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Frost Protection and Insulation for Transportation Facilities

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Contents

Foreword

A major problem for transportation facilities in permafrost areas is the action of frost. The prevention of penetration of frost into the subgrade of pavements is therefore an important consideration for pavement engineers in cold regions. In recent years, in many areas of the world, pavements have been protected using different types of insulating materials. Presented in the five papers included in this Record are the results of studies conducted on insulation for frost protection in transportation facilities.

Rune Gandahl reports on a procedure he developed in Sweden to calculate frost heave when the frost penetrates into the frost-susceptible subgrade soils. This method facilitates optimization of the frost-protective base construction with respect to local climatic, geological, and hydrological conditions.

For over 20 years expanded polystyrene has been used to insulate the subgrade at many frost-heave sites in Ontario, Canada. J. B. MacMaster and G. A. Wrong report on their evaluation of the effectiveness of polystyrene for controlling frost heave. They also discuss the establishment of specifications and standards that are in use to control frost heaving in Ontario.

David Esch reports on performance evaluation of extruded polystyrene, foamed-in-place polyurethane, and molded polystyrene beadboard insulation installed beneath several roads and airfields in Alaska. The findings indicate that the extruded polystyrene insulation has superior performance and longevity. He also compares the measured late summer permafrost thaw depths for insulated airfields and the values obtained using the Modified Berggren calculation method.

Erik Sandegren reports on the history of insulation used against frost in railways in Sweden. He also describes the properties of extruded cellular plastic of polystyrene and the application of this material to the track.

R. H. Jones reports on the evaluation of the British method of testing materials for frost susceptibility. He found particle size distribution and saturation moisture content to be poor and suction characteristics to be good indicators of the frost susceptibility of aggregates.

Polystyrene Foam as a Frost Protection Measure on National Roads in Sweden

RUNE GANDAHL

Polystyrene foam boards have been used as a frost protection measure on roads in Sweden for about 20 years. Different qualities have been tested regarding their long-term function as an insulation. Properties such as bacterial, chemical, and mechanical durability in roads and preserved low-heat conduction have been continuously followed. In the specifications of the National Road Administration in 1976, the extruded polystyrene skinboard, type HI 50 from the Dow Chemical Company, was recommended for use in an insulated road base. For skid resistance, the depth of insulation was changed from 50 cm, according to the 1976 specifications, to 40 cm below road surface in the specifications of 1984. This change was based on experience from the Swedish Road and Traffic Research Institute test field. It was observed that a conventional crushed rock roadbase showed the same icing risk potential as a base with 40 cm of sandy gravel above the insulation board. Recommended insulation thicknesses are related to average freezing index at the site. A new procedure is being developed making it possible to calculate the frost heave when the frost penetrates below the base into underlying frost-susceptible soils. The procedure is based on determination of the frost heave properties of soils by a direct freezing test. This make it possible to optimize the frost-protective base construction in relation to local climatic, geological, and hydrological conditions at the site.

Extruded polystyrene skinboard and molded bead polystyrene board have been used as a frost damage protection measure for about 20 yr on a number of national and other roads in Sweden. In the Swedish Road Administration specifications of 1976 and 1984, the insulated road base consisted of three main layers: the overlying bearing course upon the plastic foam layer; the plastic foam layer itself; and, as a bottom layer, non-frost-susceptible soil material (e.g., sand or old roadbase material). Figure 1 illustrates both this construction and bases with gravel and sand, tree bark, lightweight aggregates, and a conventional base. All of these bases were frost protected to the same degree. The layers above the plastic foam had a thickness of at least 40 cm. The thickness of the plastic foam layer depends on the mean freezing index at the site and also on the thickness of the underlying non-frost-susceptible layers. Table 1 shows the design table for insulation with polystyrene foam used on Swedish roads *(1)*.

The type of plastic foam used in insulating Swedish roads is predominantly made up of different qualities of extruded polystyrene skinboard foam from the Dow Chemical Company. The recommended quality is Styrofoam HI 50. The investigation of this product and other varieties of polystyrene foam started in 1966 and has been continued for many years. The results have led to the conclusion that polystyrene foam can also function in the long run as an efficient measure against the detrimental effects of frost.

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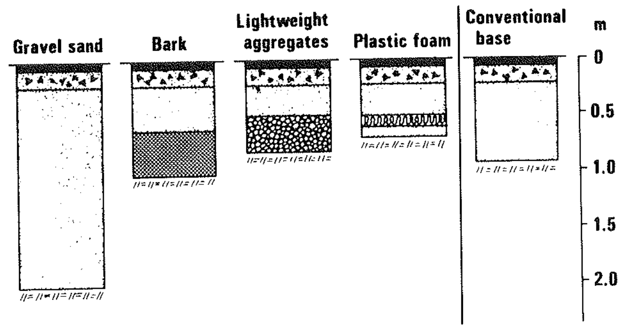

FIGURE 1 Thickness of road bases for same frost protection degree and different frost protection methods.

INVESTIGATIONS OF THERMAL BEHAVIOR ON TEST ROADS

When the Swedish Road and Traffic Research Institute (VTI) was asked to test polystyrene foam built into a roadbase in 1965, the frost-insulation properties of this type of foam were practically unknown. The tests therefore had to be both intensive and extensive. In Sweden, frost problems affect every part of the country. The climatic conditions can simply be described by the distribution of the freezing index, see Figure 2. The increasing values of the freezing indices toward northern regions indicate a greater need for frost protection in these parts of the country. Compare the frost heave shown in the three columns in the table in Figure 2.

The first test road was constructed in 1966, and was of an introductory type. The next year, 1967, the main test roads were constructed. These test roads, Edsvalla 1967 A and B in the western part of Sweden, contained sections with two types of polystyrene foam and different designs of the base. For comparison, a number of test sections with lightweight aggregates and tree bark were included. The total number of sections was 44. Some sections are to be found in Figure 3. Among other test roads, one in central Sweden is of special interest: Lasele 1972. The test road is situated in a part of Sweden where frost protection measures must always be considered. It contains sections with two different types of plastic foam: extruded polystyrene skinboard and molded bead polystyrene.

TABLE 1 DESIGN TABLE USED BY THE NATIONAL SWEDISH ROAD ADMINISTRATION FOR ROAD BASES INSULATED WITH PLASTIC FOAM *(1)*

Average Freezing Index (°Cd)	Insulating Bed[a] (mm) by Foam Layer Thickness (m)						
	100	200	300	400	500	600	700
<500	35	30	25	20			
500–600	40	35	30	25	20		
600–700	50	40	35	30	25	20	
700–800	60	50	40	35	30	25	20
800–900	70	60	50	40	35	30	25
900–1000	80	70	60	50	40	35	30
1000–1100	90	80	70	60	50	40	35
1100<	90	90	80	70	60	50	40

[a]Non-frost-susceptible material.

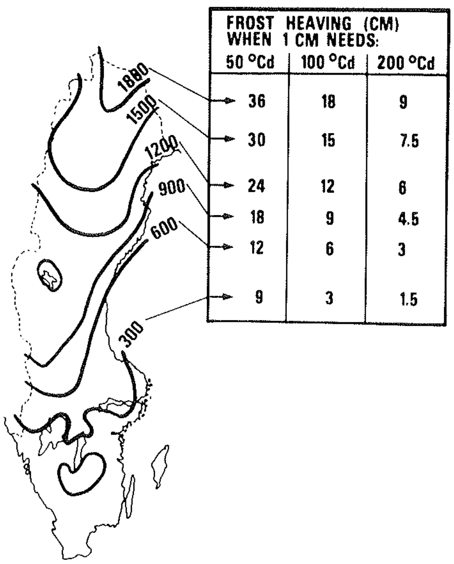

FIGURE 2 Average freezing index (°Cd) and corresponding frost heave for the frost heave quotients 50, 100, and 200 °Cd/cm.

At all the test roads mentioned, the water takeup of the plastic foam has been determined. Several specimens of the foam from the test roads have also been tested for heat insulation properties. Thermal conductivity has been determined on samples directly after being taken up from the road. The water takeup registered on the test roads at Edsvalla and Lasele is shown in Figure 4. The two Edsvalla curves represent two kinds of polystyrene foam: an expanded type (cut cell molded bead polystyrene) and an extruded type. As can be seen from the curves, the rate at which the expanded type takes up water is very high, with a water content of about 16 percent by volume after 5 yr. Consequently, this type of polystyrene foam is not recommended for frost insulation. At Lasele, one extruded polystyrene foam (extruded polystyrene skinboard) and one expanded polystyrene foam (molded polystyrene foam) have been tested. The curves show that the increase in water content is low for the extruded polystyrene foam and higher and somewhat irregular for the expanded polystyrene foam.

Of more direct interest, however, is the thermal conductivity, which has been determined on certain occasions (see Figure 5). The curves describe the development of the thermal conductivity with time from the same test roads as in Figure 4. The superiority of the extruded foam over the expanded is clearly recognized. In fact, after 12 yr at the Lasele test road, the thermal conductivity value for the expanded foam exceeds that of the extruded foam by a factor of nearly 1.5. This result has been applied by dimensioning the plastic foam layer (see design table, Table 1). The thickness of the foam layer in the table is related to the extruded polystyrene foam at Lasele (Styrofoam HI 50). When the bead-board type of polystyrene foam of the Lasele quality is to be used, the thickness in the design table must be increased by 50 percent.

The mechanical behavior of polystyrene foam material has also been investigated, but not to the same extent as the thermal behavior. Attaining sufficient mechanical strength of the polystyrene foam presents no difficulty today as the producers of the foams have all the knowledge necessary to manufacture the foam boards to the desired strength.

INVESTIGATION OF ICING IN THE TEST FIELD

When the investigation of the test roads previously mentioned was started, it was well known that icing on the road surface could occur earlier in autumn on heat-insulated sections than on uninsulated sections. Therefore, measurements of the skid resistance and surface temperature were performed along the test roads during autumn and winter. From these investigations, it was clearly apparent that there was a somewhat greater risk of icing on heat-insulated sections in comparison with uninsulated ones. This risk was strongly influenced by the construction type of the road base (see Figure 3), which shows the variation of the coefficient of friction for one special occasion on a number of the test sections on the Edsvalla test road. The friction values for the various insulated sections are related to the values of the uninsulated sections, which are given the value 1.

To study icing as a function of road construction, a special test field was organized at Linköping *(2)*. The object of the investigations in the test field was to make a general study of the interaction between climate and road construction that results in slippery conditions in cold weather, and to make a particular study of the risk potential of different road constructions, most of them insulated with plastic foam. A total of

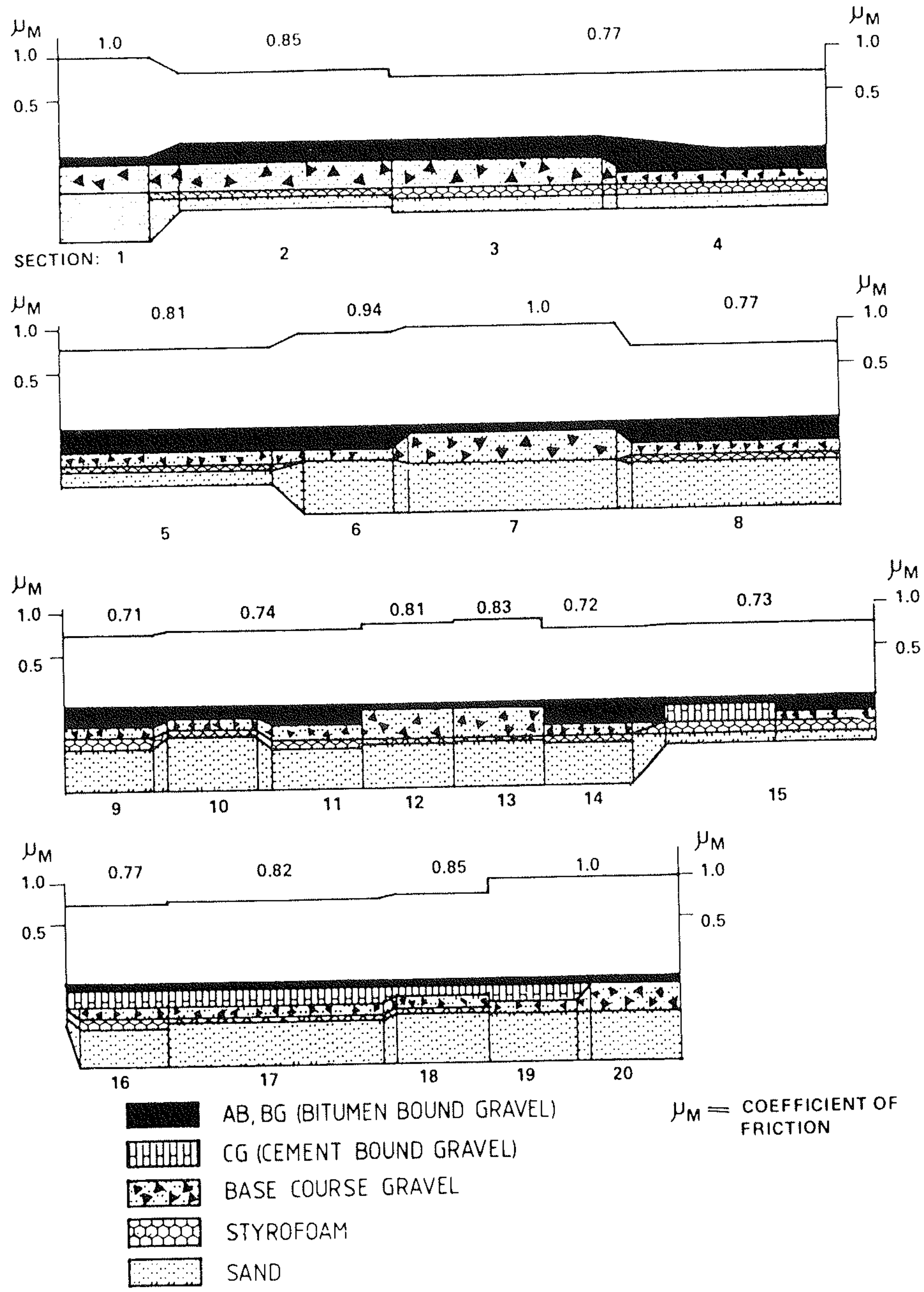

FIGURE 3 Test sections on test road Edsvalla 1967.

42 different pavement types were studied in the test field, see Figure 6.

According to K. Gustafson of VTI, the background for the icing process can be expressed as follows:

> The presence of an insulating layer in the road pavement affects the flow of heat upwards and downwards. In a heating situation, the insulation layer retards the downward flow of heat in the road, while in a cooling situation upward flow of heat from material courses underneath the insulation is retarded, and outward radiation of energy from the road surface is therefore compensated to a lesser extent than in a road without insulation. The consequence is that the insulated road generally has a somewhat lower surface temperature than most types of road without insulation. Surface temperature conditions in an insulated road are affected by (a) the depth at which insulation is laid, (b) its thickness, and (c) the materials in the courses above the insulation. The temperatures are also influenced by the thermal conductivity of the insulating material.

The results of the observations from the test field over the period of 1976 to 1980 are summarized and concentrated in a table, see Figure 7. In this table the different pavements are arranged according to the potential risk of icing. It can be seen that the highest risk, predictably, is found where the plastic foam layer is near the road surface or even far from the surface when the base is built up of crushed rock. It is also interesting to find that a base without plastic foam but with crushed rock near the road surface has a fairly high risk of icing. The lowest icing risk potential is shown by the uninsulated bases with natural ungraded gravel and sand. If the foam insulation is laid

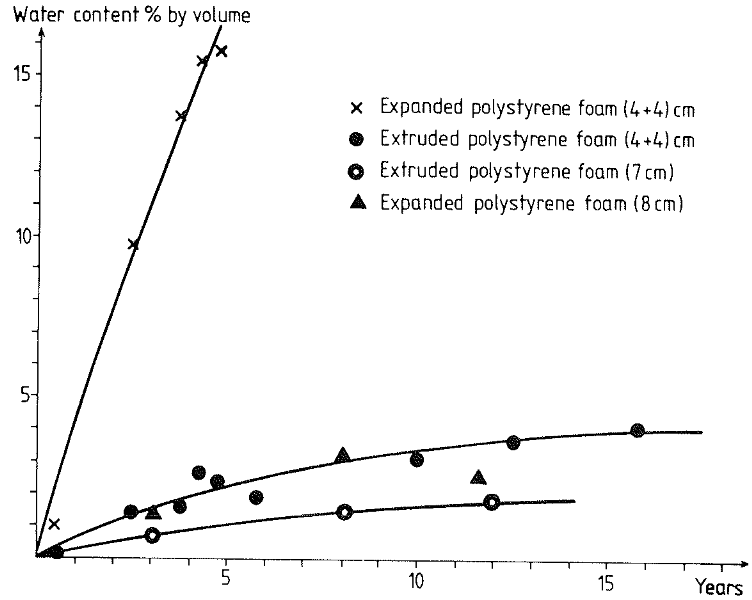

FIGURE 4 Moisture absorption in polystyrene foam, test roads Edsvalla 1966 and 1967 and Lasele 1972.

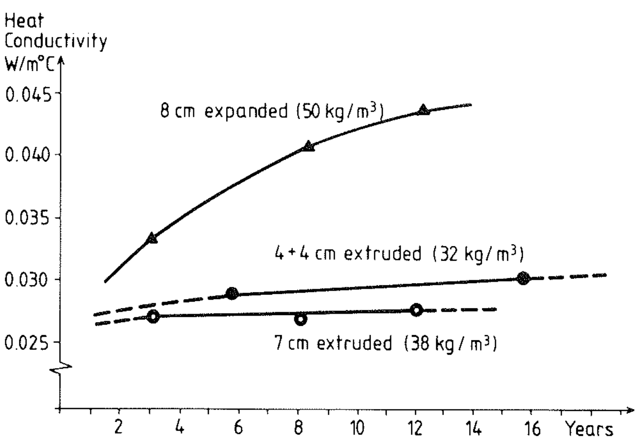

FIGURE 5 Change in heat conductivity of polystyrene foam with time on test roads Edsvalla 1966 and Lasele 1972.

at a depth of about 35 cm, the icing risk potential is high in relation to most uninsulated road structures, but is about the same as that of a pavement incorporating ungraded crushed rock below the asphalt concrete wearing course. For roads with the insulation laid at a depth of 50 cm or more, the risk of icing is relatively small, but is still somewhat higher than that of most uninsulated roads. Many roads in Sweden have been constructed with the insulation layer at a depth of 40 and 50 cm beneath the road surface without any serious icing problems. In the 1984 specifications for public roads, the lower depth of 40 cm is consequently allowed.

DESIGN AND DIMENSIONING OF A ROAD BASE INSULATED WITH POLYSTYRENE FOAM

Three layers contribute to the frost-resistance capacity (in degree-days) of a roadbase insulated with plastic foam. The greatest contribution comes from the plastic foam layer itself and only a smaller amount comes from the overlying bearing course. What is sometimes not considered, however, is the fact that the soil layers of non-frost-susceptible material immediately beneath the plastic foam layer contribute to a degree that cannot be ignored.

The frost-resistance capacity is defined as the frost quantity (freezing index) that is required for freezing the road base totally. The frost-resistance capacity is calculated by the formula of Sv. Skaven-Haug, of Norway, which is described as follows:

The calculation is based upon the model

$$F = \Sigma\Omega + E, \text{ h°C}$$

where F is the frost-resistance capacity and h°C is hr/°C.

The resistance to freezing from latent heat for a single soil layer is

$$\Omega = \frac{q \cdot s^2}{2\lambda} + q \cdot s \, \Sigma\left(\frac{s_0}{\lambda_0}\right), \text{ h°C}$$

where

s = thickness of soil layer (m),
q = frost-accumulating ability of material (kcal/m^3)
λ = heat conductivity (kcal/mh°C), and
s_0/λ_0 = resistance to heat flow of frozen layers (m^{2}h°C/kcal).

