The coefficient of determination (R^2) values were 0.86 for both equations. These equations provide significant correlations between fines and heave rate and indicate the generalized rate of change of heave rate as fines increase. However, no combination of soil gradation factors could be found that predicted heave rates with satisfactory accuracy.

Previous heave test programs performed by the Alaska Department of Transportation and Public Facilities on relatively clean gravels used for pavement-system subbase and base-course layers have demonstrated relatively uniform increases in heave rate with increases in the -0.075-mm and -0.02-mm percentages. Figure 15 shows the changes in heave rate that occurred when a Fairbanks gravel base-course aggregate that had identical gradations in the +0.425-mm sieve portions was heave tested with five different -0.075-mm and -0.02-mm contents. Test programs on aggregates from several different construction projects have confirmed this comparison between heave rates and varying fines contents.

All heave test runs that used varying fines contents with the same gravel fraction have shown direct relations between increases in -0.02-mm and -0.075-mm sieve contents and heave rate and the occurrence of some significant heave at fines contents greater than 1-2 percent. Laboratory data have consistently shown that the percentage of the -0.02-mm particle size is a better indicator of heave rate for aggregates from different sources than is the -0.075-mm particle size.

Performance Versus Heave Rate

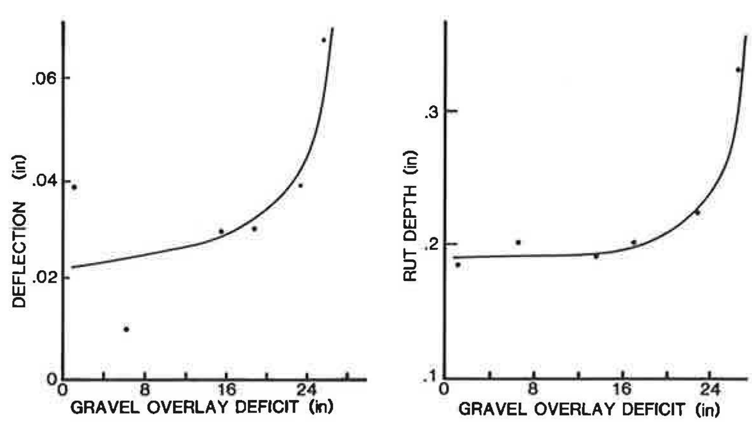
Heave test samples were selected to represent base and subbase materials from 27 pavement study sections that ranged from good to poor in performance and covered the range depth between 0 and 30 in. About half of the sampled soils at each depth had heave rates of less than 3 mm/day.

As Figure 16 shows, only a rough relation of lower heave rates to lower distress levels could be found. For all three distress types, heave rates at 9 and 18 in appeared to be the most consistent indicators of performance. Low alligator cracking, low deflection levels (0.025 in), and minimal rut depths (0.1 in) were associated with heave rates of less than 2-3 mm/day, whereas worst-case performance levels were erratically predicted by heave rates greater than 3-4 mm/day.

Specifications Based on Frost-Heave Testing

Earlier unpublished Alaska Department of Transportation and Public Facilities studies of certain base and subbase layers that had resulted in very rapid fatigue failures of pavements have shown that heave rates in excess of 3.5 mm/day, resulting from -0.02-mm and -0.075-mm sieve contents of greater

Figure 13. Overlay deficiencies by reduced-subgrade-strength design method versus maximum deflections and rut depths, based on group averages from 120 roadways.



than 8 and 13 percent, respectively, were totally unacceptable under wet coastal climatic conditions. Heave rates of 2 mm/day or less under the testing procedure previously described appeared to result in acceptable pavement layer performance. On the basis of these early studies, maximum -0.075-mm sieve contents of base and subbase layers are currently specified at 0-6 percent in an attempt to prevent frost action from causing excessive thaw weakening of the pavement-system layers. With normal sources, this ensures that -0.02-mm particle contents will not exceed approximately 4 percent.

Figure 14. Quartile plots of laboratory heave rates for different FSV classes.

