ACKNOWLEDGMENT

The financial support provided for this study by the Short Term Applied Research Program of the state of Idaho, the Phillips Petroleum Corporation, and the Department of Civil Engineering, University of Idaho, is gratefully acknowledged. The technical assistance and encouragement provided by the Division of Highways, Idaho Transportation Department, also contributed to the successful completion of this study.

REFERENCES

1. Roadway Design in Seasonal Frost Areas. NCHRP Synthesis of Highway Practice 26, 1974, 104 pp.
2. F.H. Scrivner, R. Peohl, W.M. Moore, and M.B. Phillips. Detecting Seasonal Changes in Load-Carrying Capabilities of Flexible Pavements. NCHRP Report 76, 1969, 37 pp.
3. Structural Design of Asphalt Concrete Pavement Systems. HRB, Special Rept 126, 1971, 207 pp.
4. Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Cracking. HRB, Special Rept 140, 1973, 201 pp.
5. C.L. Monismith, J.A. Epps, D.A. Kaisianchuk, and D.B. McLean. Asphalt Mixture Behavior in Repeated Flexure. Institute of Transportation and Traffic Engineering, University of California, Berkeley, Report TE-70-5, 1971.
6. A.T. Bergen and C.L. Monismith. Characteristics of Subgrade Soils in Cold Regions for Pavement Design Purposes. HRB, Highway Research Record 431, 1973, pp. 25-37.
7. Test Procedures for Characterizing Dynamic Stress-Strain Properties of Pavement Materials. TRB, Special Rept 162, 1975, 38 pp.
8. M.A. Miner. Cumulative Damage in Fatigue. Journal of Applied Mechanics, Vol. 12, No. 3, 1945, pp. 159-164.
9. R.P. Lottman. Practical Laboratory Measurement and Application of Stiffness or Resilient Properties of Soils and Granular Base Materials for Idaho Flexible Pavement Design Procedures. Department of Civil Engineering, University of Idaho, Moscow, Final Rept, 1976, 102 pp.
10. R.J. Schmidt. A Practical Method for Measuring the Resilient Modulus of Asphalt Treated Mixes. HRB, Highway Research Record 404, 1972, pp. 22-31.
11. J.H. Hardcastle and R.P. Lottman. A Rational Method for Establishing Spring-Thaw Load Limits on Idaho Highways. Idaho Short Term Applied Research Program, University of Idaho, Moscow, Final Rept, 1978, 90 pp.
12. J. Michelow, H. Warren, and W.L. Dieckmann. Numerical Computation of Stresses and Strains in a Multiple-Layered Asphalt Pavement System. California Research Corp, Richmond, CA, 1963.

Prediction of Roadway Strength From Soil Properties

DAVID C. ESCH AND ROBERT L. McHATTIE

An intensive performance study of 120 Alaskan asphalt-paved roadway sections was completed in 1980. Roadway-bearing strengths were measured weekly on each section during the period of thaw weakening by means of a Benkelman beam-type test procedure. Soil properties and layer thicknesses were evaluated by use of test pit sampling. This study, which included sections from the different climatic regions of Alaska, has indicated certain soil particle sizes to be the most critical factor in crack-free performance and resistance to spring thaw weakening. The data acquired in the performance study have provided a basis for the development of a new method of pavement design. This method predicts the maximum seasonal or design deflection level from the percentages of particles smaller than 0.075 mm (No. 200 sieve) in the different granular soil layers beneath the asphalt pavement. Required pavement thicknesses for the predicted traffic and design deflection levels are then determined by a previously developed overlay design procedure. This method is demonstrated to be more reliable than previous designs based on the frost susceptibility classification of the pavement layers.

In a field performance study of 120 existing Alaskan flexible highway pavement sections completed in 1980, more than 200 variables were analyzed to determine their effects on pavement performance (1). Each pavement was rated in terms of alligator fatigue cracking, wheelpath rut depth, thermal cracking, ride quality, and maximum springtime Benkelman beam deflection level. Construction materials were sampled from test pits at two locations within each highway section. Soils were sampled to a depth of 4.5 ft. Laboratory testing included soil gradations, percent fracture, and in situ moisture content. Laboratory frost-heave rate and Hveem R-value tests were also performed for many of the sections. Additional variables that described the environment in terms of climate, pavement age, and accumulated traffic loadings were introduced into the data base.