The freezing resistance due to heat flow is the earth to the frost line (stored heat in unfrozen soil), which can be expressed as

$$E = kGT\lambda \Sigma \left(\frac{s_0}{\lambda_0}\right), \text{h°C}$$

where

- k = a constant (usually 0.7),
- G = temperature gradient below frozen zone of February 1 (°C/m), and
- T = actual reference time (h) for stored heat.

In Figure 8 the frost-resistance capacity calculated (as already described) of a road base built up of layers of sand and gravel and of a roadbase insulated with polystyrene foam, is plotted as a function of the thickness of the gravel-sand base and of the thickness of the polystyrene foam layer *(3)*. However, these capacity values are not to be generally used. When calculating the frost-resistance capacity for any single base, appropriate input values have to be chosen *(3)*. The

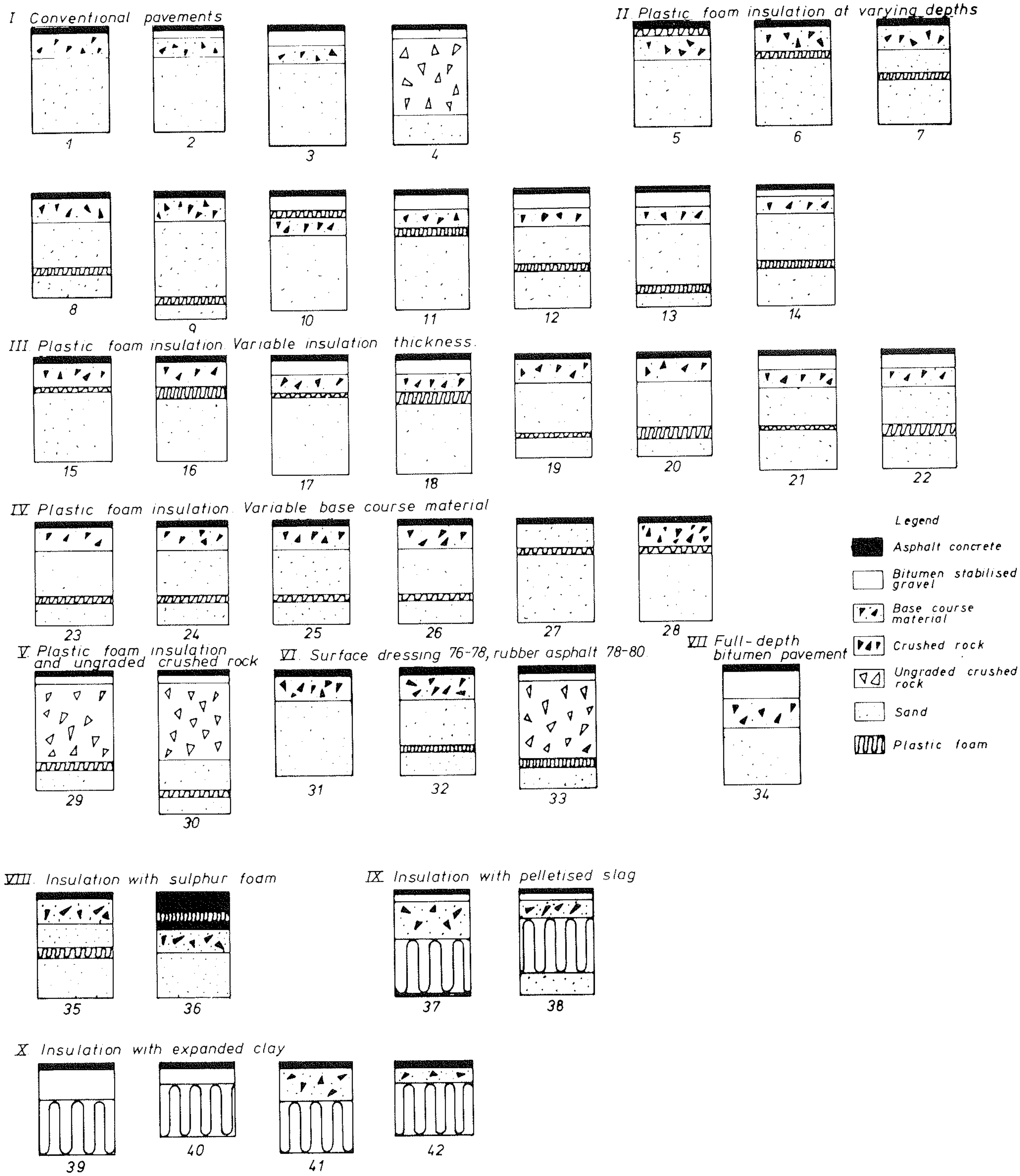

FIGURE 6 Pavement types in test field at Linköping 1976 *(2)*.

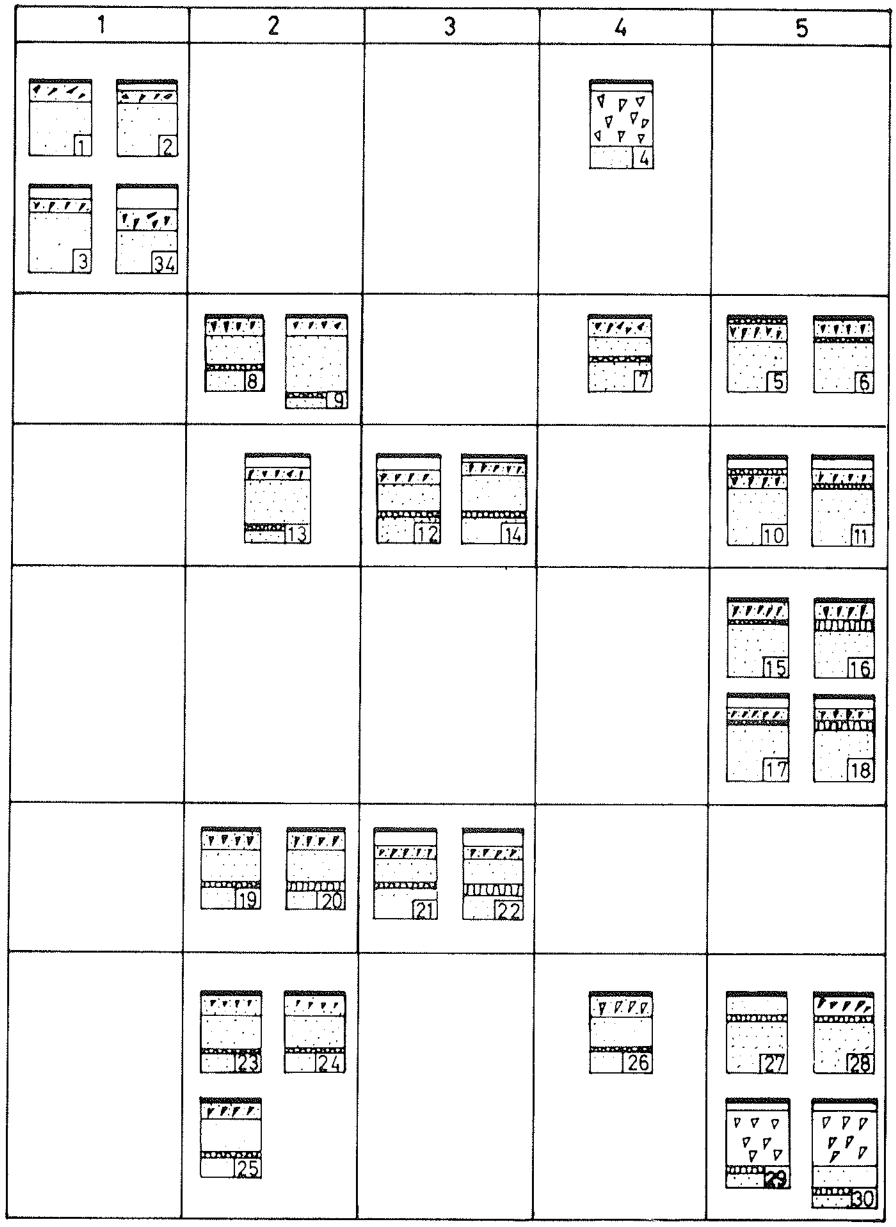

FIGURE 7 Icing risk potential of road pavements represented in the test field at Linköping 1976. Summary of measurements made over period 1976–1980 *(2)*.

design freezing index is assumed to be 1,000 degree-days for the particular part of the country. Thus, if during one winter the freezing index reaches the value of 1,000 degree-days, the bases will freeze but the subgrade will not. That means that by choosing either a 200-cm-thick roadbase of gravel and sand or a base insulated with 4.5 cm of polystyrene foam with a total base thickness of around 85 cm, 100 percent frost protection can be obtained for either base during that winter.

The frost-resistance capacity of the polystyrene foam insulated base is dependent mostly on the insulation properties of the polystyrene foam (thermal conductivity) and also on the water content of the layer of sand below. Figure 9 is a diagram describing frost-resistance capacity related to the thickness of the polystyrene foam for different thermal conductivity values of the foam.

Figure 10 shows the importance of the water content of the layers beneath the plastic foam. This acts as a freezing resistance layer and is consequently more efficient the more water it contains. As old roadbases are often built up of fine-grained and moist material, it may be an advantage to insulate them with plastic foam by merely placing the foam boards directly on the old road surface and constructing a new base upon the plastic foam layer.

The philosophy of this related art is the basis for the determination of the thicknesses needed for the polystyrene layer in the design table, Table 1. The thicknesses of the polystyrene layer including the non-frost-susceptible layer beneath it correspond to a frost protection degree of some 90 percent, allowing the frost to penetrate and act in the subbase or subgrade every tenth winter. During these winters there will consequently be some frost heave. The magnitude of the heave depends on the insulation properties of the roadbase.

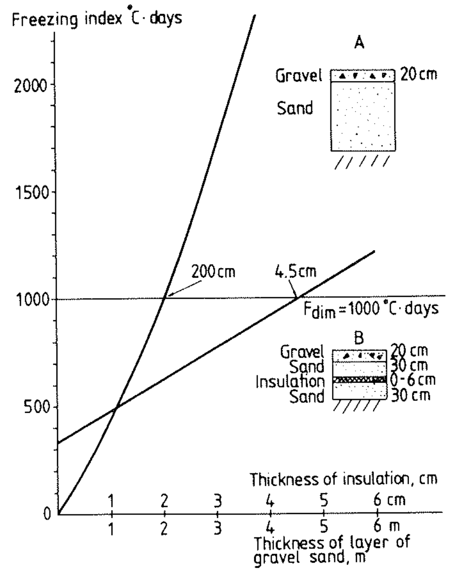

FIGURE 8 Example showing calculated frost-resistance capacity of sand-gravel roadbase and base protected against frost with polystyrene foam.

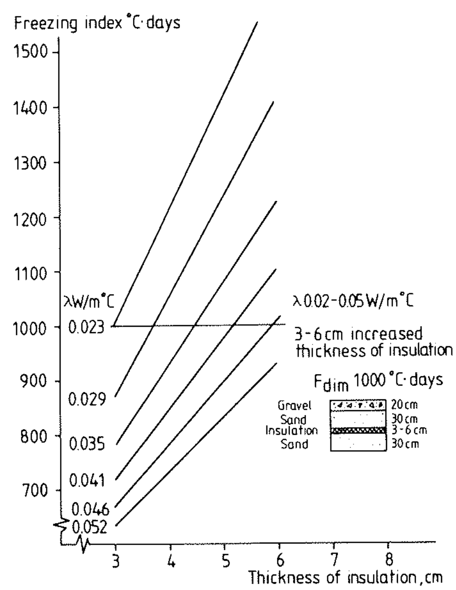

FIGURE 9 Frost-resistance capacity of roadbase with polystyrene foam for different values of thermal conductivity as a function of thickness of foam layer *(3)*.

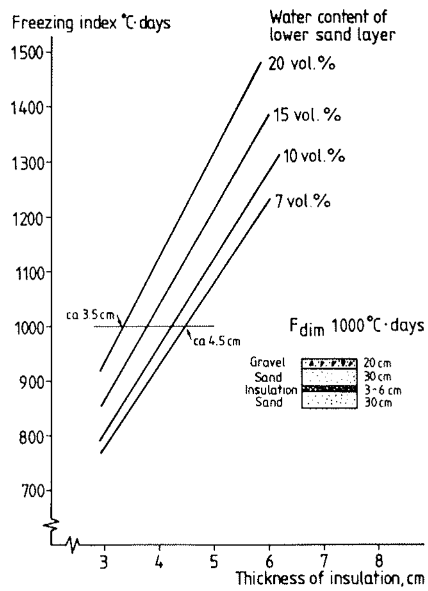

FIGURE 10 Frost-resistance capacity of roadbase with polystyrene foam for different values of water content of material in layers just beneath foam layer as a function of thickness of foam layer *(3)*.

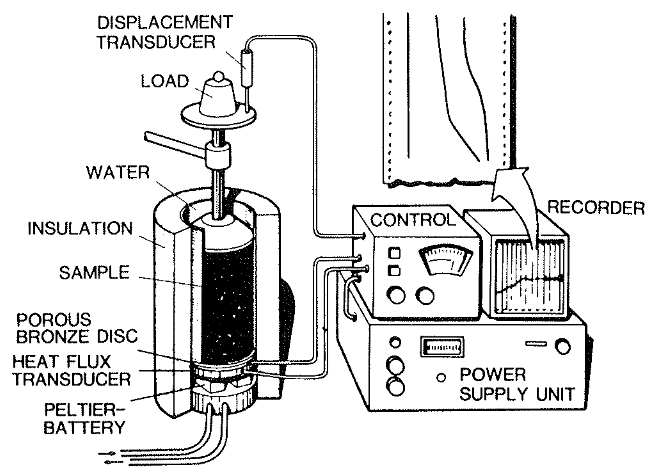

FIGURE 11 Swedish Road and Traffic Research Institute (VTI) equipment for freezing tests *(4)*.

A procedure has been developed that makes it possible to calculate the frost heave when the frost penetrates deeper than the base and the base is underlain by frost-susceptible soils. The frost heave depends on the thermal (road surface temperature) and moisture (water content and groundwater level conditions) road and subgrade structure, but most important are the heave properties of the soil in the subgrade. The latter are determined by an apparatus invented and accurately adjusted at the VTI *(4, 5)* (see Figure 11). The values describing the frost heave properties (ice segregation potential and sensitivity to load pressure) determined with the apparatus are directly usable in calculation of the frost heave. The calculations, which are computerized, run in steps from the natural autumn conditions through the winter.

The importance of heat conductivity with regard to frost heave is easily recognized from a calculation performed by L. Stenberg at VTI, see Figure 12. The thickness of a polystyrene foam layer is calculated for two values of the conductivity, both giving the insulated roadbase a specific degree of frost protection. From the curves it can be read off that when 4 cm of frost heave is allowed, the thickness of the foam needed is 3.5 cm when $\lambda = 0.045$ *W*/m°C, and 2.55 cm when $\lambda = 0.033$

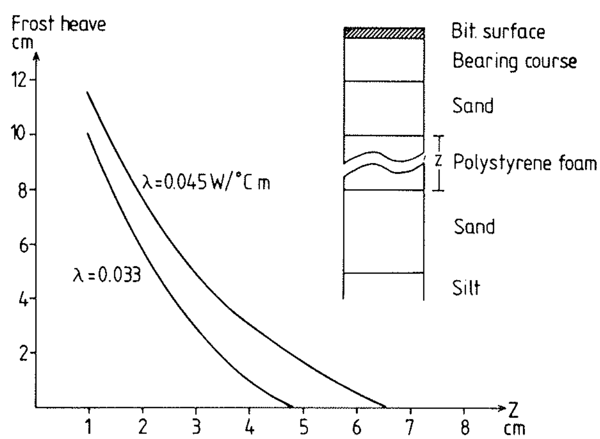

FIGURE 12 Frost heave as a function of thickness and heat conductivity in roadbase insulated with polystyrene foam. (Calculation by L. Stenberg.)

W/m°C. In this way, the thickness of the polystyrene foam layer can be appreciated for different values of the thermal conductivity.

DESIGN THERMAL PROPERTIES OF POLYSTYRENE FOAM

Polystyrene foam, which is an organic material, is relatively new and is available in several varieties and qualities. The main types of polystyrene foam are the expanded and the extruded. The extruded type has proved to have the best frost-insulating properties in the long run.

In order to discriminate between the different types of frost insulation material, it must be possible to determine the development of the heat conductivity during a proposed sequence of years.

In polystyrene foams the conductivity increases with time due to the water takeup. This applies both to expanded and extruded polystyrene foams. For extruded polystyrene foam, it must also be added that the heat conductivity increases because of the cell-gas exchange, which continues for a long time. The heat conductivity of the extruded polystyrene foams of the qualities known so far is, however, always lower than the heat conductivity of the expanded polystyrene foams.

To determine a heat conductivity value of polystyrene foam for dimensioning, it is necessary to perform accelerated tests in the laboratory and long-term tests in the field. To achieve this, a procedure has been worked out that is now undergoing final adjustment in cooperation with the National Testing Institute in Sweden. In this procedure, the water takeup is studied when the vapour gradient is increased in comparison with the natural conditions. An extra splitting of the boards forces the cell-gas exchange to accelerate. Values from the laboratory and the field are compared. Through extrapolation it is now possible to determine the development of the heat conductivity over a longer time with the help of known theories of water takeup and cell-gas exchange. From the curves describing the changes in heat conductivity with time it is possible to evaluate a heat conductivity value for dimensioning, see Figure 13.

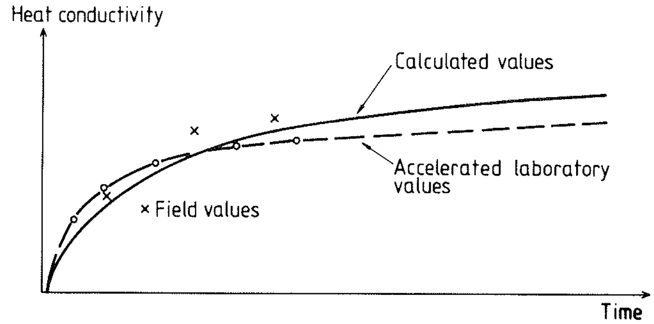

FIGURE 13 Variation of heat conductivity of polystyrene foam as a function of time due to moisture absorption and change in cell-gas content.

To assess the frost-insulating properties of a new and little-known material, a method of analysis is proposed by testing the materials in the laboratory on small test surfaces and eventually at natural field-test areas.

THE USE OF POLYSTYRENE FOAM ON ROADS IN SWEDEN

The main purpose of using polystyrene foam on Swedish roads is to counteract frost heave. As the frost heave in the north of Sweden can exceed 30 cm during an average winter, much more in an extremely cold winter, it is understandable that the frost protection measures must be of an efficient type. Insulation with polystyrene foam is one of them. According to the design table in Table 1, a standard insulation consists of a bearing course of 40 to 50 cm and a polystyrene layer of a thickness that depends on the thickness of the non-frost-susceptible layer underlying it. The thickness can be read off from the design table by starting with the average freezing index at the particular site, for example, 900 to 1,000 degree-days (°Cd). If the thickness of the non-frost-susceptible material under the foam layer is only 100 mm, the thickness of the foam should be 80 mm. But if the foam layer is to be placed on an old road surface and the road base thickness is 500 mm, the foam thickness can be reduced to 40 mm, with a saving in cost of the insulation material of 50 percent.

The thickness of the plastic foam layer in Table 1 is applicable for conditions where a uniform thickness of the insulation layer is appropriate. The most usual case is a road along which the frost-heave properties of the subgrade or the roadbase itself change irregularly. By dimensioning the foam insulation so that no detrimental frost heave arises even on those parts of the road where the heave potential is highest, longitudinal evenness of the road surface is achieved.

Difficulties caused by frost on Swedish roads become worse farther north because of cold winters and the high degree of frost susceptibility of the soils in the subgrade (see map and table on frost heave, Figure 2). The terrain that gives the most severe problem is that where the subgrade is made up of mixed layers with frost and non-frost-susceptible materials and where at the same time groundwater level is high. In this type of terrain, which is common in Sweden, insulation with polystyrene foam is used as an efficient frost protection measure.