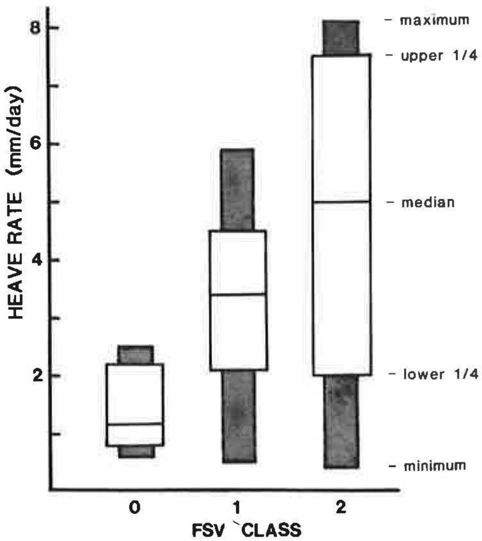
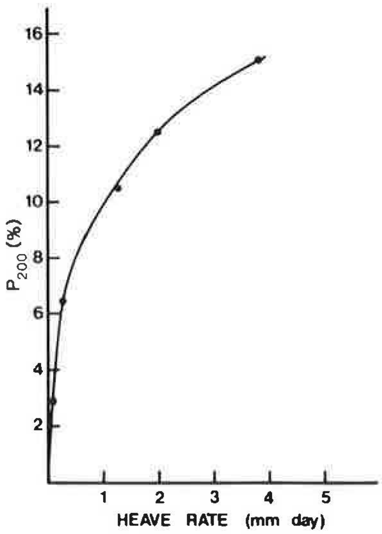


Figure 15. Changes in laboratory heave rate from progressive increases in P_{200} content in a Fairbanks gravel.



Higher laboratory heave rates result from increased moisture gain on freezing and would be expected to result in increased and prolonged strength losses on thawing. Laboratory heave-rate criteria have not been incorporated into any form of pavement structural design method or project specification at this time. However, heave-rate differences between different aggregates would appear to relate more directly to structural performance during the springtime thaw-weakening period than the simple FSV classification system. More intensive field studies are necessary to provide a basis for pavement structural design based on frost-heave testing. A maximum heave-rate criterion of 2 mm/day under the test procedure described here is currently being used by the Alaska Department of Transportation and Public Facilities for evaluation of stabilizing agents intended to reduce the frost susceptibility of soils in pavement layers.

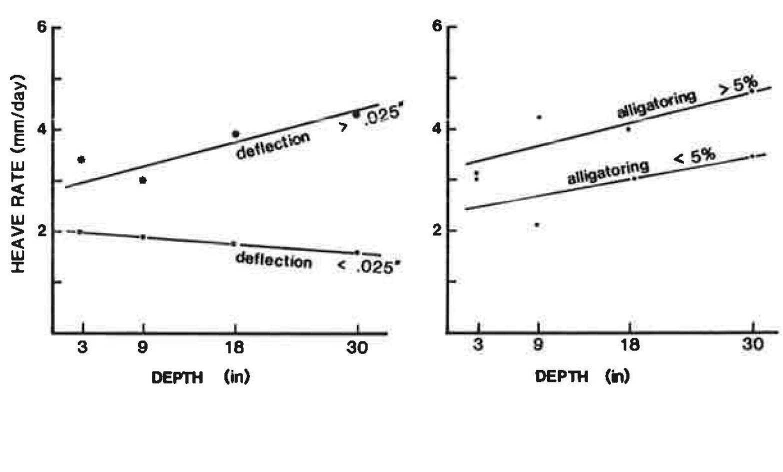
Performance Versus R-Value Test Data

Attempts were made to relate observed pavement performance to soil-sample Hveem stabilometer R-values, based on data collected from 25 study-section locations that covered a wide range of distress severity.

Figure 17 shows the average R-values at selected depths associated with high and low deflection and alligator cracking levels, based on a division of groups at 0.400 in of deflection and 5 percent alligator cracking. A decrease in observed R-values with depth is obvious for each performance factor. Interestingly, although pavements in the lower deflection group had higher R-values at depths greater than 9 in, the plot of alligator cracking versus R-values indicates that poorer performance was related to the highest average R-values at each depth location noted. Neither could any systematic relation be found between rut depths and R-values at any depth.

An analytic approach was then taken that more closely paralleled the actual design use of the R-value test. Each of the 25 sections was examined to see whether the as-built pavement structures satisfied the overall design overlay requirement in accordance with California's overlay design chart (9). Overlay deficits in terms of additional asphalt pavement thickness ranged from 0 to 3.5 in and are plotted as group average data points in Figure 18. Equivalent additional gravel-base thickness requirements would be approximately twice these values, based on a gravel equivalent factor of 2.0

Figure 16. Heave rates of samples from various depths versus deflection and alligator cracking.



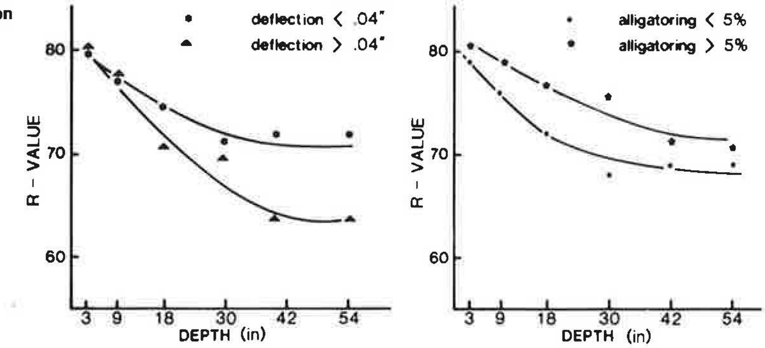


Figure 17. R-values of samples from various depths versus deflection and alligator cracking.