The factors found to relate most strongly to the springtime load-bearing capacity and to the amount of fatigue (or alligator) cracking were the percentages of particle sizes smaller than 0.075 mm in the base and subbase layers (2). These layers extend from a depth of 0 to 2 ft beneath the pavement (see Figure 1). Particles passing a No. 200 mesh sieve, i.e., particles smaller than 0.075 mm, and fractions smaller than 0.02 mm are generally used as indicators of frost susceptibility. Other ways of expressing these particle sizes in this paper are P_{200} sieve or fines, < 0.075 mm, and < 0.02 mm.

It was also found that peak spring Benkelman rebound deflection levels has a strong positive correlation with the observed severity of alligator cracking (Figure 2). Relationships between alligator cracking and pavement age indicate that this type of cracking propagates rapidly once it exceeds 5 percent of the combined wheelpath length of the test section. From these conclusions it was apparent that if an equation could be developed for maximum seasonal deflection levels based on soil and climatic factors, pavement cracking and fatigue life could be predicted.

Figure 1. Average maximum deflections for different P_{200} contents in base course.

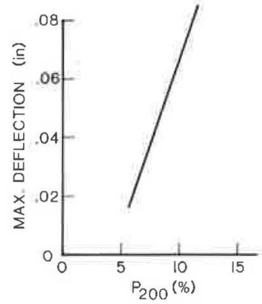


Figure 2. Relationship between maximum deflection measurements on pavements and alligator cracking.

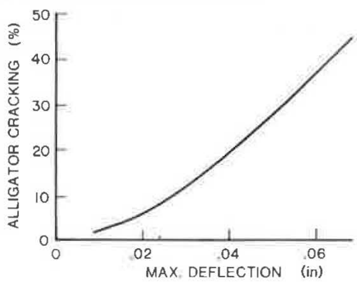
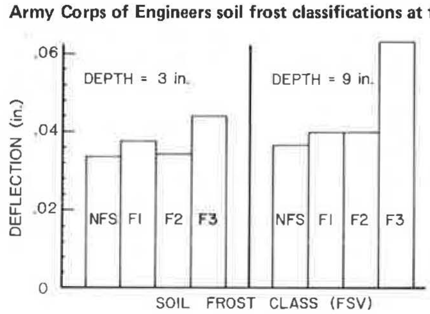


Figure 3. Average deflections for pavement structures with different U.S. Army Corps of Engineers soil frost classifications at two selected depths.



ANALYSIS OF PREVIOUS DESIGN METHODS

Subsequent analysis of the data base obtained in this study has demonstrated that the use of the U.S. Army Corps of Engineers Frost Classification System as developed originally by Cassagrande (3) did not provide an accurate basis for predicting the relative performance of the different sands and gravels used in typical roadway pavement structures (Figure 3).

The Hveem R-value test and design method, in which soil properties are determined in a soaked condition with a modified triaxial-type test loading, also proved of no value in predicting roadway fatigue performance in seasonal frost areas. This is probably because the method has no way of allowing for the effects of frost action, which may greatly reduce the strength levels of unbound gravels during thaw.

By using the results of the laboratory frost-heave-rate test as a design method basis, test procedures developed by Kaplar (4) were investigated and some potential was indicated. However, the available test data from a small number of pavement structures were not adequate to provide a statistical basis for design. The only conclusion drawn was that test heave rates in excess of 0.0784 to 1.181 in. per day, at a freezing rate of roughly 0.75 in. per day, were related to inferior field performance (3).

DATA ANALYSIS BACKGROUND

By using multiple regression analysis, attempts were made at predicting the spring thaw-weakened deflection levels of the 120 pavement sections studied. The most important factors were found to be the percentages of < 0.075-mm size particles (P_{200} sieve or fines) in the upper unstabilized pavement system layers. It is significant that these particle sizes are also commonly considered to be the primary factors controlling the laboratory frost-heave rates and resultant thaw weakening of sands and gravels. Because of the greater ease of testing, fines content (< 0.075 mm) specifications are most often used for field control of the frost susceptibility of unbound granular pavement structural layers.

Climatic factors varied widely among the different sections included in this study. However, even though precipitation ranging from 10 to 100 in. per year and freezing indexes from 800 to 6,000 degree-days (Fahrenheit) per year, the influence of climatic differences on the extent of thaw weakening measured on the various sections did not appear to be statistically significant. This may have happened because all sections studied undergo annual freeze-thaw cycling and sufficient water always appears to be available to cause frost heave and thaw weakening. For these reasons, deflection correlation equations were developed which had no climatic or regional factors.