Another field where insulation by polystyrene foam is used with success and is especially economically feasible is at culverts. Figure 14 illustrates how uneven frost heave can be counteracted. The figure gives the longitudinal section through a culvert in an ordinary road. The frost heave before and after the provision of the insulation is plotted. It is clearly demonstrated that insulation with plastic foam results in a smoother road surface.

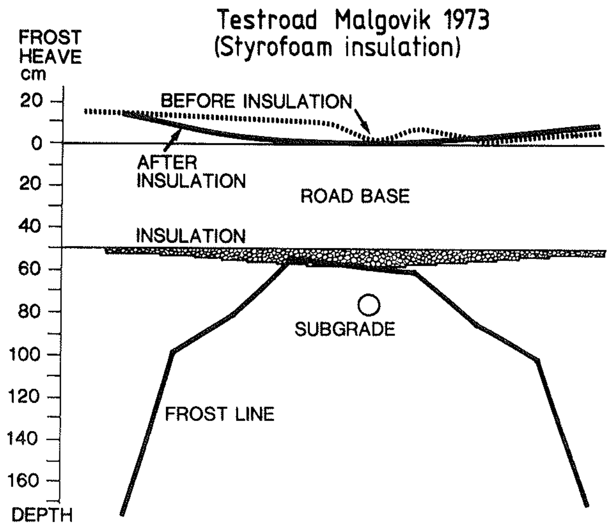

FIGURE 14 Insulation of culvert with polystyrene foam by double transitions.

Insulation with plastic foam at a culvert is achieved constructionally by tapering the foam layer. The same technique is also used as a transition when a conventional foam insulation along the road is to be terminated.

In the Swedish national road network every year, an area of about 150,000 m^2, which corresponds to a road length of 10 to 15 km, is insulated with polystyrene foam. This figure will certainly increase in the future, when the technique of designing and dimensioning has been further improved and the application of the insulation concept has been even better adapted in practice.

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The Role of Extruded Expanded Polystyrene in Ontario's Provincial Transportation System

J. B. MACMASTER AND G. A. WRONG

Frost action is an aggravating and costly problem in regions where highways are exposed to sub-zero temperatures for prolonged periods. Pavement distortion can create hazards for the driving public and contributes to the deterioration of the overall pavement structure. Special highway design and construction features are often required to deal with this problem. Such treatments have included drainage improvements to the pavement structure and excavation of frost-susceptible material. Because of the excessive frost penetration in northern Ontario, however, such treatments are not always practical. In 1966, the Ontario Ministry of Transportation and Communications began using expanded polystyrene to insulate the subgrade at frost heave sites. Traced in this paper is the work carried out by the Ministry to evaluate the effectiveness of polystyrene. The establishment of specifications and standards currently in use to control frost heaving are also discussed. Reference is made to the Ministry's site-selection criteria on minimizing the potential for pavement icing over insulated sections. The Ministry also uses expanded polystyrene to insulate sewer and water mains and to prevent differential heaving at critical installations such as truck weigh scales. Standard drawings are included that illustrate the installation details. The paper concludes by describing several special projects on which relatively large quantities of polystyrene were used to control frost heaving.

In areas where highways are subjected to extended periods of sub-zero temperatures, differential frost heaving is an aggravating and often costly problem for roadway agencies. It can create serious safety hazards for motorists and cause permanent pavement deformation. Prolonged heaving will adversely affect the performance of a pavement and may eventually threaten the integrity of the pavement structure. Utility crossings such as sewer and water mains, which are located within frost zones, are vulnerable to freezing if left unprotected.

Special roadway design and construction features are often needed to minimize the effects of frost heaving.

It is accepted that all roadways heave during the winter season. This would not necessarily be a significant problem if the magnitude of this movement was the same at all points. Unfortunately, inconsistencies in the composition of roadbed materials and inadequate transitions between different materials such as earth and rock can lead to differential heaving. During construction a conscious effort is made to ensure that materials within each layer of the pavement structure, and in the subgrade directly underneath, are uniform. Standards have been developed so that the compaction efforts applied during the placing of these processed and native materials are consistent.

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The three principal factors responsible for frost heaving include freezing temperatures, frost-susceptible soil, and a moisture supply at or near the freezing front. Obviously the latter two are the factors over which there is the most control. Nevertheless, they may still contribute significantly to differential heaving despite efforts to construct homogenous subgrades and provide uniform compaction.

Pockets of subbase and subgrade materials often include high concentrations of silt and fine sand, materials that have an affinity for moisture. The magnitude of the voids in silts and fine sands are such that water travels quickly and is attracted from relatively great distances to form the ice lenses that grow with increasing accumulations of moisture.

TREATMENT OF FROST HEAVES

Excavation

Weeks and months of prolonged freezing temperatures are common to many states and provinces, hence efforts are made to deal with one or both of the controllable factors that contribute to frost heaving. The principal method of treatment used by the Ministry of Transportation and Communications (MTC) before 1966, and one that continues to be implemented in many cases, was the removal of the frost-susceptible material by excavation. Figure 1 represents the provincial design standard for this treatment.

All frost heave sites are monitored annually in late winter and early spring until a rehabilitation project is carried out on the highway. The sites are then drilled during the summer with power equipment to determine types and depths of the native materials along with current moisture conditions. The design value for the depth of excavation, t, varies with the depth of frost penetration and is selected so as to remove as much of the frost-susceptible materials as possible, bearing in mind the amount of excavation involved and the fact that it must be properly drained. The value for t varies from 1.2 m in southern Ontario to 1.5 m in northern and northwestern Ontario.

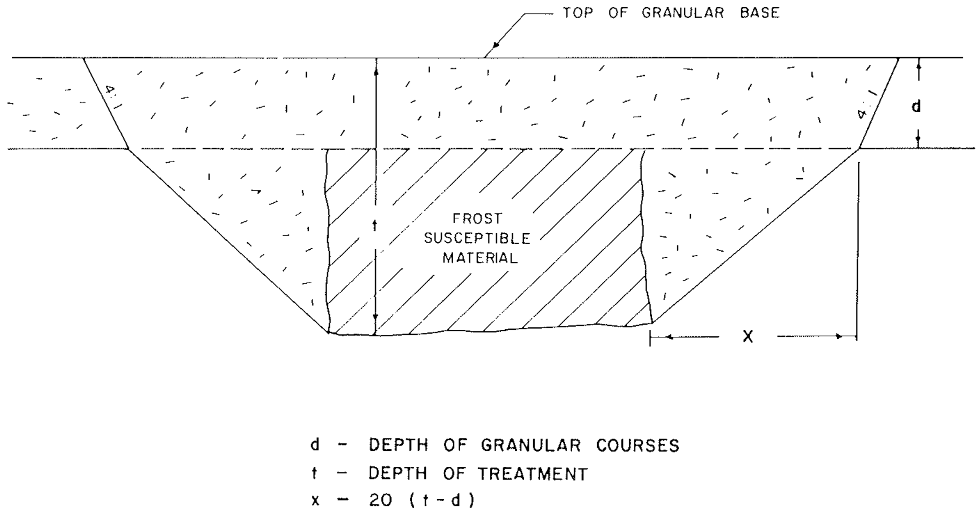


FIGURE 1 Excavation for treatment of frost heaves.

Drainage of the treated zone is influenced by the depth and efficiency of the highway ditch system. Side ditches are normally excavated to at least 0.5 m below subgrade level. The length of taper required for smooth transition into and out of treated areas is determined using the formula $x = 20\ (t - d)$,

where

x = length of taper,
t = depth of treatment, and
d = depth of granular courses.

Drainage

Frost heaves of low severity are often treated successfully by improving the drainage of the pavement structure. This can take the form of improved ditching or ditch cleanout, or the installation of perforated subdrains beneath the shoulder of the highway. The depth of the ditches or drains is, obviously, dependent on the availability of outlets to remove the collected water. Another factor that has a major bearing on the success of drainage is the presence of bedrock. Bedrock that is relatively close to the surface makes excavation, ditching, and the installation of drains difficult as well as expensive. Contract tender quantities for rock removal tend to be small and special equipment and expertise have to be brought to the site.

Insulation

Research into the use of expanded polystyrene began in the United States in the early 1960s *(1)*. The first major research sites in Canada were established near Winnipeg, Manitoba, in 1962 and Sudbury, Ontario, in 1965 *(2)*.

At the Sudbury site on Highway 69, seven frost heaves were treated with expanded polystyrene. The depths installed were 50 mm and 75 mm. Results obtained from these trials between 1966 and 1968 showed that with an average of 1,200 degree-days C, the 50 mm of insulation permitted frost to penetrate from 0.3 m to 0.8 m, whereas the 75-mm layer prevented virtually all frost penetration below the boards.

As a result of the Sudbury trials, the MTC proceeded to treat numerous frost heave sites with the intention of preventing differential vertical movement. It was discovered, however, that although the main heave was eliminated, bumps often developed at either end of the insulated area. This was attributed to the fact that the surrounding roadway was heaving more than the treated section. Through experimentation and performance evaluation, the MTC eventually revised its policy to one of providing for controlled heaving. This meant that the differential heaving would be reduced to the extent that any hazard to the motorist would be eliminated and driving discomfort would be minimal.

EARLY CRITERIA FOR HIGHWAY INSULATION

The thicknesses of expanded polystyrene used by the MTC in the early days of highway insulation depended primarily on the number of degree-days of frost at each site (Figures 2 and 3). The rule of thumb used was 25 mm of insulation for each 555 degree-days C to prevent any penetration. Through trial and observation, other variables such as the minimum depth of granular material required above the insulation boards were determined. Development of the appropriate taper lengths and configurations necessary to minimize the previously mentioned bumps was also carried out. Preliminary transition standards introduced by the insulation manufacturer detailed a decrease in thickness from 80 mm to 25 mm in two steps of 2.4 m and 4.8 m respectively (Figure 4). The boards were placed so that the ends of adjacent rows were staggered by 1.2 m.

To establish the most effective use of expanded polystyrene as a highway insulating agent, the MTC constructed a full-scale insulation research site in northern Ontario in 1972. It was hoped that solutions to specific problems involving current construction techniques and design standards could be formulated.

VAL GAGNE TEST SITE

Objectives

The research installation was established in cooperation with Dow Chemical of Canada, Ltd. *(3, 4)*. The location of the

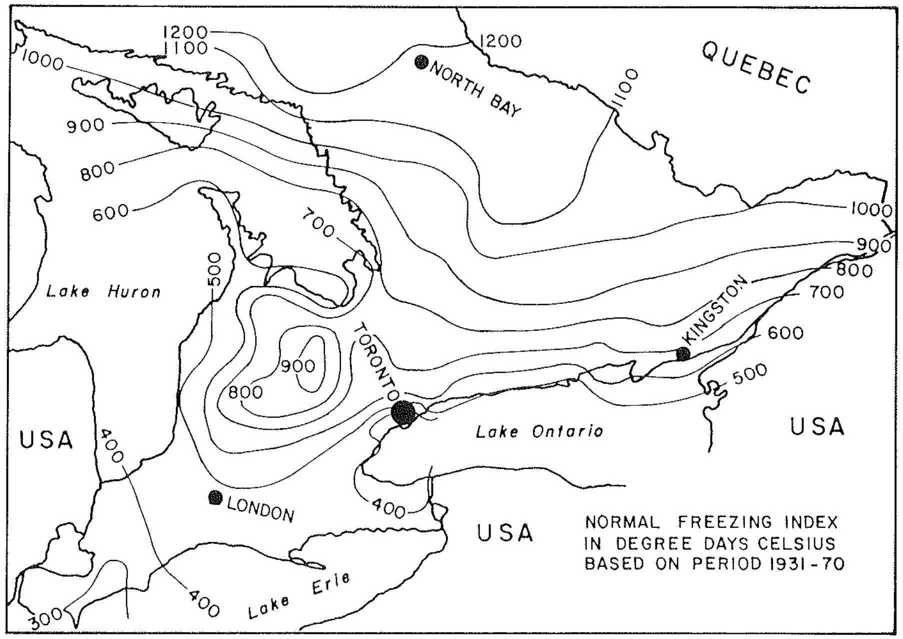


FIGURE 2 Freezing indices in southern Ontario.

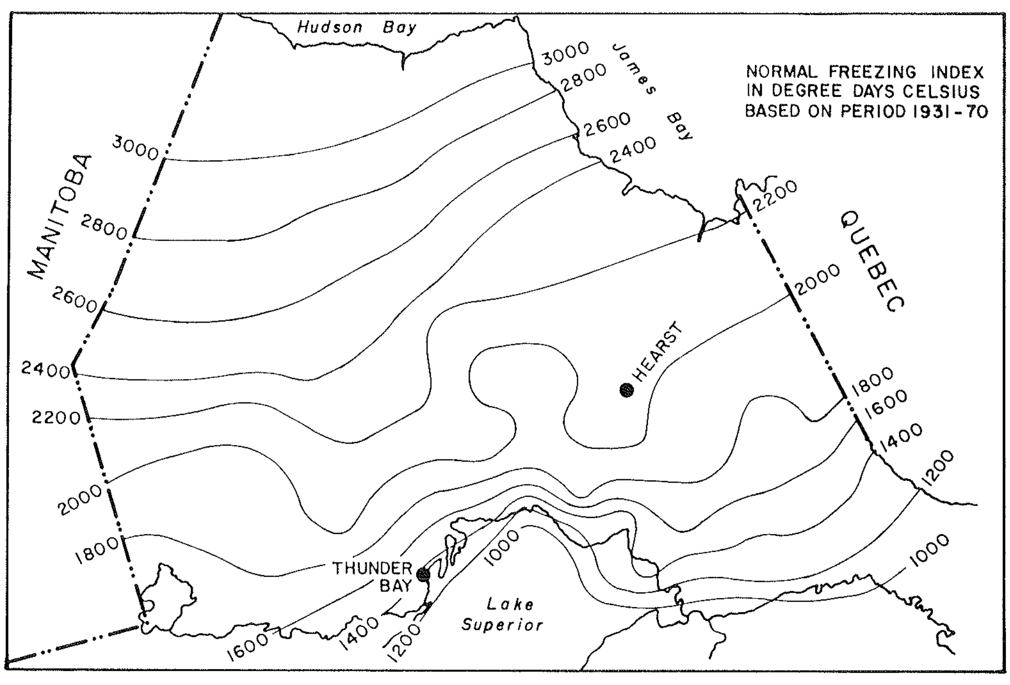


FIGURE 3 Freezing indices in northern Ontario.

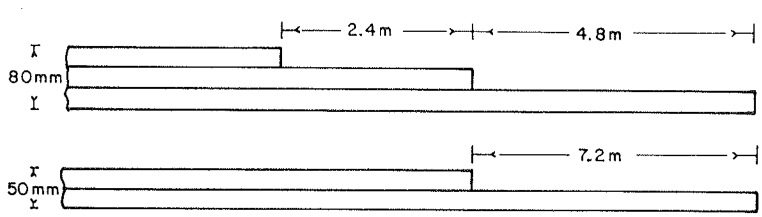


FIGURE 4 Early transition treatments for expanded polystyrene.

experimental site was at Val Gagne, 65 km south of Cochrane, Ontario.

Included in the basic objectives of the experiment were the following:

1. The development of a layout design to control the bump created at the transition from insulated to noninsulated sections,
2. The investigation of the depth of frost penetration below various thicknesses of insulation,
3. Investigation of the phenomenon of pavement icing over insulated sections,
4. Observation of thermal conditions at the edges of the insulation layer, and
5. Study of the durability of the polystyrene insulation boards.

Site Conditions

The Val Gagne area traditionally experiences severe winter weather with freezing indices of 1800 degree-days C or greater. Frost depth indicators have shown that penetration to a depth of 2.4 m to 3.0 m is common *(4)*.

The section of highway chosen for the experiment was situated in a 1.2 m to 1.5 m cut. The native soil consisted principally of a silty clay, classified as CH under the Unified Soil Classification System or an A-7-6 by the AASHTO system.

Design and Construction

The experiment was designed to evaluate three different thicknesses of expanded polystyrene: 25 mm, 50 mm, and 75 mm. Trial areas were 29 m in length plus transitions. They were separated from each other by noninsulated control sections. The width of the insulation layers was extended 1.8 m beyond the edge of pavement.

The insulation boards were 2.4 m × 0.6 m in size and tack-coated to the existing asphalt pavement with an emulsion. A 16.0 mm minus granular base material was end dumped and spread over the insulation to a depth of 450 mm with a small bulldozer. The entire site was then paved with 75 mm of asphaltic concrete.

Full site instrumentation was installed to measure ambient ground temperatures, frost penetration, and pavement heaving. Thermocouples and modified Swedish-designed frost depth indicators were used to monitor the first two variables and pavement movements were recorded by taking levels of nails embedded in the pavement. Details of the instrumentation schemes are described by Chisolm and Phang *(4)*.

Results from Monitoring Data

During the winter months, temperatures above insulation are expected to be lower than those encountered below. The effect 50 mm of polystyrene had on the temperature regime within the roadbed for a typical Val Gagne winter is shown in Figure 5. Above the insulation, a maximum temperature of –40°C was recorded, whereas directly beneath the polystyrene the coldest temperature recorded was –3°C.

Frost penetration depths ranged from 1.0 m for 75 mm of polystyrene to 2.5 m for no insulation during the winter of 1975–1976. Frost penetration for all test sections over the 5-year monitoring period is summarized in Table 1.

The extent of frost heaving was significantly reduced with increasing thicknesses of insulation. The 50 mm and 75 mm thicknesses, illustrated in Figure 6, practically eliminated heaving at the site.

As stated in the objectives, it was intended to monitor the three test sections for icing. Because of the site location, however, only sporadic observations were possible and no further efforts were expended in this area. On a few occasions, a small amount of icing was noted.

Frost penetration measurements made transversely indicated that reducing the standard width of the insulation beyond the

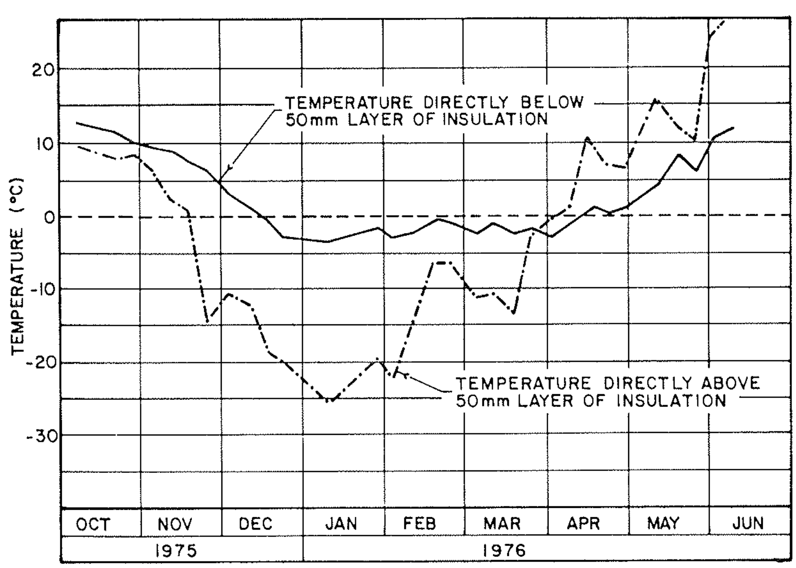


FIGURE 5 Temperature regime at test section containing 50-mm expanded polystyrene.

TABLE 1 AVERAGE FROST HEAVE AT CENTERLINE OF HIGHWAY

Section	1972-73	1973-74	1974-75	1975-76	1976-77
Control,no insulation	30	37	0	34	18
25 mm insulation	9	24	3	9	2
50 mm insulation	0	6	3	3	0
75 mm insulation	0	6	0	3	0

pavement edge from 1.8 m to 1.2 m would not substantially alter the lateral penetration profile.

Samples of the insulation board were recovered after 5 years and submitted to the laboratory for testing. The lab results are summarized in Table 2. Only the 25-mm thick sample exhibited a loss in thermal conductivity in the wet (field) state. The 50 mm and 75 mm samples showed negligible moisture pickup. Very little decrease in vertical compressive strength was observed in all samples.