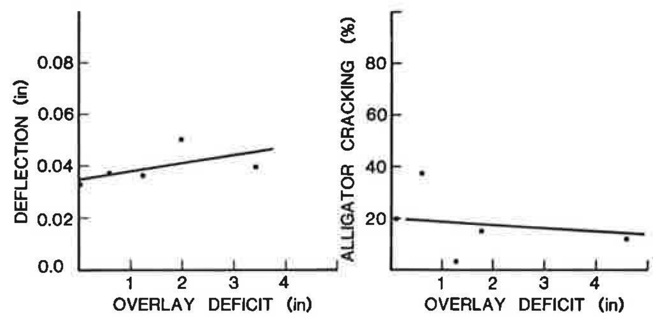


Figure 18. Overlay deficiencies versus deflection and rut depth based on R-value design method.

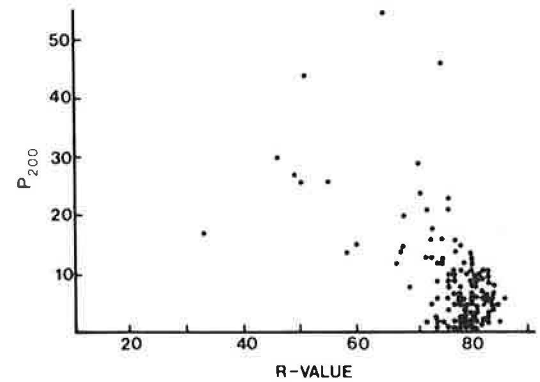


Figure 19. R-values versus P_{200} contents for all soils tested in 1976.

for asphalt concrete pavements. No significant correlations were found between overlay deficits and any of the three principal performance variables--namely, rut depth, alligator cracking, and deflection.

The absence of any relations between R-value test results and pavement performance in Alaska is considered to be the effect of overriding frost-susceptibility considerations that control the field performance of pavements. For Alaska soils, progressively increasing the fines content of a base or subbase aggregate will not result in any major decline in R-values until the fines content exceeds 12-14 percent, as shown in Figure 19. However, at fines contents greater than 6-10 percent, base and subbase aggregate may become extremely susceptible to frost heave and thaw weakening while retaining high R-values.

SUMMARY

Analyses were performed to determine the critical factors in good pavement performance based on 120 roadway sections on which the pavement structural layers were measured, sampled, tested, and classified for frost susceptibility. This study was confined to flexible pavements between 1 and 4 in in thickness with unbound granular bases. The soils encountered were primarily gravels and silts; uniform sands and clays were conspicuously absent. It must be realized that this was not a controlled experiment; i.e., no one variable could be altered while all others were held constant. For this reason, the effects of a single factor could not be absolutely determined under specific conditions. The primary factors omitted from this study were the relative densities and elastic properties of the pavement structural layers and quantifications of the groundwater available to support frost action. In spite of multiple regression analyses of 175 material, dimensional, and environmental factors pertinent to the performance of the 120 study sections, no empirical equation could be derived to predict the relative performance levels of different pavement structures with reasonable accuracy.

CONCLUSIONS

1. From analyses of the material and environmental factors commonly related to good performance of flexible pavement structures in Alaska, the most important factors in good performance are low percentages of -0.075-mm and -0.02-mm particles in the base and subbase layers.

2. Best performance is related to maximum base and subbase -0.075-mm contents of 6 percent and to maximum -0.02-mm contents of 3 percent. Poor performance predominates when base and subbase fines reach 11 percent, and when -0.02-mm contents reach 7 percent in these layers.

3. Soil frost-susceptibility ratings of the pavement structural layers by the Corps of Engineers FSV classification system are only moderately related to pavement performance. Differences in performance between the F-1 and F-2 classes were found to be nearly insignificant, and differences between the F-3 and F-4 classes were also small. Percentages of soil fines should therefore be considered for direct use as a design parameter for frost areas instead of the FSV classification system.

4. The Hveem stabilometer R-value test is not a useful indicator of relative pavement structural performance for frost areas because it does not indicate differences between soils with low and high frost susceptibility. Modifications to the moisture-conditioning phase of this test, such as the addition of a frost-heave stage, should be considered to make the R-value test more applicable to the structural design of pavements in frost areas.