All references to maximum deflection levels in this paper refer to the highest probable Benkelman-beam-rebound deflection level as measured and calculated in accordance with the Asphalt Institute test method (5,6). Deflection measurements were made weekly for 1 month during the spring thawing season on each section studied. Data from each measurement date were used to calculate the maximum level for that date as the average deflection level plus two standard deviations. The highest of these maximum deflection levels obtained during the repeated weekly surveys was used subsequently to represent the design or maximum deflection level (D_{max}) for that roadway section.

In the original study analyses, the P_{200} content that related to increasing maximum Benkelman deflection and cracking levels was identified at depths from 0 to 4.5 ft beneath the asphalt pavement layer, as shown in Figures 4 and 5. It is apparent from inspection of these figures that increases in P_{200} content above those shown for the lowest deflection or zero cracking levels will result in increased thaw weakening and greater cracking.

Inspection of all soils gradation data obtained in this study, however, indicated probable significant direct relationships between the P_{200} content at the top and bottom of the pavement structures examined. This can be explained geologically because, in regions having clean gravels commonly available, the entire roadway embankment will most probably be constructed of relatively clean gravels from top to bottom. As a result, simple inspection of existing pavement structures did not provide any information as to the maximum depth at which the designer must be concerned about the fines contents and frost susceptibility of soil layers.

DEPTH OF WHEEL LOAD INFLUENCE

To resolve the question of the relative significance of excessive fines content versus depth, it was necessary to make some assumptions on the stress-strain properties of the different layers of pavement structure. For analysis purposes, a standard dual wheel loading was converted to an equivalently loaded square 12.5 in. on a side at the bottom of the 2-in.

Figure 4. Dual wheel-load strain reduction versus depth.

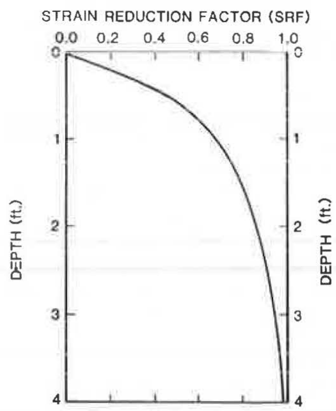
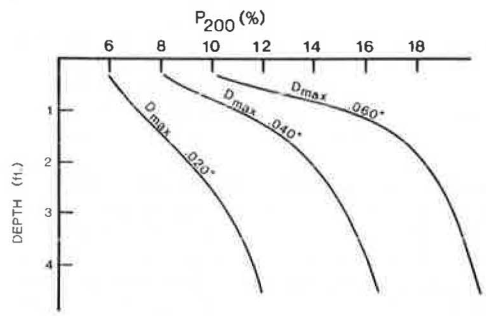


Figure 5. Average fines content at different depths for roadways having different maximum deflection levels.



pavement thickness typical of the sections studied. The underlying pavement structure was assumed to be represented by a homogenous elastic solid having a Poisson ratio of 0.25. The vertical stresses and relative strains at any depth beneath the surface can than be determined from a typical Boussinesq stress chart (7). Then the total surface deflection is determined by summing the net strains in each layer. To simplify this analysis method, a plot of wheel-load strain reduction versus depth was prepared as shown in Figure 4. This curve was obtained by integrating the Boussinesq stress versus depth curve, to include the effects of stress and strain and layer thickness on surface deflection. This plot can then be used to establish the relative contributions of layers at different depths to the surface deflection.

From Figure 5 it is apparent that in a homogenous pavement structure, as might occur when all layers were equally resistant to frost action and thaw weakening, the deflection related to wheel load is reduced by 50 percent at a depth of about 9 in. and by 95 percent at a depth of 40 in. This indicates that soil properties in the base and subbase layers directly underlying the pavement are most critical and that properties below the 3.5-ft depth are of minor concern.

DEVELOPMENT OF EXCESS FINES CONCEPT

From Figures 5 and 6, it is apparent that increasing P_{200} content will result in increased pavement deflections and increased fatigue or alligator cracking. These plots, however, represent the average fines content of roadway sections grouped by similar levels of deflection and cracking and do not provide a design basis as discussed above. The design concept derived from these figures was that at any given

Figure 6. Average fines content at different depths for roadways having different percentages of cracking.