Based on measurements made at the Val Gagne experimental site, a diagram of frost penetration versus thickness of insulation was created (Figure 7). This chart provides the designer with guidelines to determine the insulation thickness needed to control frost penetration and hence frost heaving.

MTC CRITERIA FOR HIGHWAY INSULATION

Selection of Insulation Sites

As mentioned at the outset, the excavation and backfill technique of treating moderate and severe frost heaves on Ontario highways is still in use. The decision to use this technique is, however, based on several factors such as ease of drainage and potential for disruption to traffic flow. If the length of the area affected by the frost action is extensive, then polystyrene may be used to avoid excessive excavation of roadbed materials. Occasionally several frost heaves are encountered in close proximity. In this situation, the use of insulation often has a distinct economic advantage.

Frost heaving in highway cuts can be caused by high water tables. The obvious course of action would be to improve ditching; however this approach can be complicated because of impediments such as rock knobs within the ditch line, or the need to ditch through bedrock to obtain a suitable outlet.

Frost heaving is common at transitions between earth and rock fill, or at the limits of rock cuts. Expanded polystyrene becomes a viable option for treatment to avoid costly and difficult removal of roadbed materials. This option also minimizes disruption to motorists where traffic movement must be maintained.

In northern Ontario, highways cross numerous deposits of swamp and muskeg that may be many meters deep. In order to minimize long-term settlements of the roadbed, the designer must keep the profile of the pavement as low as possible to

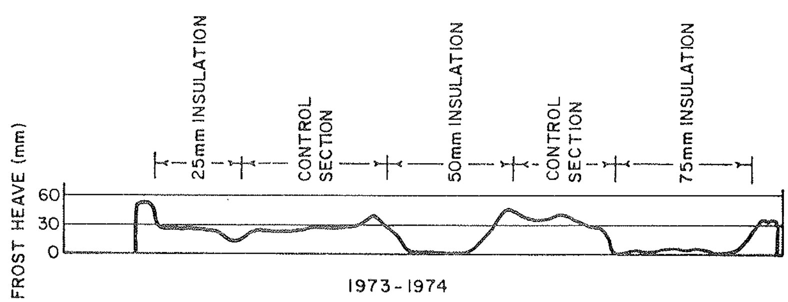


FIGURE 6 Frost heaving at Val Gagne test site.

TABLE 2 LABORATORY TEST RESULTS OF SALVAGED EXPANDED POLYSTYRENE

Test Section	'K' Factor (wet) ASTM C-518 $\frac{W}{m^2 - 0^oC}$	'K' Factor (dry) ASTM C-518 $\frac{W}{m^2 - ^oC}$	Water Content % By Volume	Vertical Compressive Strength @ 5% Defl. kPa	@ 10% Defl. Kpa
25 mm Thickness	1.329	1.203	3.95	291.7	337.9
50 mm Thickness	1.260	1.203	1.29	519.2	577.8
75 mm Thickness	1.243	1.197	0.93	344.1	344.1

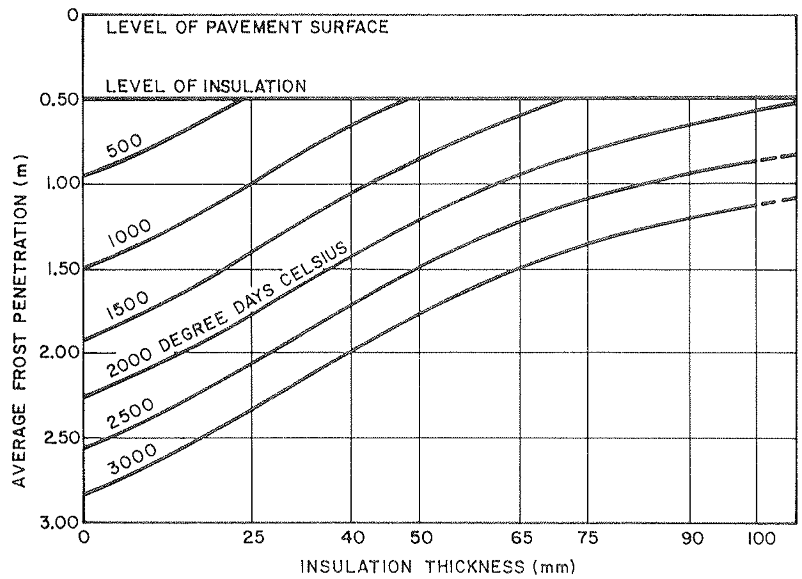


FIGURE 7 Design guidelines for selection of insulation thickness.

avoid overloading the sensitive subgrade. Drainage of the roadbed, however, can become a concern especially if the water table is at or near the surface of the swamp. The efficiency of standard highway ditches is substantially reduced as provision of outlets is difficult.

Because of the extensive penetration of frost in this part of the province, severe differential heaving of the roadbed can take place over swamps and muskeg. This differential heaving often results in cracks that develop roughly parallel to the centerline (Figure 8). They can occur at the centerline or anywhere within the pavement surface. Cracks up to 100 mm in width have even appeared within the granular shoulder.

The formation of these severe cracks is accentuated by the fact that the outer shoulder areas and side slopes are covered with snow during the winter. This deep cover of snow acts as insulation and prevents the frost from penetrating as deeply as it does beneath the cleared travelled portion of the highway. This is shown in Figure 9 *(5)*. The resultant heaving thus creates a "broken back" type of movement in the roadbed and the stresses induced in the bituminous surface literally cause the pavement to split in a longitudinal direction.

FIGURE 8 Cracking due to frost heaving in swamp.

This phenomenon had for years repeated itself in a section of Highway 17 that crosses an area known as the Raith Swamp, some 100 km west of Thunder Bay. The cracks along the centerline opened up each year to widths in excess of 200 mm, thus presenting a serious traffic hazard. Several remedial treatments were attempted over the years with little success. A trial area was established in which a 2.6-mm gauge wire mesh was embedded in the asphalt, but the extreme forces generated in the expanding pavement caused the mesh to rupture.

In 1977, a short reconstruction contract was carried out and experimental sections using polystyrene were included in the project to evaluate its effectiveness in controlling the serious heaving experienced in this area *(5)*. Two 150-m test sections were established in which 50 mm and 80 mm thicknesses of expanded polystyrene were used to insulate the roadway. The insulation was placed within the subbase layer and covered with a total of 600 mm of granular base course and 50-mm hot mix.

After 2 years of detailed monitoring, the serious longitudinal cracking and stepped transverse cracking previously found at this site had not recurred, whereas extensive patching was required in the noninsulated section. It was also noted that the overall pavement performance was quite similar in both the 50 mm and 80 mm trials.

Since the experiment, a total of 740,900 board ft of expanded polystyrene have been placed in two contracts over the Raith Swamp, effectively eliminating the serious heaving problem.

Standard Treatment

From the results of the Val Gagne trials and observations of the performance of numerous subsequent installations, a set of

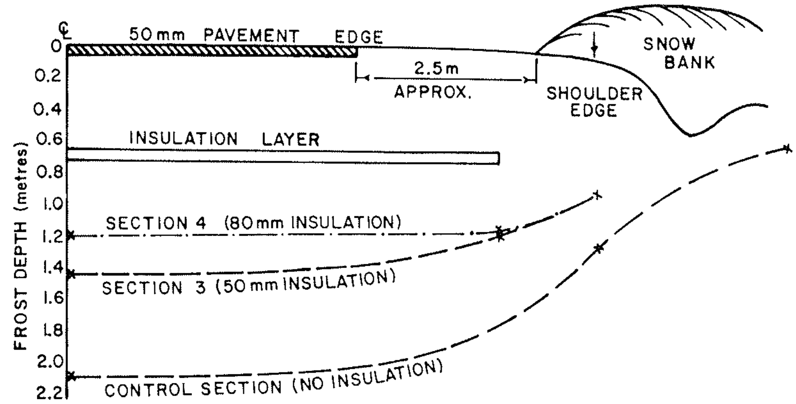


FIGURE 9 Effect of snow cover on frost penetration.

MTC standard drawings has been developed for expanded polystyrene installations on Ontario highways. Shown in Figure 10 is the layout for a single 25 or 40 mm layer of insulation *(6)*. Experience has shown that the "saw tooth" configuration at the ends of the treatment has been most effective in preventing the formation of bumps in the transition zones.

The insulation is extended beyond the edge of pavement a distance of 1.8 m, or that equivalent to the shoulder width, whichever is less. The polystyrene can be placed in the roadbed using excavation and backfill construction. This design suits conditions where raising the profile grade of the roadway is undesirable (Figures 11 and 12). The boards are placed at the bottom of the excavation and held in place with wooden skewers. Alternatively, several shovels of granular material can be placed on each board to prevent it from shifting or being blown about. If more than one layer of insulation is applied, succeeding layers are placed so that they cover the joints in the layer immediately underneath *(7)*.

Where it is more economical, the insulation boards can be placed directly on the existing pavement and covered with a granular lift. In order to provide maximum protection for the insulation, the compacted granular cover should be no less than 300 mm in depth.

Selection of Insulation Thickness

When the MTC began using expanded polystyrene to treat frost heaves, the intent was to eliminate any frost penetration, hence relatively thick layers of insulation were used. It was common to place anywhere from 75 mm to 100 mm polystyrene on

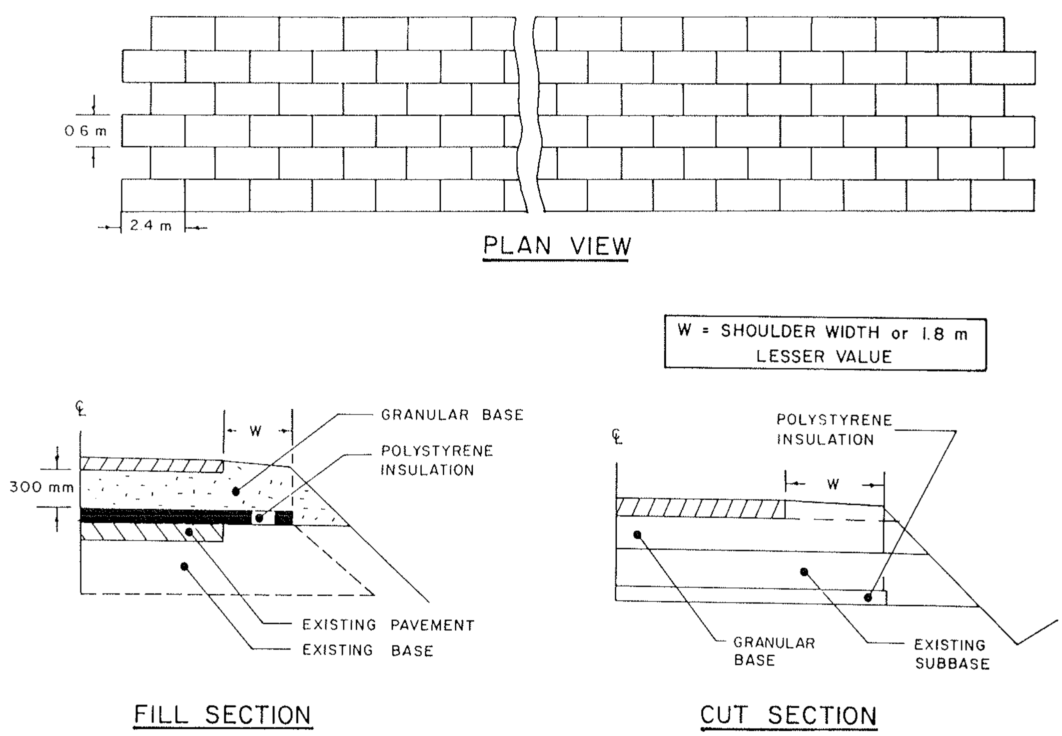


FIGURE 10 Use of expanded polystyrene for frost heave treatment.

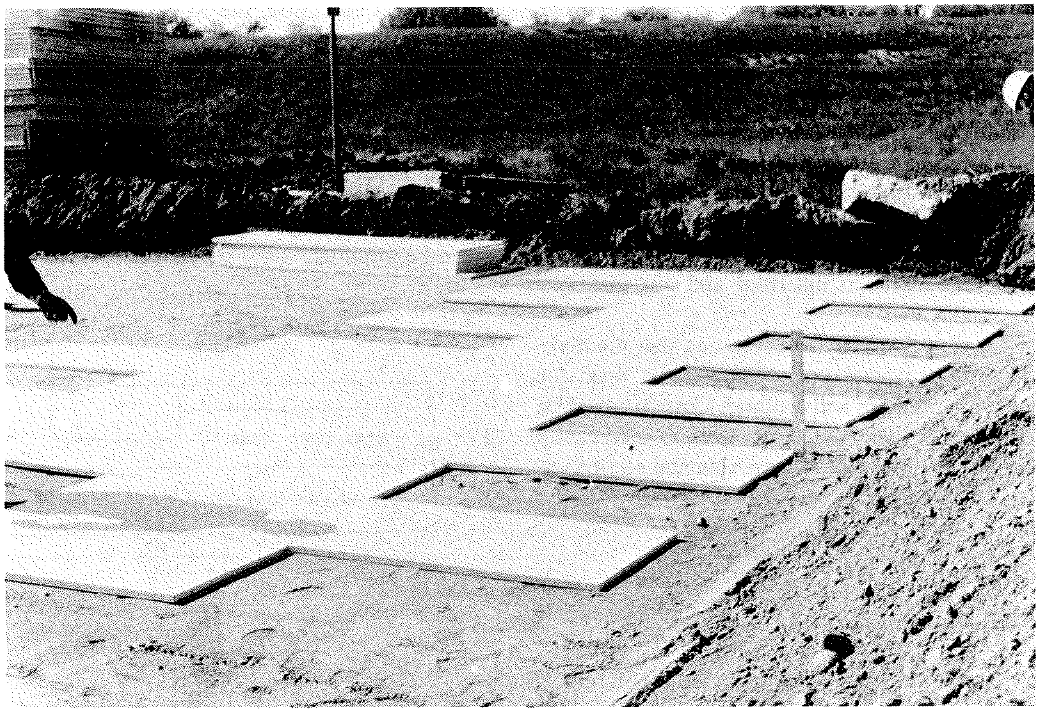


FIGURE 11 Placement of insulation boards at end of treated area.

FIGURE 12 Frost heave treatment using expanded polystyrene.

highways located in northern Ontario in an effort to prevent heaving *(8)*.

As MTC's experience with frost action and insulation increased and it became apparent that all roads heaved to a certain degree during the winter months, the philosophy of preventing frost heaving gradually changed to one of controlling frost heaving by permitting some frost penetration. This would provide for a tolerable ride over the frost heave area and greatly reduce the severity of any bumps that might develop in the zones of transition between insulated and non-insulated roadbed.

In 1978, a policy directive was issued stating that the maximum thicknesses of expanded polystyrene to be used for treating frost heaves in northern and southern Ontario would be 40 mm and 25 mm respectively *(9)*. This criterion is currently being adhered to in Ontario except for experimental sections or applications such as weigh-scale sites where very little heaving can be tolerated.

Insulation of Utilities

Protection of sewer and water mains from freezing is an important consideration in the design of such systems. Ideally, the lines are located below the maximum frost penetration limit; however, this may not always be possible. Locating the mains below the frost line could involve excavations up to 3.0 m in depth in northern Ontario. Excavating trenches through rock is very expensive and difficult and high water tables may necessitate locating the lines close to the ground surface. The protective covering over mains may be reduced if road profile grades require lowering during reconstruction of streets.

A cost-saving alternative is to insulate the main with expanded polystyrene. Illustrated in Figure 13 is a typical design used by the MTC in the town of Geraldton where the water table was unusually high. The freezing index for this area is approximately 2,000 degree-days C. Native soil is used for backfill between the insulation covering and the pavement structure to maintain uniformity within the subgrade. The width and height of the polystyrene installation is dependent on the diameter of the main. Sufficient clearance must be allowed between the pipe and the insulation for proper placement and compaction of the backfill.

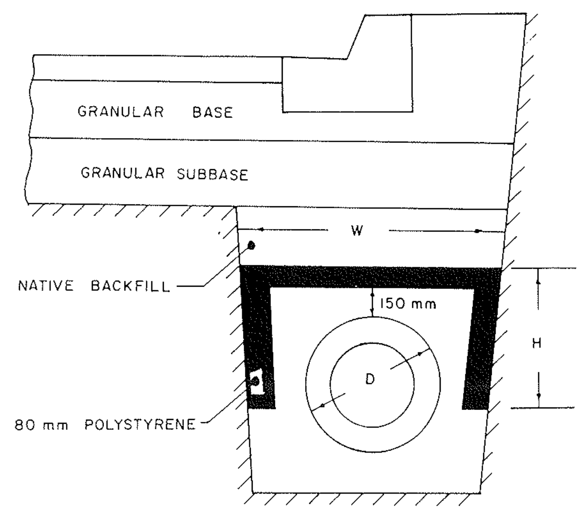
Pipeline crossings present a slightly different problem with respect to frost heaving. Because the contents in the pipeline are usually warm, the material surrounding the pipe does not experience the same degree of freezing as that at a greater distance. A situation is thus created where a dip in the road surface occurs caused by heaving away from the pipe.

On several occasions pipelines have been boxed in with polystyrene boards, thus preventing heat loss into the surrounding backfill material, and modifying the differential vertical movement.

Weigh Scale Sites

The MTC is responsible for regulation of the trucking industry in the province of Ontario. In order to monitor excessive loading and to ensure that commercial vehicles are in sound mechanical condition, the MTC operates 48 truck inspection stations throughout the province.

These sites are located adjacent to the highways and consist basically of an underground scale, scale house, and paved parking area. It is crucial that these scales operate accurately at all times. The differential in vertical movement between the concrete approach slabs and the scale platform should be kept to a minimum. The overall heaving of all these components must be minimal.



D (mm)	W (m)	H (m)
600 AND LESS	1.2	0.6
675 / 750	1.5	0.6
840	1.5	0.9
900 & GREATER	1.8	0.9

FIGURE 13 Expanded polystyrene insulation for sewers and water mains.

Expanded polystyrene is placed at all truck inspection stations, as shown in Figure 14, to control heaving because of frost action. The standard thickness employed in Ontario is 75 mm. The insulation is placed below the approach slabs and the scale pit and is extended outward to protect the asphalt driving lane and parking area.

Pavement Icing

A phenomenon associated with highway insulation is the occasional occurrence of pavement icing above treated areas *(5)*. The principal cause of pavement icing is the trapping of ground heat beneath the insulation. The pavement is hence somewhat cooler than that in noninsulated sections, where it tends to be warmer due to the latent ground heat.