5. Results of laboratory heave tests indicated that the best pavement performance is related to

heave rates in base and subbase layers of less then 3 mm/day.

6. No significant differences in performance relations were noted between the wet coastal areas and the dryer interior areas of Alaska.

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REFERENCES

1. Guide for Flexible Pavement Design and Evaluation. Alaska Department of Transportation and Public Facilities, Fairbanks, 1978.
2. E.J. Yoder and J.W. Witczak. Principles of Pavement Design, 2nd ed. Wiley, New York, 1975.
3. Asphalt Overlays and Pavement Rehabilitation, 1st ed. Asphalt Institute, College Park, MD, Manual Series 17, 1969, pp. 109-117.
4. A Guide to the Structural Design of Flexible and Rigid Pavements in Canada. Canadian Good Roads Assn., Ottawa, Ontario, 1965.
5. C.W. Hartman and P.R. Johnson. Environmental Atlas of Alaska, 2nd ed. Institute of Water Resources, Univ of Alaska, Fairbanks, 1978.
6. C.W. Kaplar. Experiments to Simplify Frost Susceptibility Testing of Soils. U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH, Tech. Rept. 223, 1971.
7. C.W. Kaplar. Freezing Test for Evaluating Relative Frost Susceptibility of Various Soils. U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH, Tech. Rept. 250, 1974.
8. A. Casagrande. Discussion of Frost Heaving. HRB, Proc., Vol. 2, Part 1, 1932, pp. 168-172.
9. League of California Cities. Structural Section Design Guide for California Cities and Counties. DPW Reproductions, Sacramento, CA, Rept. 13051 3-73, 1973.

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Simulating Frost Action by Using an Instrumented Soil Column

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The use of an instrumented soil column in tests to develop a mathematical model of the frost-heave process is described. Tensiometers, heat-flow meters, thermocouples, and electrical resistivity gages were installed throughout a soil column filled with Fairbanks silt, Chena Hot Springs silt, or West Lebanon gravel. The column was 100 cm long and about 14 cm in diameter. An open system was used and absorption was monitored during the freezing process. Three freezing tests were conducted on Fairbanks silt, two with a surcharge of 3.45 kPa and one with 34.5 kPa. The water level was held at the 45-cm depth. Eleven tests were conducted using Chena Hot Springs silt. The water level was set at 15-, 50-, or 100-cm depths, and the surcharge was 3.45 or 34.5 kPa. Tests were conducted by using a constant rate of frost penetration, a constant heat-flow rate, or three sequentially lower temperature step changes at the soil surface. Two tests were conducted with West Lebanon gravel. Both included three step changes in the surface temperature and were surcharged with 3.45 kPa. The water levels were at the 15- and 100-cm depths. The soil column has provided critical data for verification of a one-dimensional mathematical model for estimating frost heave. As more soils are tested, this equipment will assist in improving and developing algorithms for the mathematical model and in identifying the most critical parameters that affect frost heave in a given soil—e.g., surcharge, free water level, and hydraulic conductivity. A procedure is also presented for determining the saturated and unsaturated hydraulic conductivity and moisture-retention characteristics of a soil.

In July 1975, the Federal Highway Administration (FHWA), the Federal Aviation Administration (FAA), and the U.S. Army Corps of Engineers initiated a cooperative project to develop a mathematical model of the frost-heaving process. The one-dimensional finite-element computer program resulting from the study was documented (1,2, and Guymon and others elsewhere in this Record).

The results reported here were obtained from an instrumented soil column that was originally constructed to provide data for developing an algorithm describing the effects of overburden (or surcharge) on soil moisture stress. The column was designed so that the water table could also be varied. We have also used data from the soil column to aid in verifying the mathematical model of frost heave (2 and Guymon and others elsewhere in this Record) and have developed a more refined design that will be used with a dual gamma system. With the dual gamma system, we will have the capability of monitoring changes in moisture content and density nondestructively. The dual gamma system, in conjunction with the other instrumentation in the soil column, will permit developing the moisture-characteristic curve and the relation between moisture content (or matrix suction) and hydraulic conductivity for the soil within the column. The soil column will also be used for other portions of the cooperative study—e.g., to assist in developing the thaw-weakening algorithm and in applying the mathematical model to layered pavement systems.

TEST APPARATUS

The soil was molded within a 100-cm-long circular cylinder that had a diameter of about 14 cm. The inside of the upper 15 cm of the cylinder tapered slightly and was lined with Teflon tape to minimize sidewall resistance to heaving. The top portion of the cylinder was detachable from the lower portion (see Figure 1).

Thermocouples were inserted through the cylinder walls and into the soil at intervals of 1 cm in the upper portion and intervals of 2.5-10 cm in the