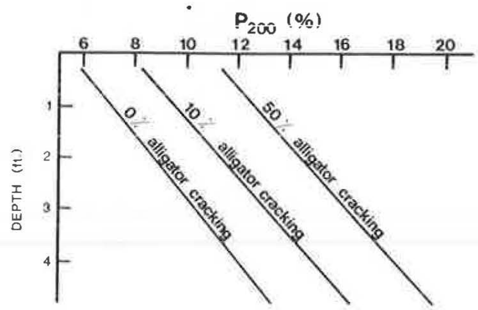


Table 1. Pavement layers analyzed, stress reduction factors, and critical fines content.

Layer No.	Depth Interval (ft)	Thickness (ft)	Strain Reduction (%)	Critical Fines (% <0.075 mm)
1	0.0-0.50	0.5	45	6
2	0.5-1.0	0.5	23	6
3	1.0-2.0	1.0	19	9
4	2.0-3.0	1.0	7	20
5	3.0-4.0	1.0	3	80
6	4.0-4.5	0.5	1	100

depth there is a critical fines content (P_{cr}) which, if exceeded, will result in some amount of frost action and thaw weakening detectable through increased springtime-roadway deflections. The percentage by which the fines in any pavement layer exceeds this critical fines value may then be termed the excess fines content for that layer.

To determine the actual critical fines content at each depth, the 120 pavement structures on which this analysis was based were broken into six equivalent layers, with thicknesses of 0.5 ft for the two uppermost layers (commonly termed the base and subbase courses) and thicknesses of 1 ft below the subbase layer. The percentages of a typical wheel-imposed stress carried by each of these layers was then determined from the difference in stress reduction between the top and bottom of that layer (Figure 4), as shown in Table 1. The actual critical fines content at the center of each layer was found by inspection and analysis of the entire data base, as explained later. It was discovered in this analysis that fines content (and other soil properties) at depths greater than 3.5 ft had no significant effect on measured, maximum-pavement deflections.

FINAL DESIGN BASIS

The basis used for empirically predicting maximum deflections from the excess fines in each pavement layer involves using two factors to characterize each layer. The first is the percentage by which the fines (P_{200}) content of each layer exceeds, or is expected to exceed, the critical fines content (P_{cr}) for that layer. The second factor is the depth-related stress and strain reduction which occurs between the top and bottom of each layer and the resulting contribution of each layer to the total surface deflection (Figure 4).

Pavement structures having no excess fines in any layer are considered not frost susceptible and will have some relatively low maximum deflection levels. Each layer that exceeds its appropriate critical fines content will contribute to increasing the maximum deflection level. The summed effects of all

Figure 7. Critical (maximum) fines contents at various depths to assure the lowest maximum pavement deflections during seasonal thawing.

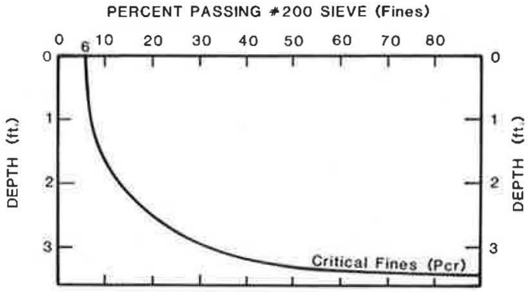
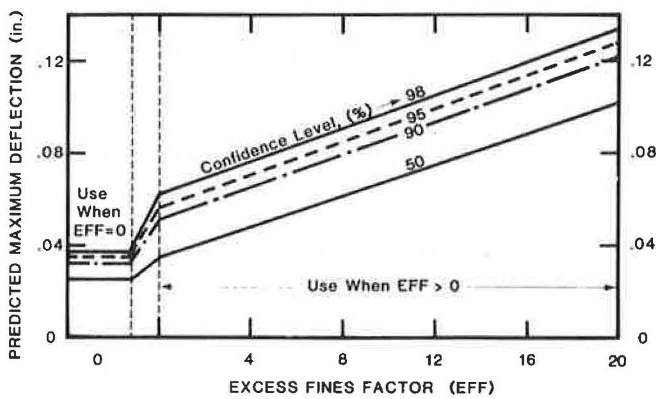


Figure 8. Maximum deflections for different excess fines factors from Equation 1.