Under certain conditions of high humidity, or if rain or snow is present, ice may form on the cold surface and present a

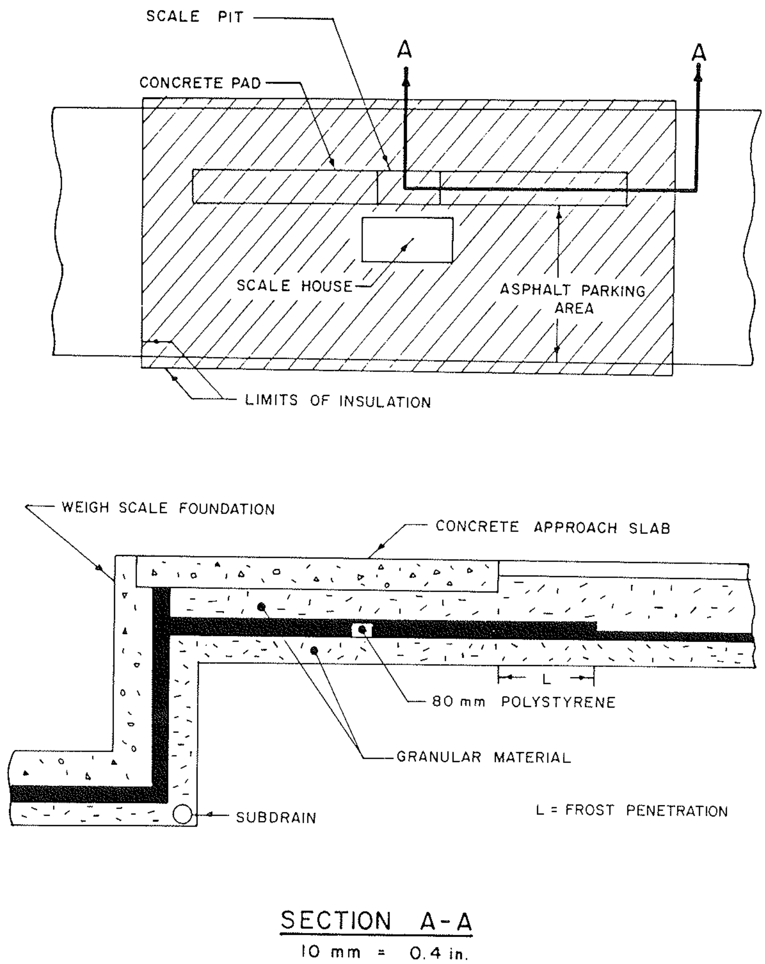


FIGURE 14 Expanded polystyrene insulation at weigh scale.

potential hazard to the motorist. The possibility of ice formation increases with the thickness of the insulation layer. It has also been noted that the closer the polystyrene is to the roadway surface, the greater the possibility of icing.

In northern Ontario where the bulk of expanded polystyrene has been placed, temperatures tend to be consistently cold all winter, so pavement icing has not been a serious problem. District maintenance staff, nevertheless, give the same attention to sections of insulated highways as they do to all other areas to ensure that adequate sanding is carried out should icing occur.

In the interests of safety, MTC policy *(9)* dictates that expanded polystyrene will not be used for treatment of frost heaves where a demand for increased traction or braking is foreseen. Such sites include horizontal curves, crests of hills, intersections, and railway crossings.

USE OF EXPANDED POLYSTYRENE IN ONTARIO

Since the introduction of expanded polystyrene to Ontario in 1966, the technique of controlling frost heaving in the highway system has been implemented on a continuing basis.

The bulk of the polystyrene used has been placed in the northern and northwestern regions of the province, although the number of rehabilitation projects containing expanded polystyrene on an annual basis has declined somewhat since the late 1970s. In recent years, insulation has been applied on fewer jobs, albeit in larger quantities per project. Some of these specific uses are described later in this report.

In the late 1960s, the MTC was using up to 1.5 million board ft each year. By 1972, there were approximately 300 frost heave sites containing insulation *(8)*. Quantities used in the northwestern region alone are summarized in Table 3. Since 1981, the application of expanded polystyrene in Ontario highways has been as follows:

Year	*Expanded Polystyrene (board ft)*
1981	1,650,000
1982	2,400,000
1983	1,575,000
1984	1,380,000
1985	2,244,700
1986	362,500

Since 1983, the number of installations has dropped significantly. The total amount of polystyrene used in 1984 and 1985

TABLE 3 ANNUAL USE OF EXPANDED POLYSTYRENE INSULATION IN NORTHWESTERN REGION

YEAR	BOARD-FEET OF INSULATION USED	YEAR	BOARD-FEET OF INSULATION USED
1966	31,500	1976	19,200
1967	19,000	1977	198,020
1969	620,700	1980	19,700
1970	193,900	1981	87,600
1971	400,000	1982	5,500
1972	282,000	1983	4,600
1973	133,500	1984	673,000
1974	276,300	1985	7,400
1975	30,300	1986	18,300

was high mainly because of certain projects on which large quantities were usually placed.

By the end of the 1986 construction season, the MTC will have installed in excess of 15,000,000 board ft of expanded polystyrene on Ontario highways.

SPECIAL APPLICATIONS

Highway 417: Ottawa

In 1976, an MTC contract was awarded for construction of a section of a four-lane rural highway approximately 10 km west of the city of Ottawa. Because of potential noise problems for a new subdivision development adjacent to the right-of-way, it was required that the grade be depressed some 3.0 to 4.0 m below the original ground line. The depressed grade crossed 600 m of a poorly drained area where the native soil consisted of a wet, sensitive Leda clay. Because of complications in transporting and disposing of approximately 153,000 m^3 of this wet material, it was decided to treat the clay with hydrated lime and use it to construct adjacent embankments *(10)*. Construction was completed in 1977.

During the winter of 1978, the Ottawa area experienced a brief period of unusually mild weather with rain. When temperatures returned to normal, the pavement over the embankments distorted dramatically, so much so that the outside driving lanes had to be closed to traffic. Subsequent investigations revealed that the addition of 2 percent lime had apparently changed the clay to a more friable silt type of material, which heaved when inundated with moisture.

Initial attempts to control the severe heaving included placing subdrains below the shoulder, cold-planing the most severely distorted areas, and waterproofing the shoulders.

Although the situation improved somewhat, further treatment was warranted. In 1984, it was decided to blanket the main lanes and paved shoulders with insulation. A total of 219,600 board ft of 25-mm expanded polystyrene was placed directly on the existing pavement and covered with 300 mm of granular base plus 110-mm asphalt concrete. A high-density insulation board having a compressive strength of 410 kPa was used in this case. The 1985 annual average daily traffic (AADT) for this section of highway was 18,000, with 19 percent made up of commercial vehicles.

Since the installation of the polystyrene, pavement performance during the spring has been satisfactory. Plans are underway to extend the treatment into other affected areas.

GO-ALRT TRANSIT LINE

In 1985, the MTC began construction of the rail bed for a light rail transit line intended to transport commuters between the city of Toronto and neighboring towns to the east.

The type of transit car under design was relatively new to the MTC, hence a plan was developed to provide a zero-maintenance design for the facility.

Construction of the subgrade involved excavation of several cuts, which consisted of soils containing a relatively high percentage of silt. It was decided to place expanded polystyrene insulation within the ballast layer to minimize heaving of the guideway during winter and to avoid unnecessary shutdowns of the system for maintenance purposes.

A total of 1,125,000 board ft of polystyrene was installed over the section between Pickering and Oshawa in 1985 (Figure 15). The 40-mm layer of insulation was located at approximately the midpoint of the 420-mm thick subballast layer.

Within the above section, a test area was established to evaluate the effectiveness of various thicknesses of expanded polystyrene used for this purpose. To date, initial monitoring data has not been processed; however, general observations have revealed very little difference in performance among the trials.

Lightweight Backfill to Structure

Settlement of embankments over unstable soils or at structure approaches is a serious concern to the highway engineer.

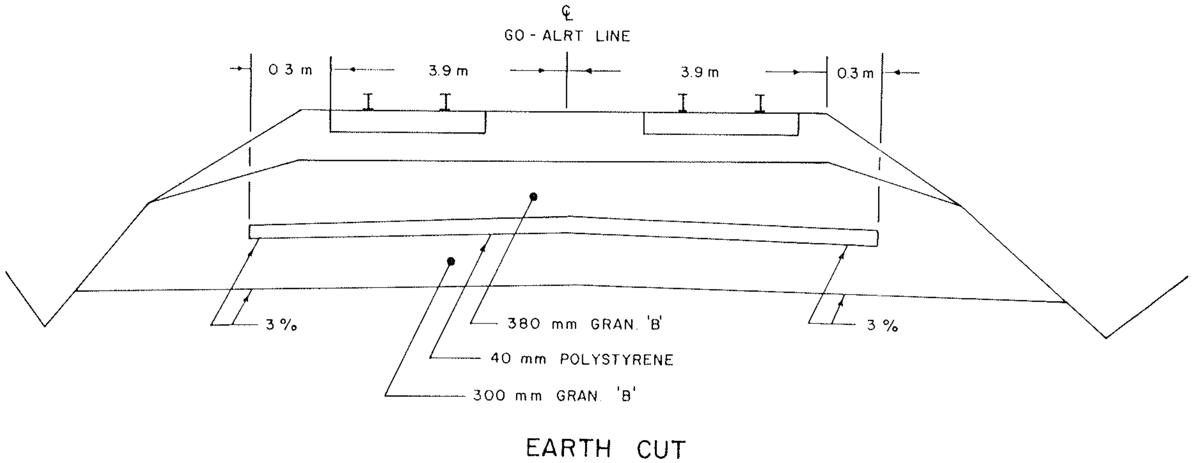


FIGURE 15 Insulation of rapid transit rail line.

Although attempts are made to maintain low profiles over swampy terrain, long-term settlements are often unavoidable, especially where organic deposits are many meters deep and cannot be economically excavated and replaced with suitable materials.

In recent years the use of various types of lightweight fill materials has become quite common. Expanded polystyrene has been promoted and used successfully for this purpose, primarily as an integral part of bridge approaches. Its light weight makes it easy to handle and engineering properties such as high strength and low moisture absorption are conducive to its use in this way.

Recently it was noted that the abutments at an MTC bridge on the Buskegau River in northern Ontario were slowly moving inwards. It was concluded that this movement was caused by pressure from the backfill to the structure. In 1984, a decision was made to remove a portion of the existing backfill at each approach and replace it with expanded polystyrene.

The insulation boards were tied together to form 2.4 m × 0.6 m × 0.3 m bundles and placed behind the abutments, as shown in Figure 16. In each layer the bundles were laid at right angles to those below. The polystyrene was then covered with two layers of 6-mm polyethylene sheeting, followed by granular material compacted in 225-mm lifts.

The structure will be closely monitored to see if the treatment described has been successful in stabilizing the abutments.

CONCLUSIONS

The heaving of pavements is an expected phenomenon in geographical regions that experience prolonged periods of freezing temperatures. It is therefore important during the design and construction of highway pavement structures to attempt to provide uniformity of materials in the roadbed.

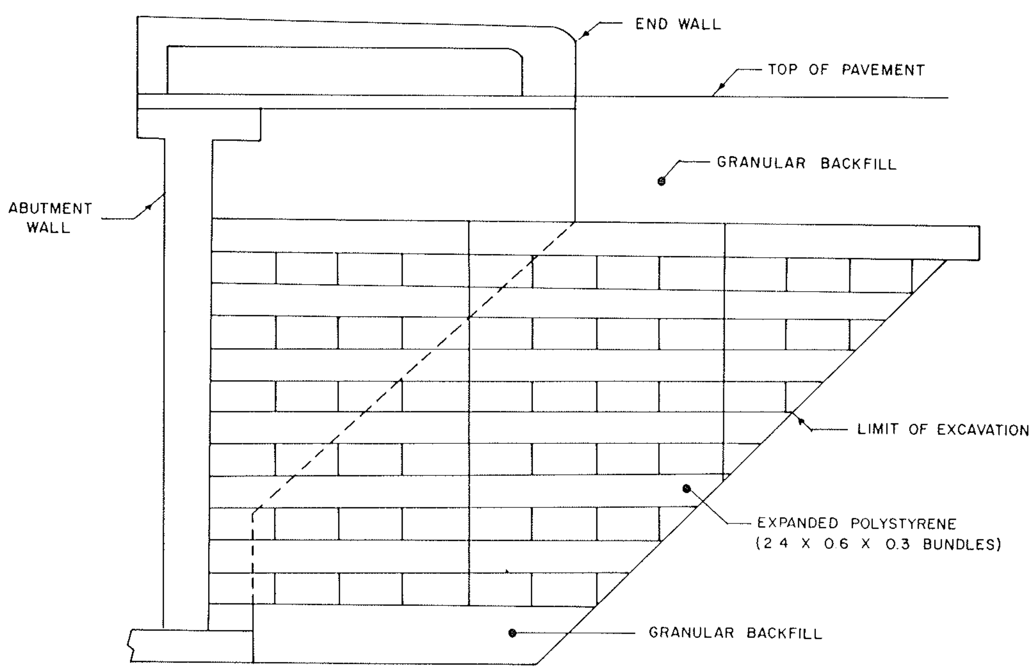


FIGURE 16 Use of expanded polystyrene as lightweight fill at structure.

Because of varying textures of subgrade soils and non-uniform moisture contents in these soils, differential frost heaving does occur. If this differential movement is severe enough, potential hazards can be created for the motorist and long-term performance of the pavement itself will be adversely affected.

The MTC has always depended on conventional treatments such as excavation and backfill, and subsurface drainage to abate the effects of frost heaving. These efforts have generally been successful; however, they are not always feasible in regions where frost can penetrate up to 3.0 m in depth.

Experiments carried out by the MTC have shown that expanded polystyrene insulation can be an effective tool in alleviating the problems created by differential heaving. Test results have indicated that 80 mm of polystyrene will effectively prevent frost penetration into the subgrade in areas where the freezing index can reach 1,800 degree-days C.

At the transitions between insulated and non-insulated pavements, staggering the ends of the polystyrene boards by 1.2 m has been shown to be effective in minimizing the formation of bumps caused by abrupt changes in the amount of heaving.

On reviewing its early experience with expanded polystyrene as an insulating agent, the MTC adopted a design philosophy of controlled frost heaving as opposed to the elimination of frost penetration. This approach provides a tolerable ride for the travelling public and at the same time minimizes the potential for pavement icing. To achieve this aim, MTC policy has been established to restrict the maximum thicknesses of polystyrene in installations designed to control frost heaves to 40 mm and 25 mm for northern and southern Ontario respectively.

Recognizing that icing of pavements placed over highway insulation is possible, the MTC has prohibited the use of expanded polystyrene in locations where traction and braking requirements are critical. These sites include horizontal curves, crests of hills, intersections, and railway crossings.

To date, extensive use of insulation boards has been the only major effort to prove successful in preventing damage due to heaving in pavements that are constructed over swamps having high water tables.

The provision of a comfortable and safe driving surface is the principal goal of any roadway agency. The Ontario MTC has adopted the use of expanded polystyrene as a significant tool in its efforts to achieve this goal over the past 20 years. It will continue to be an important feature of the province's construction and maintenance programs.

ACKNOWLEDGMENT

The authors wish to acknowledge the help of R. Hamilton and D. Greeley of Dow Chemical of Canada Ltd. in providing data used in this report. Appreciation is also extended to the staff of the MTC Regional Geotechnical Sections.

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Insulation Performance Beneath Roads and Airfields in Alaska

DAVID C. ESCH

In 1968, the Alaska Department of Highways constructed its first experimental installation of expanded plastic foam for frost heave control at a site 11 mi south of Anchorage. This was followed, in 1969, by construction of both the first insulated roadway section over permafrost in North America at a site near Chitina, and the first insulated airfield runway at Kotzebue. Since that time, six additional roadway sections on permafrost, totalling 3.6 lane-mi, have been insulated by the Alaska Department of Transportation, along with four additional airport installations. Applications of insulation for frost heave control have been numerous, totalling 11 lane-mi. Materials used for subgrade insulations have been primarily extruded-expanded polystyrene foam (Dow's Styrofoam HI and UCI Foamular) with one installation of polyurethane foam and three of molded polystyrene beadboard. Evaluations of the long-term thermal performance of these installations have included sampling and testing of the insulations to determine the retained thickness, thermal conductivity, and compressive strength properties. Based on these observations, foamed-in-place polyurethane insulation is not accepted for use as a subgrade insulation by the Department of Transportation, whereas extruded polystyrene insulation has demonstrated superior performance and longevity. Molded polystyrene beadboard insulation layers have given acceptable performance, but must be installed at a thickness 30 percent to 50 percent greater than the extruded polystyrenes to provide comparable thermal performance. Comparisons were made between measured late summer permafrost thaw depths for insulated airfields, and calculated thaw depths using the Modified Berggren calculation method and actual site soil and insulation properties. These comparisons demonstrated that this method of calculation results in calculated thaw depths slightly greater than the actual values, but provides reasonable values for a conservative design.

In 1967, the Alaska Department of Highways constructed its first experimental installation of expanded plastic foams for frost heave control at a site 11 mi south of Anchorage. This was followed, in 1969, by construction of both the first insulated roadway section over permafrost in North America at a site near Chitina, and the first insulated airfield runway at Kotzebue. Since that time, six additional roadway sections on permafrost have been insulated by the Alaska Department of Transportation, along with three airport runways and one taxiway. Applications of insulation for frost heave control have been numerous, particularly in the Anchorage-Wasilla area, where varying glacial deposits can result in severe differential frost heaving. Insulation layers are also frequently used beneath roadway crossings of buried water and sewer lines and subdrain systems. Materials used for subgrade insulations have been primarily extruded-expanded polystyrene foam (Styrofoam HI and UCI Foamular 400) with one installation of polyurethane foam and three of molded polystyrene beadboard.

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To evaluate the long-term thermal performance of these installations, air and gravel temperature monitoring instrumentation, consisting of thermocouples, thermistors, and chart recorders were installed at five sites. Temperatures have been monitored on a monthly basis for various periods at the different sites, the longest period being 16 yr at the Chitina permafrost site. In September of 1984, insulation samples were taken from selected road and airfield sites and tested for thermal conductivity and moisture absorption to analyze their long-term performance. At the permafrost insulation sites, soil moisture contents and thaw depths were also measured. Thaw depth calculations were then made by the Modified-Berggren calculation method to compare predicted versus actual thaw depths.

Roadway and airfield insulation sites constructed by the Alaska Department of Transportation and Public Facilities to control frost heaving or permafrost thaw settlements are listed in Tables 1 and 2 and locations are shown in Figure 1.

GENERAL SITE PERFORMANCE OBSERVATIONS

Frost Heave Sites

The first sites insulated in 1967 for frost heave control were monitored for 3 yr and their performance was documented *(1)*. The basic conclusions of this work were that the foamed-in-place polyurethane insulation used was not adequately resistant to water absorption, even when coated with asphalt, and it was not recommended for further roadway insulation applications. The extruded polystyrene, however, performed very well and was recommended for future frost heave control applications. For these instrumented sites, the Modified Berggren calculation method predicted freeze depths beneath the insulation with reasonable accuracy with a tendency to slightly overpredict these depths for thick insulation layers. This study also demonstrated that the full extent of the frost heave problem area must be known before designing the length of the insulation section to avoid creating heave bumps at the ends of the insulation. Based on data from these sites, a composite thermal design approach is recommended, with specified thicknesses of non-frost-susceptible gravels or sands placed above and below the insulation layer. By allowing freezing to penetrate into gravel fill placed beneath the insulation, the most economical design can be achieved. A depth of cover of at least 18 in. of gravel and pavement above the insulation is recommended to provide tolerable wheel-load stresses on the insulation. The problem of

TABLE 1 ANCHORAGE AREA AND PARKS HIGHWAY—FROST HEAVE CONTROL SITES

Year Built	Site or Route	Specific Location	Insulation Type	Thickness Inches	Cover Depth Inches	Length Lane Miles
1968	Seward Hwy	MP 114.9	FIP Urethane	2	18	0.036
1968	Seward Hwy	MP 115.1	Styrofoam HI	3	18	0.042
1970	Parks Hwy	MP 75.0	Styrofoam HI	4	18	0.055
1970	Parks Hwy	MP 84.2	Styrofoam HI	4	18	0.097
1971	New Seward	Tudor-East	Styrofoam HI	3	24	0.909
1971	Talketna Spur	MP 9.3	Styrofoam HI	3	18	0.170
1971	Parks Hwy	MP 117.0	Styrofoam HI	4	18	0.776
1975	Minnesota Drive	15 ST. Jct.	Styrofoam HI	4	27	0.076
1975	New Seward	Tudor-East	Styrofoam SM	3	24	1.310
1978	Parks Hwy	MP 36.7	Styrofoam HI	4	60	0.110
1981	Minnesota Dr.	At Dimond	Sty. Beadboard	4	24	0.760
1983	Northern Lights	Goose Lake	Styrofoam, HI	4	44	0.833
1984	Minnesota Ext.	At Dimond	Sty. Beadboard	4	36	2.340
1985	A-Street	@ 23 St.	Styrofoam HI	4	47	0.303
1985	A-Street	@ 15 St.	Styrofoam HI	4	47	0.530
1984	Parks Hwy	Panguingue	Styrofoam HI	4	24	0.038
1986	Parks Hwy	293.4-297	Foamular 400	2&4	42	2.650

TABLE 2 PERMAFROST INSULATION SITES

Year Built	Site or Route	Specific Location	Insulation Type	Thickness Inches	Cover Depth Inches	Length Lane Miles
1969	Chitina N.	MP 27	Styrofoam HI	2&4	60	0.070
1973	Parks Hwy	MP 231.8	Styrofoam HI	4	54	0.170
1974	Parks Hwy	Alder Ck. S.	Styrofoam HI	4	54	0.125
1974	Parks Hwy	Alder Ck. N.	Styrofoam HI	4	120	0.072
1979	Farmer's Loop	Fairhill Rd.	Poly. Beadboard	4	48	0.077
1985	Canyon Creek	Rich. MP 299	Styrofoam HI	3	33	1.061
1986	Edgerton Hwy	MP 1.3-7.0	Styrofoam HI	2	42	2.080
	Insulated Airfield Runways on Permafrost					
1969	Kotzebue Airport		Styrofoam HI-35	4	42	1900'
1981	Buckland Airport		Styrofoam HI-35	6	36	2400'
1981	Deering Airport		Styrofoam HI-35	2	30	2700'
1985	Nunapitchuk Airport		Styrofoam HI-60	6	30	2750'
1985	Deadhorse Taxiway B		Styrofoam HI-60	2	56	360'

differential frost formation on the road surface in insulated areas is lessened by increased depths of burial.