layers with excess fines is then used to determine a single factor, termed the excess fines factor (EFF), which represents that pavement structure for use in a correlation equation to determine the probable maximum deflection level. This design concept can be stated by Equations 1 and 2.

$$EFF = (n/j=1)\ SRF_j (P_{200} - P_{Cr})_j^{0.8} \quad (1)$$

where

SRF_j = change in strain reduction factor for each layer (from Figure 4),
j = layer number,
n = total number of layers,
P_{200} = average percent of <0.075 mm particles for each layer,
P_{Cr} = critical percent of <0.075 mm particles for each layer (from Figure 7), and
0.8 = exponent selected for best statistical fit.

$$D_{max} = D_0 + K(EFF) \quad (2)$$

where

D_0 = a constant which depends on the required confidence level and
K = regression coefficient.

To establish the highest level of fines at each depth for which no excessive maximum deflections occurred, a series of alternative positions of the critical fines versus depth P_{Cr} line were used to calculate alternative EFF factors for all 120 road sections. These factors were then plotted against the maximum deflections for each section. The final position of the P_{Cr} versus depth line was established by inspection of these scatter plots and is shown in Figure 7. For all pavement structures that had a fines content at or below the critical value at all depths, the maximum deflection level averaged 0.025 in. with a standard deviation of 0.0056 in. and never exceeded 0.036 in.

For those sections that exceeded the critical contents of Figure 7 at any depth, the maximum deflection was predicted by the regression equation shown by Equation 3, which had a standard error of estimate of 0.015 in. When using this equation, the designer must be concerned with the maximum probable deflection level rather than the average (50 percent confidence) deflection. The 95 percent confidence level is suggested as a suitable basis for design.

$$DD_{max} = D_o + 0.0035\ (EFF) \quad (3)$$

where

DD_{max} = design maximum deflection,
EFF = excess fines factor from Equation 1, and
D_o = constant selected from the listing below depending on required level of confidence that actual D_{max} will be less than that predicted by Equation 3.

Confidence Level (%)	Constant, D_o (in.)
98	0.061
95	0.056
90	0.050
50	0.031

The result of this design approach is indicated by Figure 8. Note that predicted deflections assume a 2-in. asphalt pavement thickness. After determination of the probable maximum deflection level for a given structure, the pavement designer can then provide an increased pavement thickness, if needed, for the anticipated traffic levels. A suitable design chart for this purpose, from Asphalt Institute Manual MS-17 (5), is shown in Figure 9. Details and examples of this complete design approach are contained in the Guide for Flexible Pavement Design and Evaluation (8).

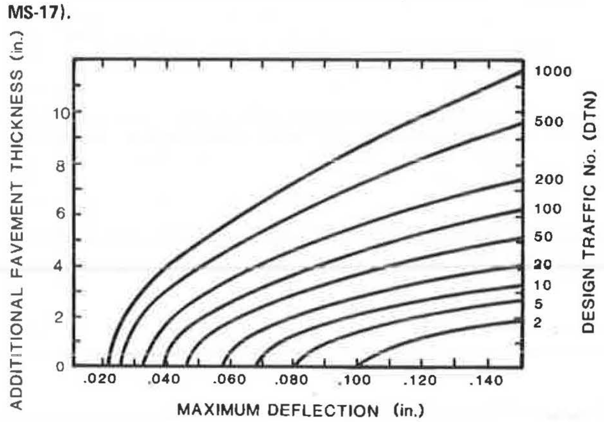
LIMITATIONS OF METHOD

Because this design approach was developed empirically based on existing Alaskan pavement structures, it should not be applied to analysis of conditions not represented by those sections observed. The deflection prediction method is considered applicable primarily to asphalt pavements ranging from 2 to 6 in. in thickness in areas where annual pavement structure freezing occurs and placed on unstabilized gravel base and subbase layers that contain no more than 16 percent of particles less than 0.075 mm to a depth of 1 ft beneath the pavement. The soil fines encountered in the pavement structures comprising the data base were essentially all nonplastic silts, which are considered most susceptible to frost action. Clayey gravels or clay subgrade soils were almost never encountered and therefore are not represented in the data base. The ratio of the < 0.02 mm to < 0.075 mm particle size content averaged approximately 0.6 percent for all samples tested in this study.