As shown in Table 1, the depth of cover over frost heave insulation has varied from 1½ to 5 ft. Surface frost forms more quickly on bridge decks and above insulated areas than on normal road structures. The resulting decreases in traction have recently (in 1985) been treated by installing rubber-modified asphalt pavement surfacing above the insulated areas at the A-Street, New Seward at Tudor, and Canyon Creek sites. The benefits of this pavement type have been investigated by Esch *(2)*. If this treatment proves successful in eliminating icing problems, it may permit cover depths above the insulation to be reduced to the 18-in. minimum required by stress considerations.

Permafrost Sites

The Chitina insulated road site, constructed in 1969 over relatively warm (+30°F) permafrost, has generally performed well, with full annual refreezing of the soils beneath the roadway and a long-term subroadway permafrost temperature of 31.0°F to 31.5°F. However, the gravel side slopes of the embankment, which are insulated by snow in winter and exposed to direct sunlight in summer, have caused progressively deeper annual thawing, averaging about 3 in./yr into the permafrost beneath the slopes. This has resulted in progressive slope sloughing and cracking in the shoulder areas *(3)*. By comparison, the adjacent uninsulated roadway areas have continued to settle annually at a rate of 0.1 to 0.2 ft/yr. Side slope thawing movements have been similar in insulated and uninsulated areas, and some settlements related to stress-related creep of the warm permafrost foundation soils continue to be noted in all areas of this roadway.

The performance of most other insulated permafrost roadway sites has been similar to that at Chitina, with full annual refreezing occurring beneath the insulated traveled roadway area, but with progressively deeper annual thawing and related movements of the side slope areas.

Cover depths used over permafrost insulated layers tend to be greater than for frost heave control, primarily because of construction traffic considerations. Permafrost insulation has generally been designed based on borings and installed during embankment construction. The heavy construction equipment used requires care to avoid crushing. By contrast, frost heave insulation sections have nearly always been designed as corrective measures for heaves on existing roads, where minimizing the depth of excavation is of major concern.

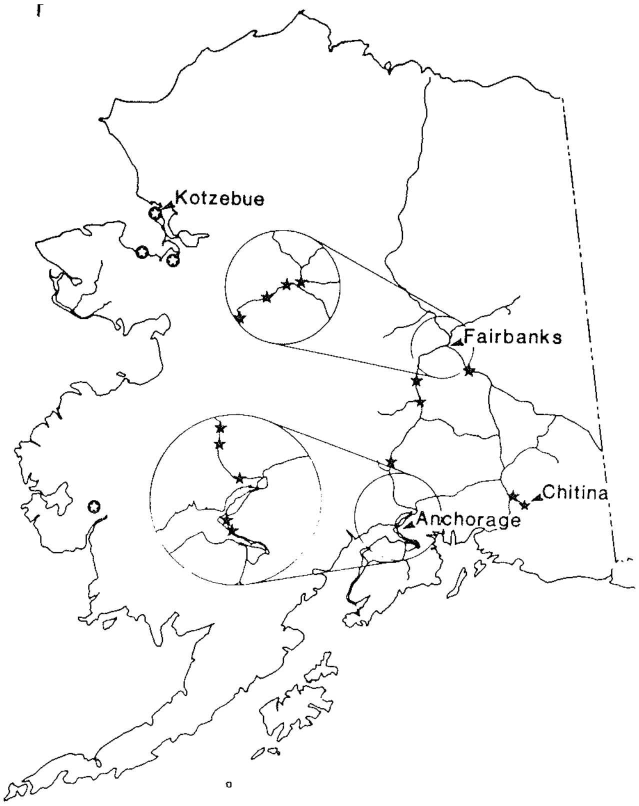

FIGURE 1 Location of insulated roadways (✪) and airfields (✯) constructed by Alaska Department of Highways and Department of Transportation.

FIELD SAMPLING AND INSULATION TESTING PROGRAMS

Insulation sampling and testing to measure moisture absorption and compression under field conditions has been done at various times and locations to verify the field performance of the buried insulation layers.

Polyurethane Foams

The most extensive testing of polyurethane road insulation samples was in September 1972 at an Arco Chemical road test site near Prudhoe Bay. At that location, urethane foam board samples taken from beneath a permafrost area roadway, after 2 yr of exposure on a wet tundra foundation, demonstrated moisture contents as high as 30 percent by volume or 1000 percent by weight of the dry foam. Unfortunately, data from this polyurethane field test and sampling program were never published because of the unexpectedly poor insulation performance. This investigation was followed in March of 1979 by a foamed-in-place urethane insulation sampling program at the Potter frost heave insulation test site located near Anchorage *(1)*.

At Potter, several samples were obtained from the wheelpath areas after 12 yr of insulation exposure to traffic and the environment. In addition, two samples were obtained from an extension of the urethane insulated area, placed in 1974. Six representative samples of the 1967 insulation were tested for absorbed moisture and had total moisture contents, calculated as percent water by volume, ranging from 3.1 to 72.0 percent.

Thickness measurements were made in many locations that were believed to be within the original (1968) 2-in. insulation area, and an average thickness of approximately 0.9 in. was noted. No foam thicker than 1.2 in. could be found. Exact thickness comparisons are not possible because of the somewhat random thickness obtained from in-place foaming of urethanes, but measurements at the time of placement indicated an average thickness very close to 2 in. Many areas of thickness as low as ½ in. were noted and foam in these areas was generally nearly saturated. This foamed-in-place urethane had an average compressive strength of 31.5 psi when placed in 1967. All data indicate that this foam insulation failed by moisture absorption and compression under field service.

By contrast to the generally compressed and relatively wet state of the 1968 urethane foam, the samples from the 1974 extension appeared to be in excellent condition, maintaining some hope that polyurethane foams may perform reasonably well under direct soil burial conditions. Two samples of this insulation had dry densities of 2.0 and 2.1 pcf, absorbed water contents of 1.3 and 1.0 percent by volume, and average compressive strengths of 28.1 and 26.3 psi at 5 percent strain. The strength and moisture properties of this product appeared satisfactory for acceptable performance in direct burial. Unfortunately, this entire test site was excavated in 1979 so the long-term performance of this material could not be followed. The reasons for the good performance of this foam compared with the failure of the two urethanes previously mentioned could not be determined.

Polystyrene Foams

Field samples of in-service polystyrene subgrade insulations were taken in September of 1984 from various road and airfield sites in Alaska. Samples were typically taken from hand-dug test pits located at the edge of the asphalt pavement. All samples were sealed in Ziplock bags and subsequently tested for moisture absorption and wet thermal conductivity, using the thermistor bead technique detailed by Atkins *(4)*.

In this method a single thermistor bead, approximately .04 in. in diameter, is inserted into the insulation board and a controlled electrical current is applied to cause resistance heating of the thermistor. Periodic readings are taken of temperature rise versus time, from which the thermal conductivity can be calculated. Multiple readings at various depths within a foam sample are used to obtain a profile of conductivity versus depth, from which an average value is calculated. Results are consistent and agree well with the more precise laboratory "guarded hot plate" method.

Two of these sites, the Fairbanks area Fairhill Access Road, constructed in 1979 *(5)* and Anchorage's Minnesota Extension—Phase I, constructed in 1981, contain white polystyrene beadboard, with a compressive strength greater than 30 psi. This product is molded in blocks from pre-expanded foam beads. The remaining sites were all insulated with blue Styrofoam HI insulation, which is foamed in an extrusion process. Results of all testing are included in Table 3 and averages for each site are shown in Figure 2.

The best-fit trend lines (Figure 2) showing the relationships between moisture content and thermal conductivity for both extruded and molded foam boards were found to be in excellent

TABLE 3 POLYSTYRENE INSULATION TEST RESULTS

Site	Layer	Thickness	Foam Density (pcf)	Insulation Type	% Water/Vol	Kavg.	Year Placed
Kotzebue	Top	2"	2.39	E	2.38	0.214	1969
Kotzebue	Bottom	2"	2.14	E	0.89	0.194	1969
Buckland	Top	3"	2.26	E	0.41	0.204	1981
Buckland	Bottom	3"	2.10	E	0.23	0.208	1981
Deering	Single	2"	2.16	E	1.37	0.204	1981
Chitina	Top	2"	2.56	E	0.71	0.237	1969
Chitina	Bottom	2"	2.63	E	0.88	0.225	1969
Chitina	Single	2"	2.63	E	1.54	0.216	1969
Bonanza Creek	Single	2"	2.24	E	1.48	0.247	1974
Bonanza Creek	Single	2"	2.83	E	2.38	0.248	1974
Fairhill	Single	4"	2.44	BB	1.18	0.278	1979
Fairhill	Top	2"	2.17	E	0.50	0.222	1979
Fairhill	Bottom	2"	2.27	E	0.20	0.213	1979
Fairhill	Single	4"	2.98	BB	1.48	0.289	1979
Minnesota Dr.	Top	2"	2.46	BB	5.88	0.358	1981
Minnesota Dr.	Bottom	2"	2.71	BB	2.90	0.266	1981
Geneva Woods	Single	3"	---	E	0.64	---	1970
Geneva Woods	Single	3"	---	E	0.53	---	1970

Type E = Extruded Expanded Foam
Type BB = Molded and Cut Beadboard Foam

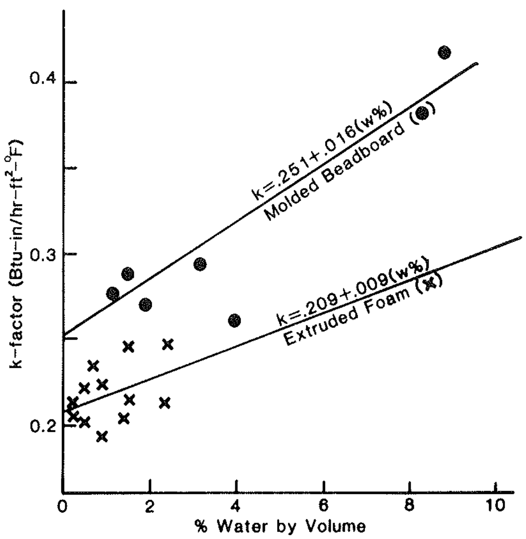

FIGURE 2 Moisture contents and thermal conductivities at 72°F of polystyrene insulation samples from roads and airfields in Alaska.

agreement with other data from laboratory tests at higher moisture contents *(6–9)*. All samples were within 5 percent of the nominal thickness, indicating that compression and creep with time is not a problem.

The extruded styrofoam insulation board, sampled from six sites with a maximum age of 15 yr and averaging 9.5 yr of service, had an average moisture content of 1.16 percent by volume. Thermal conductivities (K) of these samples average 0.22 Btu-in./hr-ft^2-°F at 72°F. Maximums were a moisture of 2.38 percent and a K of 0.248. By comparison, the molded beadboard samples, from sites 3 and 5 yr old, had an average moisture of 2.9 percent and an average K-factor of 0.30. Maximum values for the beadboard were 8.7 percent moisture and a K of 0.42. When analyzed by a standard statistical approach, the long-term minimum R values/in. expected at the 95 percent confidence level are 3.9 for extruded polystyrene foam and 2.6 for high-strength molded polystyrene beadboard. This results in a thickness ratio for equivalent performance of 1.5 in. of molded to 1.0 in. of extruded foam. If this ratio is based on average rather than minimum R values, a thickness ratio of 1.36 to 1 is indicated; however, a ratio this low would be unfair to extruded foams, which are more consistent and better able to resist moisture gains and R-value losses with time in service.

THAW DEPTHS: INSULATED AIRFIELDS ON PERMAFROST

Observations of the thaw depths were made between September 4 and 6, 1984, at the three insulated airfields investigated in this study, and soil moisture contents were measured at intervals to permit comparisons between thaw depth prediction methods and actual field values.

Thaw depth calculations were made for each of these runways using the actual measured soil and insulation properties, the Modified-Berggren calculation method as programmed by Braley *(10)*, the recorded Kotzebue air thawing index of +1480°F-days, and surface n-factors of 1.70 for Kotzebue (paved) and 1.30 for the other (unpaved) sites *(11)*. These calculations overpredicted the thaw depths by 0.2 to 0.5 ft, as shown in Table 4, indicating that this calculation method is conservative for designing insulation layers on cold ($T > 30$°F) permafrost.

SUMMARY

Since 1967, the Alaska Department of Transportation has insulated a total of 14.7 lane-mi of roadways and 9,750 ft of airfield runway to control frost heaving and permafrost thawing. Insulation materials used have included foamed-in-place polyurethanes, molded polystyrene beadboard, and extruded-expanded polystyrene foam.

Polyurethane foams have varied greatly in field performance, with high moisture absorption and compression failure

TABLE 4 MEASURED AND PREDICTED TOTAL THAW DEPTHS FOR 1984 FOR INSULATED AIRFIELDS ON PERMAFROST (based on modified Berggren calculation method using measured soil moistures and insulation thermal properties)

Site	Thawing Index (F-Days)	Surface N-Factor	Insulation Depth	Measured Thaw Depth (ft)	Predicted Thaw Depth (ft)
Kotzebue	1480	1.70	3.5'	4.7'	4.9'
Buckland	1480	1.30	3.3'	3.8'	4.3'
Deering	1480	1.30	2.3'	2.8'	3.2'

noted at two of three sample locations. For this reason polyurethane foams are not presently accepted by the Alaska Department of Transportation for use beneath roads or airfields.

Based on observations from field sampling of the insulations after various exposure periods, the superiority of extruded-expanded polystyrene foam is evident after as much as 15 yr of service. Molded polystyrene beadboard products, evaluated after 3 to 5 yr of service, were somewhat less resistant to moisture absorption in the subroadway environment. To provide equivalent long-term thermal performance under soil burial conditions, beadboard insulation thicknesses should be 30 to 50 percent greater than extruded foam thicknesses. When installed beneath roads and airfields, the functional design life of extruded foams is projected to be much greater than 20 yr.

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The Use of Cellular Plastic in Swedish Railways To Insulate the Track Against Frost

ERIK SANDEGREN

Presented in this paper is a description of current practice in the use of cellular plastic (polystyrene) for insulation against frost in railways. In the first part of the report the history of the use of insulation against frost in railways is briefly described. In the second part, the properties of the best material currently available—extruded cellular plastic of polystyrene (Styrofoam, Styrodur, Ecoprim)—are described. A material intended for insulation against frost that is laid in the soil under the permanent way in a railroad must fulfill the three following requirements: (a) it must have adequate compressive strength, (b) it must not take up moisture to such an extent that the insulating properties deteriorate unacceptably, and (c) it must not be affected by time (e.g., bacteriologically or chemically) in the soil. Stress-strain curves are also detailed, showing the highest limits allowed for deformation and the lowest breaking force allowed for material in a railway track. Dynamic tests and tests with different petroleum products are also described. Part three deals with the application of the material to the track. Finally, the value of extruded cellular plastic as insulation against frost damage with different applications is reviewed. In this respect it is noted that only prescribed qualities are used and that the materials are continuously controlled at delivery.

The problem of the need for insulation in buildings and on traffic routes is an old one in Sweden. Even in the eighteenth century damage was observed on culverts and bridge abutments on the stagecoach roads. It is reported that brush mats were used in northern Sweden in the 1870s as protection against frost lift. At the end of the 19th century peat was also used as a protection fill under the track, principally in northern Sweden. Today these fills have been damaged by the increased weight of trains and drains. In 1910, sleepers of wood were put on the track on the Karungi-Övertorneå line. In 1965 (more than 50 years later) the insulation was inspected. The wood in the sleepers was still in good condition. The weight of the trains gradually increased, further reducing trafficability.

In the 1920s some countries with cold winters began to investigate the problem of frost action in soil. In Sweden the leader of this area of study was Gunnar Beskow. In 1935 he published a summary of his findings *(1)*, which is now well-known throughout the world. Based on these theories, practical solutions to the problems of frost action were developed. The methods used by the Swedish State Railways (SJ) are described by Sandegren 1972 *(2)*.

Because it is costly to insulate to full frost penetration depth, the problem was to find a highly effective insulating material with such a high strength that it could be laid immediately under the ballast. In the early 1960s, G.A. Leonards was the first to use extruded cellular plastic. In Norway, S. Skaven-Haug employed this method in 1964, but used a bead plastic. In 1968, SJ laid the first test section (200 m) also using a molded bead plastic.

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PROPERTIES OF CELLULAR PLASTIC

SJ has tested both molded and extruded cellular plastic. A cellular plastic used to insulate against frost action in the soil must fulfill the following requirements:

1. Have the necessary strength.
2. Be so nonabsorbent that its insulating ability does not decrease significantly with time.
3. Be so resistant (e.g., to bacteria or chemicals) that it is not affected in the soil.

The second point is difficult to fulfill for molded bead plastic. Nowadays only extruded cellular plastic is used with bead-board being used previously. The material must also carry the load of the traffic, including the dynamic contribution within the elastic stage.

The International Union of Railways (UIC) lists specifications for the insulating material in its Code 719 R (see Figure 1) *(3)* as follows:

Axle load (kN)	<200	200–220	>220
Dry density (kg/m)	30 to 35	35 to 40	40 to 50
Deformation limit (%)			
0 kPa	<1	<1	<1
350 kPa	<5	–	–
400 kPa	–	<5	–
450 kPa	–	–	<5
Breaking force (kPa)	>250	>350	>450

These specifications must be used when the insulation slabs are 0.3 m below the underside of the sleeper and of 4 m width. They can be relaxed when the slabs are at a greater depth.