SUMMARY

An empirical method was developed for using the minus No. 200 particle-size (fines) content of soils in the different layers of unstabilized pavement structures to predict the maximum thaw weakened deflection levels. Equations were prepared for different confidence levels. This method provided improved per-

Figure 9. Design chart for required increases in pavement thickness based on maximum deflection levels and traffic factors (Asphalt Institute Manual MS-17).



formance predictions as compared to the U.S. Army Corps of Engineers Reduced Subgrade Strength design method and frost susceptibility soil classifications, and it is also superior to the use of Hveem R-value test data as a basis for predicting performance under conditions of seasonal freezing and thawing. The Alaska Department of Transportation and Public Facilities has recently adopted this approach for pavement design purposes, and a design guide has been prepared. This new design method was based on soils and performance data from 120 roadway sections. The maximum or critical fines content at various depths that can be allowed without causing significant thaw weakening of the pavement structure were established for all roadway study sections. Soil layers that have a fines content greater than this critical fines relationship contribute to thaw weakening and are used to predict the extent of weakening and the pavement thickness required. This approach has been termed the excess fines design method.

REFERENCES

1. R.L. McHattie, B. Connor, and D. Esch. Pavement Structure Evaluation of Alaskan Highways. Alaska Department of Transportation and Public Facilities, Res. Rept. FHWA-AK-RD-80-1, 1980.
2. D.C. Esch, R.L. McHattie, and B. Connor. Frost Susceptibility Ratings and Pavement Structural Performance. TRB, Transportation Research Record 809, 1981, pp. 27-34.
3. A. Casagrande. Discussion of Frost Heaving. HRB, Proc., Vol. 2, Part 1, 1932, pp. 168-172.
4. C.W. Kaplar. Experiments to Simplify Frost Susceptibility Testing of Soils. U.S. Army Cold Regions Research and Engineering Labora- tory, Hanover, NH, Tech. Rept. 250, 1974.
5. Asphalt Overlays and Pavement Rehabilitation. Asphalt Institute, Manual Series 17, College Park, MD, Nov. 1969.
6. R.I. Kingham. Development of Asphalt Institute's Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements. Asphalt Institute, Res. Rept. 69-3, College Park, MD, 1969.
7. E.S. Barber. Effect of Soil Blankets and Preloading on Settlements. Public Roads, Vol. 33, No. 12, 1966.
8. Guide for Flexible Pavement Design and Evaluation. Alaska Department of Transportation and Public Facilities, Division of Standards and Technical Services Materials Section, Anchorage, AK, Aug. 1982.

The Val Gagne Pavement Insulation Experiment

T.M. LOUIE, W.A. PHANG, AND R.A. CHISHOLM

Since the first successful treatment of a frost heave problem in 1966 using Styrofoam HI brand (trademark of Dow Chemical, now called Dow Chemical Canada, Inc.) extruded polystyrene foam insulation, Ontario has used substantial quantities of insulation. However, knowledge of the performance of the insulation has been limited to sparsely documented observations of some sites where different types and amounts of insulation were used. In 1972 a joint experiment was launched by the Ministry and the Dow Chemical Inc. to construct, instrument, and observe the performance of pavement insulation at Val Gagne in Ontario. Three thicknesses of insulation and four different taper designs were used in the experiment. Winter temperature profiles, frost penetration depths, and frost heave measurements were observed during the winters of 1973-1977. The results of these observations were used to verify a two-dimensional finite element heat flow computer program intended for use in the design of insulation for controlling frost penetration. Although the program accurately predicts ground temperature for uninsulated situations and thinly insulated sections, changes in the program are needed to correct inaccuracies in predictions of ground temperatures when thick insulation is used. The results were also used to develop a set of frost penetration prediction curves for various thicknesses of insulation and for locations with different degree-days of freezing temperatures. These curves may be used to select the appropriate insulation thickness for any acceptable depth of frost penetration. Styrofoam insulation recovered from the test site after 5 years shows virtually no structural changes or changes in thermal properties.

Differential frost heaving poses serious potential safety hazards and has perennial aftereffects on highways in Canada and the northern areas of the United States. Before 1966 all treatments of frost-heaves in Ontario involved removal of the frost-susceptible soil to great depths and/or ditching and drainage to remove entrapped water and to lower the water table. These treatments were not always successful.

Research into the use of Styrofoam for highway insulation began in the United States in the early 1960s, and by 1965 several full-scale experimental projects had been constructed (1-4). These early research sites were instrumented with thermocouples and frost depth indicators to measure the insulating properties of various thicknesses of Styrofoam in-