The insulation slabs must be placed at least 0.3 m below the sleepers to be protected during maintenance work. Figure 1 shows these given limits and also the curves for material normally used by SJ. The key to data in Figure 1 is as follows:

1 = Styrofoam HD 300 (according to a Dow Chemical letter);

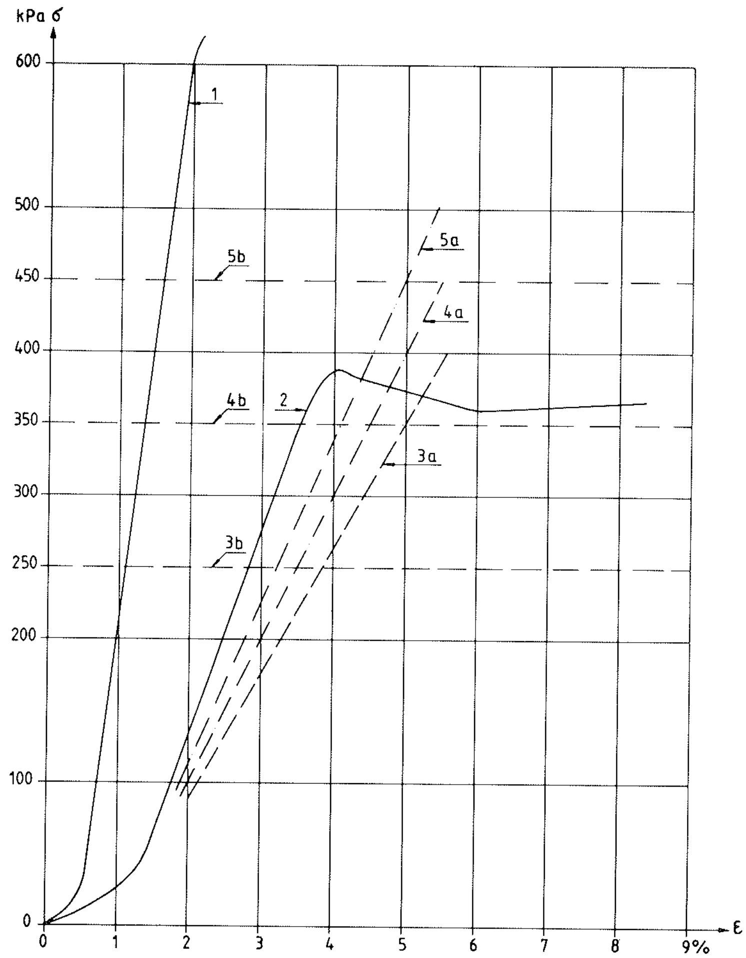

FIGURE 1 Specifications according to the UIC Code 719 R and stress-strain diagrams for different cellular plastics used by Swedish State Railways.

2 = Styrofoam HI 50 (mean value, according to a Dow Chemical letter);
3a, 4a, 5a = deformation limits according to UIC Code 719 R by axle load *3a* = < 200 kN, *4a* = 200 to 220 kN, and *5a* = > 220 kN; and
3b, 4b, 5b = the lowest-allowed breaking force by the same axle loads.

The dynamic behavior of the materials is of great interest. Many dynamic endurance tests in a repeated dynamic-stress testing machine were performed. First of all, testing between the load 30 kPa and the different highest loads was undertaken until the material could endure $2 \cdot 10^6$ repeated impacts. Today the material is tested between 10 and 150 kPa during $2 \cdot 10^6$ repeated impacts. The allowable deformation must then be < 5 percent. Three different materials are used for normal railroad lines: Ecoprim 937-00 from Rockwool, Styrodur 4000 S from BASF, and Styrofoam HI 50 R from Dow Chemical. They all have a deformation < 5 percent by a load of 350 kPa, a density > 38 kg/m and a thermal conductivity of about 0.033 W/mK. For lines with more than 220 kN axle load, the slabs are either laid deeper or Styrodur 5000 S or Styrofoam HD 300 E are used, which can also be used under highly loaded bridge abutments. The resistance of cellular plastic to petroleum products is in some cases low (e.g., petrol rapidly destroys even a high-class extruded cellular plastic). Diesel fuel also has the same effect but it does not happen as fast.

To investigate how spills from tank cars might affect the slabs in a track, petrol, diesel fuel, and heavier oils such as lubricating oils and gear oils have been tested both on ballast over the slabs and directly on the slabs. The tests show that spills on the ballast do not harm the slabs but a direct outflow in a track (e.g., by a derailment) will destroy the slabs. The probability for a derailment on an insulated part of the line is, however, very low. A more complete report on the philosophy of the use of high-insulation materials and performed tests is given by Sandegren 1978 *(4)*.

THE USE OF CELLULAR PLASTIC ON THE TRACK AS INSULATION AGAINST FROST

When cellular plastic was first used as insulation against frost, the choice of dimensions was rather limited and the slabs had no shiplaps. The length was limited to 2.0 m and the breadth was 0.6 m. As the width of the insulating area under the ballast should be at least 4.0 m, the slabs could be laid parallel with or perpendicular to the track. As the joints do not coincide, the slabs must be staggered in two layers. If the slabs are laid parallel to the track, the total width is 4.2 m (7 × 0.6 m). However, the practice showed that it was better to lay the slabs perpendicular to the track. Therefore, a layer of two slabs with 2.0-m lengths were first laid out and on this a layer, with one slab 2.0 m long in the middle and two slabs 1.0 m long outside, this slab was placed. However, after some time it was possible to get slabs with a length of 2.5 m. One slab with 2.5 m and one with 1.5 m length were then laid in the first layer and in the second layer one with 1.5 and one with 2.5 m length were added.

Later slabs 4.0 m long were developed. The last step to make it possible to lay the insulation in one layer was to introduce shiplaps. The effective width of the slabs thereby diminished to 0.54 m (see Figure 2). Thus the insulation was placed in one

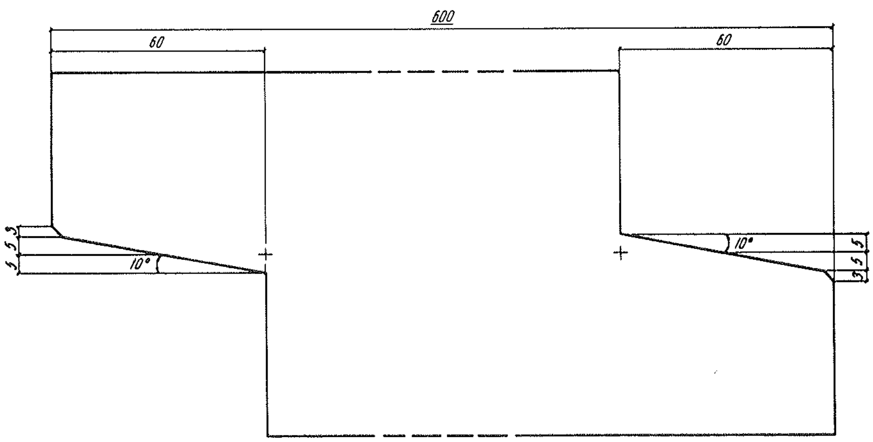

FIGURE 2 Shiplap for insulating slabs in track.

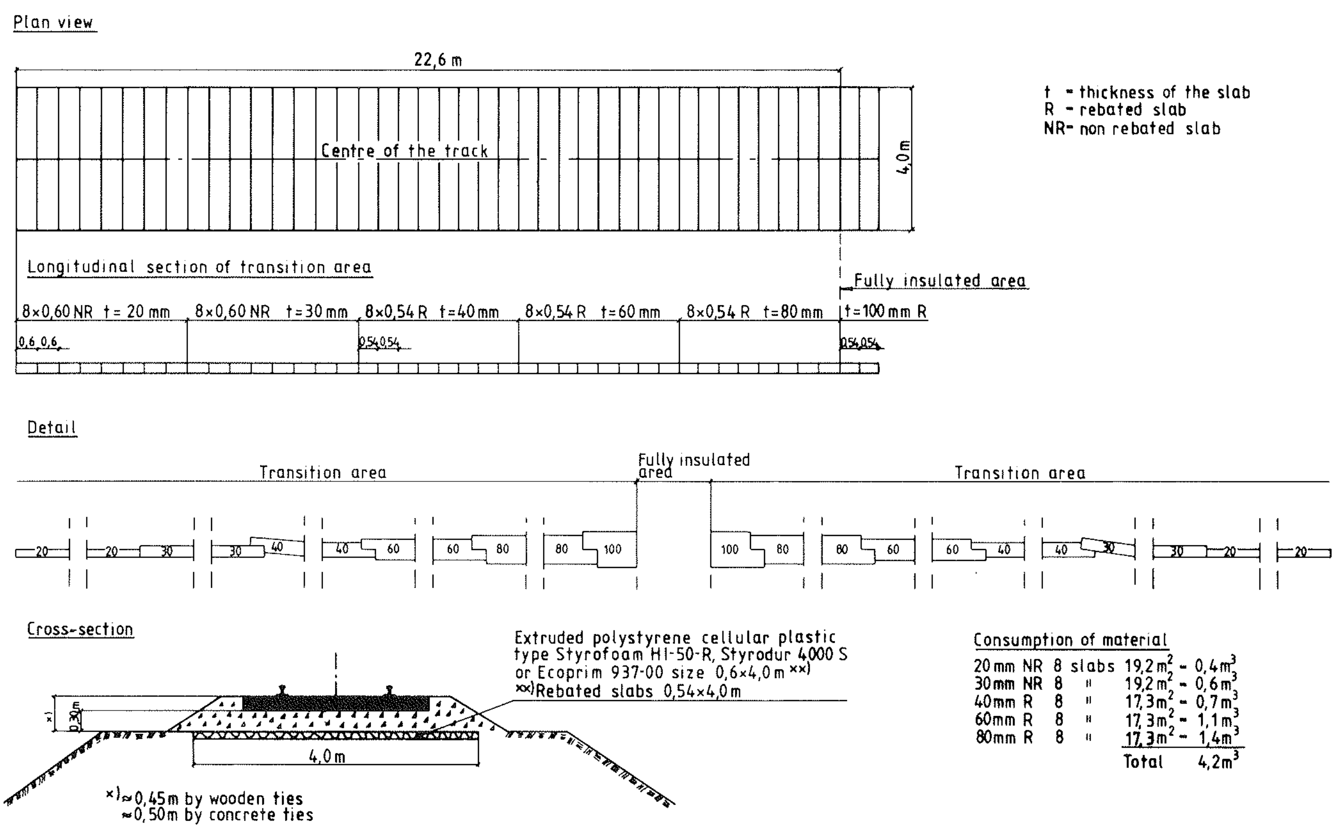

FIGURE 3 A transition area for 100-mm-thick slabs with shiplaps.

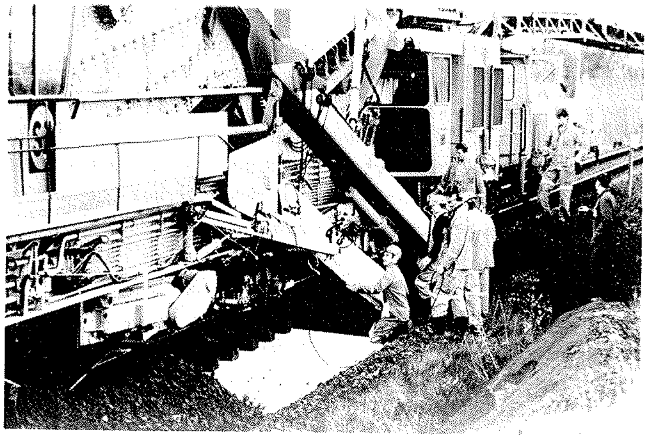

FIGURE 4 The ballast-clearing machine lifting the track and digging out the ballast to make it possible to put in the insulating slabs.

layer with 4.0-m long slabs perpendicular to the track. The thickness depends on the amount of cold (–°Ch) in the winter at the actual place and ranges from 60 mm in southern to 140 mm in northern Sweden. See also work by Sandegren *(4)*. Great care must be taken with the transition areas at the ends of a stretch of insulated track because the material is so highly insulating. The lengths required range from 16 m in southern to 38 m in northern Sweden. An example of a transition is shown in Figure 3.

From 1972 to 1974 all slabs were laid by hand. The rails were either taken away one bearing at a time or the sleepers only were removed. The old ballast was then removed with an excavator. It was time-consuming work. In March 1974, a ballast-clearing machine [hired from the Norwegian State Railways (NSB), a Plasser and Theurer machine type RM 82] was used for the first time (see Figure 4). The machine first removed the old ballast and, during cleaning, the slabs were put

TABLE 1 RANGES OF APPLICATION FOR HIGH-INSULATION MATERIALS FOR INSULATION AGAINST FROST

Material / Range of application	Extruded cellular plastic compressive strength kPa			Molded cellular plastic (bead-plastic) compressive strength kPa		Rockwool	
	≥450	≥350	≥250	≥350	≥250	821	817
Railways with axle load >220 kN	X						
Railways with axle load 200 to 220 kN		X					
Railways with axle load <200 kN			X				
Under foundations for buildings and bridge abutments							
Load >200 kPa	X						
" 200-100 "		X					
" <100 "			X				
Outside and along buildings							
dry condition			X	X	X	X	X
wet "			X	X			

in under the track by hand and the cleaned ballast was replaced on the slabs. After the ballast was replaced and tamped down, the track was fit for traffic again. The outcome of the test was so good that new machines were not only rented but purchased for the improvement work. However, it was still time-consuming work to transport the slabs to the track, lay them out, and then put them in by hand. Thus the idea was conceived of modifying the machine so that the slabs could be automatically transported from the freight wagons to the machine and down under the track.

In April 1984, the prototype, RM 80-SPX, was ready to work. The machine did all the work except putting in the slabs under the track. The new machine worked successfully and the work in southern Sweden ended in November. In 1985 the crew was so experienced that it could insulate about 900 m during a shift of 8 hr at its best.

During the winter of 1985 to 1986, the machine was again modified slightly to make it possible to lay the slabs under the track automatically. In 1986 this modification was tested. During the first months there were some problems but later it worked satisfactorily (see Figure 4).

CONCLUSIONS

After working with high-insulation materials for nearly 20 yr, it is concluded that extruded cellular plastic is the best material and that it can help to solve different types of frost problems on the railroads. However, in special cases, other highly insulating materials can also be used. Table 1 shows present recommendations.

Molded material (bead plastic) cannot be used in soil under a bearing construction (abutment or foundation), as chance of the possibility of augmented moisture ratio accrues because the material in this position cannot breathe.

To insulate with high-insulation materials is both an economical as well as a technically sound solution if the right material is chosen for the right purpose. However, it is a required condition that if only prescribed kinds of material are used, these kinds are continuously controlled at delivery.

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Developments in the British Approach to Prevention of Frost Heave in Pavements

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In Britain, frost heave (and also thaw weakening) is prevented by ensuring that all materials within 450 mm of the road surface are non-frost susceptible as defined by the Transport and Road Research Laboratory test, the latest version of which is known as the SR829 test. Materials are not frost susceptible if their mean heave in valid SR829 tests is not greater than 12 mm. For materials close to the borderline, tests are required at three different laboratories to define the mean heave. The test, which now has improved reproducibility, is to be included in British Standard Specification 812. In this paper the evolution of the test is summarized and the implications of the revisions and of other factors in design, such as depth of frost penetration and the availability of water, are examined critically. Results of a series of tests are presented that show that the British frost-susceptible materials correlate with the "highly frost-susceptible" classification in the United States and French systems. Particle size distribution and saturation moisture content are shown to be poor indicators of the frost susceptibility of aggregates, because neither can reflect the contribution of between-particle and within-particle pore size distributions. Suction characteristics, determined by an osmotic technique, overcome this shortcoming and correlate well with the frost susceptibility of the materials tested.

For almost 20 years British specifications have required that all material within 450 mm of the road surface be non-frost susceptible on the basis of the Transport and Road Research Laboratory (TRRL) test. Both the test and the limits have evolved during that period; the latest version being SR829 *(1)*. This approach, which automatically controls thaw weakening, implies that

1. Frost-susceptible materials can be identified by the TRRL test and hence both reproducibility of the test and the classification applied to the results are adequate.
2. There is a strong probability that frost will penetrate to a depth of 450 mm during the design life of the road.
3. A supply of water to feed the freezing frost is readily available.

These implications are discussed in the light of recent developments. The correlation between the British classification system and those used in both the United States and France is then examined on the basis of experimental results. Finally, the physical properties of aggregates that render them frost susceptible are considered and guidance is given on the best way of using borderline materials.

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THE TRRL FROST-SUSCEPTIBILITY TEST

The test has its origins in early work in the United States *(2, 3)*. By the 1950s procedure was standardized within the then-titled Road Research Laboratory (RRL) *(4)* and was essentially that described in LR90 *(5)*.

The test's introduction into compliance testing began with the publication of the 1965 edition of Road Note 29 *(6)* and was completed with the issue of the fourth edition of the *Specification for Road and Bridgeworks* in 1969 *(7)*. Following difficulties with poor reproducibility, both the test and its interpretation have been revised in the last 15 years. TRRL reports SR318 *(8)*, MM64 *(9)*, and SR829 *(1)*, together with advice and instructions issued by the Department of Transport (DTp), the Scottish Development Department (SDD), and the Welsh Office, and also new editions of the *Specification for Road and Bridgeworks* issued in 1975 and 1986, detail these changes.

The SR829 Test in Relation to Its Predecessors

The SR829 test is undertaken on 102-mm dia × 152-mm high specimens compacted with a vibrating hammer to the maximum dry density at optimum moisture content. Particles larger than 37.5 mm are removed before compaction.

Nine such specimens, encircled in waxed paper, are placed on porous stones in contact with water (at + 4° ± 0.5°C) in a self-refrigerated unit (SRU) in which the water level is maintained automatically *(10)*.

The space between the specimens is filled with loose dry sand so that the specimens are frozen vertically downwards for 4 days when the air temperature above them is lowered to −17° ± 1°C. Before freezing, the specimens are allowed to temper (that is, imbibe water) for 5 days. During freezing, the air, water, and specimen boundary temperatures are measured routinely at selected points and the heave of individual specimens is measured daily.

Normally, specimens are tested in sets of three. For an initial test to be valid, the temperature should be within specified limits; the mean heave should not be suspiciously low (<2 mm); and, if the mean heave is less than 18 mm, the range of individual results should not be more than 5 mm. If a wide spread is obtained, the test is repeated using six specimens. Materials are immediately classified if the mean heave, in a valid test, is either less than 9 mm or more than 15 mm as non-frost susceptible and frost susceptible, respectively. If the mean heave is between 9.1 and 14.9 mm, further tests are required at two other laboratories. The material is then classified as non-frost susceptible if the grand average heave is less than 12 mm.

The evolution of the test from LR90 to SR829 is summarized in Table 1. Significant developments include

- The introduction of SRUs,
- The change to vibratory compaction,
- Shortening the freezing period (from 250 to 96 hr),
- Lengthening the tempering period (from 24 to 115 hr), and
- The possible involvement of several laboratories to classify a single material.

The reduction in the freezing period has been suggested by many previous investigators. The increase in tempering time was introduced following indications, from a series of tests on sand/limestone filler specimens, that it would result in improved repeatability *(11)*. Subsequently, a few tests on real aggregates indicated a marginal improvement in repeatability that was accompanied by a slightly increased heave. An unwelcome side effect of this change is the loss of an early warning of probable noncompliance that was particularly useful to contractors during the tendering period. It is too early to evaluate the effects of the possible need to test at three laboratories before classification, but concern has been expressed about both the time and costs involved. Currently there are some three dozen SRUs in Britain, although not all are in independent test houses.

As far as the test is concerned, there has been a marked improvement in reproducibility, which is quantified as the value below which the absolute difference between two single test results obtained by the same method, in different laboratories with different operators, may be expected to be within a probability of 95 percent. A study on the same batches of materials indicated a reproducibility value of 6 at a mean heave of 12 mm *(1)*. This estimate was based on trials with both sand/limestone filler mixes and a hoggin (gravel-sand-fines). A later trial with a flint gravel *(12)* gave a reproducibility of 9 on a mean heave of 13.5 mm. These trials have attempted to measure the precision of the test and do not include sampling error or variations in target density and moisture content. However, the inherent variability of specimens [which may be high, particularly for a flint gravel *(10)*], is included. Further work is being considered to obtain a more reliable estimate of the precision of the test over a greater range of materials.

The new procedure is intended to be applied to a source rather than a routine control. The importance of proper sampling cannot be overemphasized. In particular, sampling and testing of recovered material from compacted subbases is at best problematical *(13)*.

Background on the British Classification Limits

The upper limit of 13 mm (after 250 hr of freezing) for non-frost susceptibility (given in LR90) was based on a limited amount of field evidence, some of which was obtained from subgrades rather than subbases, dating back to the 1940s *(14)*. Because the subbase effectively replaced subgrade when thicker pavements were introduced, the same limits were thought appropriate *(5)*. Croney *(4)* summarized a few more recent failures including the Preston By-Pass in 1958 and the 1962/1963 incidents at Maidstone By-Pass (M20) and the Ross Spur (M50). A common feature was the use of frost-susceptible material with little more than 300 mm cover.

Preston is particularly interesting because the wet-mix limestone base, having a 250-hr heave of up to 15 mm and a cover of only 85 mm of surfacing, did not appear to heave significantly. The problem was with the compacted colliery shale subbase (which had a 250-hr heave of up to 65 mm). Subsequently, the 250-hr limit of 13 mm was maintained in England and Wales. However, after 1972, for well drained roads with impermeable surfaces, a 250-hr limit of 18 mm was applied in Scotland, where all the testing was undertaken in a single laboratory.

For 96-hr freezing tests, the limit set at 10 mm for the MM64 procedure was increased to 12 mm in SR829. For tests with 1-day tempering, the direct equivalence was probably nearer 10 mm. However, the effects of increased tempering time, backed by engineering judgment based on the known performance of, say, carboniferous limestone over many years, suggest that the limit of 12 mm for the SR829 procedure is appropriate. These developments should obviate the need for special consideration for Scotland.

DEPTH OF FROST PENETRATION

The general use of 450 mm of frost penetration for design has been reexamined recently *(15)*.

The mean annual freezing index was obtained for nearly 50 meteorological stations throughout Britain for the period 1959 to 1979 (Figure 1). The maximum freezing index for a single winter was 302 (for 1962/1963, the coldest winter for nearly 200 yr). The grand mean was 50°C days, which corresponds to a frost penetration somewhat in excess of 250 mm. For three sites a freezing index exceeding 50 was not recorded in the study period, and for a further five this figure was only exceeded in 1962/1963. It would appear that the appropriate design depth for frost penetration should vary within the range 200 to 450 mm, depending on the location. A more detailed study of 1959 to 1981 data was undertaken for seven sites. However, still further research, particularly into the effects of bridging periods (intervals of, say, up to 3 days during which the temperature does not rise above +1°C) is needed before appropriate design depths can be assigned with confidence. The same quality of non-frost susceptible material will be required within the design depth but savings may be achieved by using poorer materials beneath.

AVAILABILITY OF WATER

The dramatic reduction of frost heave because of a low water table or an impermeable subgrade has been demonstrated by tests at TRRL in which full-scale freezing of road structures was simulated in 1.5-m deep test pits *(16–18)*. On average, lowering the water table depth from 0.6 m to 1.4 m reduced the surface heave from about 70 mm to less than 10 mm.

The number of subbase/subgrade combinations that can be investigated at full scale in a pit is limited. To provide further information, a series of TRRL frost-susceptibility tests was undertaken on composite specimens in which the bottom portion was subgrade soil and the remainder represented

TABLE 1 EVOLUTION OF THE TRRL TEST

	LR90 (1967)	SR318 (1977)	MM64 (1981)	SR829 (1984)
PRELIMINARY				
Sample Size		200 Kg (64 Kg for FH)	200 Kg (60 Kg for FS)	500 Kg (100 + 200 Kg for FS)
Grading		Representative	Nearest average (Must comply with D.Tp. Spec.)	Truly representative & BS812: Pt102: 1984
D_{100}/mm	50	37.5	37.5	37.5
MDD + OMC	Test 12 (BS1377)	as LR90	Test 14 BS1377 or MVHT (BS5835)	BS5835 Pt 1: 1980 or Test 14 BS 1377:1975
SPECIMEN PREP				
No. of Specimens			3 (then 9 if needed)	3 (plus 6 if needed; then 3 or 6 at different lab (4))
Mixing			Machine (after 16 hours tempering)	Machine (after 16 hrs tempering)
Compaction	Rodding + Static (>300kN)	Tamping (3 layers) + static (>400 kN) (Gap <2mm)	Tamping + vibrating hammer (3 layers) Gap ≯2mm	Tamping + vibrating hammer (3 layers) Gap ≯ 4mm
FREEZING				
Facility	CR	CR (/SRU)	SRU	SRU
Temperatures - Water	+4°C	+4 ± 0.5°C	+3.5 to +4.5°C$^{(3)}$	+3.0 to +4.5°C$^{(3)}$
Air	-17°C	-17 ± 1.0°C	-16 to -18°C$^{(3)}$	-16 to -18°C$^{(3)}$
Monitoring			10 Thermocouples	10 Thermocouples
Test duration				
Tempering /days	1	1	5	5
Freezing /days	10	10	4	4
Criteria for validity of test (assuming temperatures OK)			Range (of 3) > 5 mm s.d. (of 9) > 3 mm	1) Range ≯ 5mm (for 2<H <18mm) 2) H > 18mm 3) Repeat if H < 2mm or if >2 No. (from 9). specimens show heave drop > 1mm.
REPORTING		H_1 H_2 H_3; H	H_1 H_2 H_3 ...; H ρ_d + w PSD bulk PSD trial specimen	H_1 H_2 H_3 ...; H. ρ_d + w PSD bulk PSD trial specimen
UPPER LIMIT OF HEAVE/mm FOR NFS$^{(1)}$	13$^{(2)}$	13	10	12

SYMBOLS: H_1 H_2 etc = heave of individual specimens H = average of set. CR = Cold Room.

NOTES (1) Limit for SR318 and MM64 test specified separately by D.Tp. Based on 'grand average' heave of valid tests. NFS = non frost susceptible. (2) Metric equivalent. (3) Subject to more detailed requirements (see Appendix 2 of MM 64 or SR829 respectively.) (4) Tests at different labs if first lab obtains average between 9.1 and 15mm.

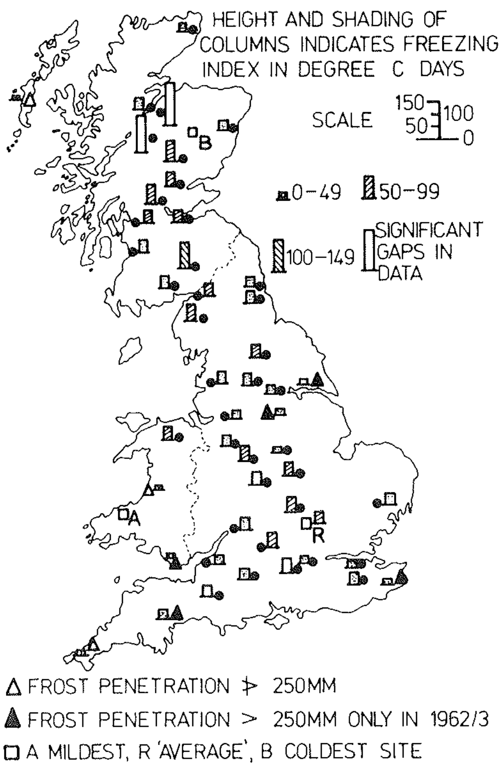

FIGURE 1 Mean yearly freezing index (1959 to 1979) for sites studied [after Sherwood and Roe *(15)*].

subbase aggregate. The results of all the tests known to the author are summarized in Table 2. The subgrade was a 20-, 30-, or 38-mm thick layer of London clay, gault (both heavy clays), Keuper marl (a silty clay), or Attenborough silt (nonplastic). The subbases consisted of whin (a basalt), dene (a carboniferous limestone), and two surrogate materials (a limestone filler and a limestone filler/sand mix). Most of the values quoted in Table 2 are the mean of three tests *(19, 20)*, and Thompson, private communication. Also included are two TRRL tests *(5)*.

The heave of the composite specimen was always less than that of the "subbase," with very considerable decreases occurring for the highest heaving "subbases." Roughly 75 percent of the results showed the composite to have an even lower heave than the subgrade, with the remaining composites having a heave intermediate between those of the individual components.

The observed heave is a product of the suction gradient and the permeability. The freezing front was always within the subbase, which would be expected to exert a lower suction than the finer-grained subgrades. Permeability, on the other hand, may decrease with suction, particularly if the bubbling point is exceeded. Thus the heave of the composite may be greater or less than that of the subgrade.

The important point is that, if there is restricted access to water, frost heave will be reduced. In the actual highway however, water can enter from the top (through hair cracks, joints, and so on), or possibly from the sides, and may form a perched water table. Often, subbases excavated for reconstruction have been found to be saturated. In general, there is a conflict between constructing a stiff well-graded subbase and a less stiff one with an open grading that will permit drainage. This is a matter of current research and debate; but meanwhile the possibility of a perched water table is difficult to discount in design.

TABLE 2 RESULTS OF TESTS ON COMPOSITE SPECIMENS

SUB-BASE	SUBGRADE THICKNESS/mm 0(1)	20	30	38	152(2)	SUBGRADE Type	LL	PL	PI	Class	Source Ref.
Silty Clay	40	-	-	13	10	London Clay	78	24	54	CV	(5)
Chalk	182	-	-	33	10	- do -	78	24	54	CV	(5)
Lst, Filler (LF)	102	-	-	2	11	- do -		31			(19)
SF3/60	47	-	-	5	12	- do -		31			(19)
SF60	45	9	4	2	11	- do -	59	26	33	CH	(20)
Whin	22	9	6	7	11	- do -	59	26	33	CH	(20)
Dene	5	2	-	2	11	- do -	59	26	33	CH	(20)
SF3/60	47	-	-	7	12	Gault		27			(19)
Lst Filler (LF)	102			18	16	Keuper Marl		18			(19)
SF3/60	47	-	-	23	16	- do -		18			(19)
SF60	45	9	12	5	23	- do -	31	16	15	CL	(20)
Whin	22	11	11	14	23	- do -	31	16	15	CL	(20)
Dene	5	2	-	2	23	- do -	31	16	15	CL	(20)
SF60	45	43	31	35	31	Attenborough silt	33	NP	-	NP	(20)
Whin	22	22	16	15	31	- do -	33	NP	-	NP	(20)
Dene	5	2	-	2	31	- do -	33	NP	-	NP	(20)

(1) heave of sub-base alone (2) heave of sub-grade alone

COMPARISON OF THE BRITISH, UNITED STATES, AND FRENCH CLASSIFICATIONS

In both the Cold Regions Research and Engineering Laboratory (CRREL) test in the United States *(21)* and the French Laboratoire des Ponts et Chaussées (LCPC) test *(22)*, cylindrical specimens are subject to unidirectional freezing with open access to water from below. The diameter of the specimens is 152 mm in the CRREL test but only 76 mm in the LCPC test (which precludes its use with aggregates). The CRREL specimens carry a surcharge of 3.5 kPa and are subjected to a constant rate of frost penetration of 13 mm/day.

Classification is based on the rate of heaving in the CRREL test and on the slope, p, of the heave versus FI graph in the French test. FI, the freezing index, is the area under the negative temperature-time graph.

An idea of how the TRRL classification relates to the CRREL and LCPC systems was obtained from TRRL tests in which additional monitoring of internal temperatures was undertaken. Some such tests were included in an extensive testing program undertaken at Nottingham University during the development work associated with the revision of the TRRL test. The results are included in Table 3. The materials tested included both sandy gravels and crushed-rock aggregates. The "as dug" or "crusher run" materials were washed, screened, and recombined to achieve reproducible gradings. This procedure tended to produce gradings that had rather less fines than the targets (see Figure 2) but this was unimportant provided that all the gradings within a set were consistent. Tests were undertaken on both Type 1 (essentially those with C to F1 gradings) and Type 2 aggregates (essentially those with F1 to F2 gradings) *(7)*. In addition, some tests were undertaken on mixtures of sand and limestone filler designated SF4/80 (see grading curve, Figure 2).

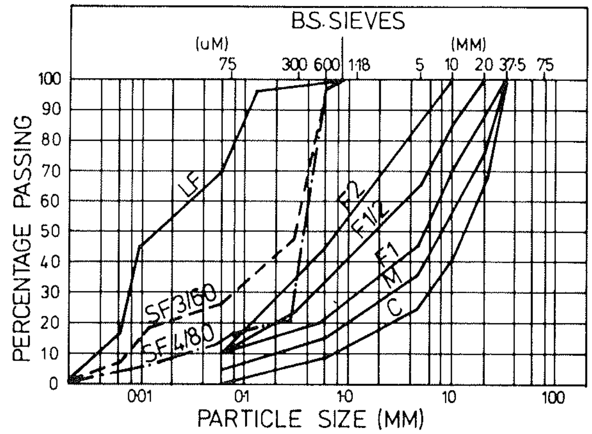

FIGURE 2 Grading curves.

The test procedure was generally similar to SR829 except that tempering was for 1 day only. Series *H* tests were undertaken in a cold room rather than an SRU but the two facilities have been shown to give similar results *(10)*.

Measurements showed that the frost penetration rate decreased continuously in the TRRL tests. In several tests on three materials (whin 2, 00L2, and concrete) an "instantaneous" value of 13 mm/day was recorded at times ranging from 16 to 30 hr, with an average of 24. Because the heaves of these three materials cover the main range of interest, it is reasonable to assume that the heave rate at 24 hr, $\dot{H}_{24}$, in the TRRL test will approximate that imposed throughout the CRREL test. The relationship between $\dot{H}_{24}$ and the 96 hr's heave H_{96} is shown in Figure 3.

For all the TRRL tests in which sufficient data had been recorded, the slope P, of the heave versus $(FI)^{1/2}$ was deter-

TABLE 3 PROPERTIES OF MATERIALS AND FROST HEAVE RESULTS

1 Material		2 Description	3 Grading	4 <20μ %	5 SMC %	6 MDD Mg/m^3	7 OMC %	8 $\theta_{2.5}$ %	9 N	10 H_{96} mm	11 $\dot{H}_{24}$ mm/d	12 P
SANDY GRAVELS												
Ashton Keynes	(102)	Lst (Jurassic)	M	1.0	4.0	2.11	7.3	9.5	9	11.7	4.5	
Spencers Farm	(103)	Flint hoggin	M	0.5	2.7	2.14	6.6	7.2	9	4.8	1.7	
Stanley Ferry	(106)	Gritstone	M	0.5	4.0	2.12	7.5	8.7	9	4.2	2.1	
Woodhall Spa	(114)	Sandy hoggin	F1	0	3.4	2.13	6.9	10.6	9	15.1	6.5	0.54
		with chalk	F1/2	0	2.9	2.04	7.9	-	9	14.7	5.8	0.50
		and flints	F2	0	2.4	1.96	8.9	9.3	9	11.3	5.1	
CRUSHED ROCKS												
Croft	(105)	Granite	M	0.5	1.6	2.15	6.9	4.3	9	1.6	1.0	0.08
Dene	(119)	Carb. Lst	M	1.0	2.1	2.17	5.4	5.8	3	3.8	1.4	
		" "	C	1.0	2.1	2.05	6.7	-	9	3.1	1.5	0.10
Whin 2	(B)	Dolerite	M	2.5	2.8	2.22	8.0	9.6	9	13.3	4.9	0.50
OOL 2	(B)	Shelly mudstone	M	-	4.1	2.05	7.4	12.5	9	12.7	3.8	0.71
OOL 1	(H)	Oolitic Lst	M		5.4	2.10	9.0	20.0	12	29.5	8.4	1.30
		" "	F1		5.4	2.11	9.0	21.3	4	25.5	6.8	
DOL	(H)	Dolomitic Lst	M		4.2	2.23	7.5	13.4	12	7.6	3.5	0.45
		" "	F1		4.2	2.25	7.8	14.7	4	12.6	4.1	
MIC	(H)	Microcellulitic	M		13.9	1.85	13.5	25.9	12	16.2	7.6	
		Calc/dolomite	F1		13.9	1.87	14.5	23.6	4	23.5	7.4	
CAL	(H)	Dolomitic Lst	M		4.6	2.20	7.5	15.4	12	25.6	7.9	
		" "	F1		4.6	2.21	8.0	17.3		25.1	6.7	
SILTY SAND												
SF4/80		Sand/Lst filler	-	1.0	-	2.00	9.00	12.8	18	17.5		

NOTES: 1. Reference code in parenthesis, 2. Lst = Limestone:Hoggin = gravel, sand & clay, 3. See Fig. 2. 4. Test 7C BS1377:1975. 5. Saturation moisture content of particles, 6-7. Maximum dry density and optimum moisture content, Test 14 BS1377:1975, 8. See text, 9. Number of specimens tested, 10. Heave at 96 hours, 11. Heave rate at 24 hours:mm/day, 12. Heave/(freezing index)$^{0.5}$:mm(°Chr)$^{-0.5}$

mined. The relationship between P and H_{96} is shown in Figure 4. P is a close analogue of p, which measured in the French test but is not identical to it because of differences in the testing regimes. There is some evidence *(23)* that the limits appropriate for P may be slightly higher than those for p, but any discrepancy appears quite small.

An approximate equivalence is demonstrated between the TRRL test and each of the other tests, especially if allowance is made for the 1 day's (rather than 5 days') tempering in these TRRL tests. Essentially, frost-susceptible materials in the United Kingdom correspond to those with high frost-susceptibility in the United States and France.

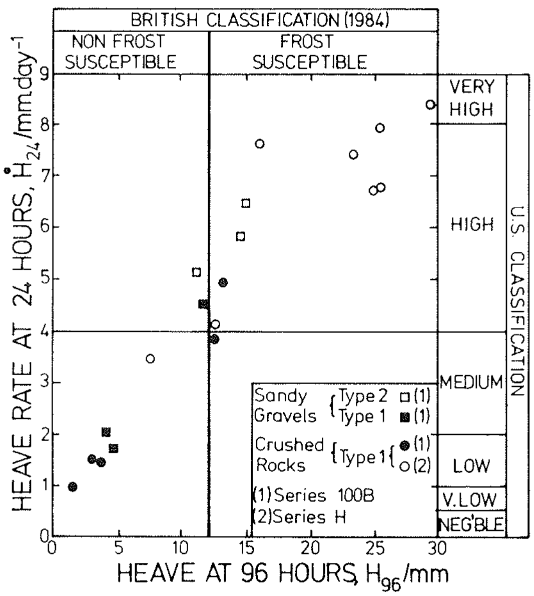

FIGURE 3 Approximate correlation between frost-susceptibility tests at the Transport and Road Research Laboratory in England and the Cold Regions Research Engineering Laboratory in the United States.

PHYSICAL FACTORS CONTROLLING THE FROST-SUSCEPTIBILITY OF AN AGGREGATE

Theory suggests that pore size distribution is a major determinant of frost susceptibility. Attempts have been made to use grading or saturation moisture content (SMC) criteria for frost susceptibility. Although the heaves shown in Table 3 are influenced by grading, all the aggregates have less than 3 percent of fines (< 20 μm) and would therefore be classified as non-frost susceptible by the Casagrande criteria *(21)*. In the TRRL tests, half of these materials proved to be frost susceptible. The criteria did however correctly classify the silty sand (SF4/80) as frost susceptible. Comparison of columns 5 and 10 in Table 3 shows that SMC also is an imperfect guide to frost susceptibility.

The inadequacies of grading and SMC criteria are not surprising because the former reflects only the between-particle pore size distribution and the latter only the within-particle porosity. In principle, suction characteristics, which reflect the volumetric distribution of both pore systems, are likely to yield a more reliable indicator of frost susceptibility.

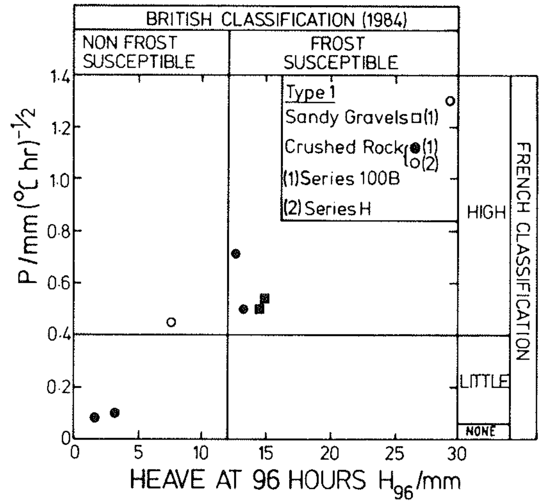

FIGURE 4 Approximate correlation between frost-susceptibility tests at the Transport and Road Research Laboratory in England, and the Laboratoire des Ponts et Chaussées in France.

Measurement of Suction Characteristics

A technique of obtaining suction characteristics of aggregates has been developed *(24)*. Briefly, aggregates at their optimum moisture content are compacted into a mold of plastic pipe, frozen and sliced with a diamond saw into disc-shaped specimens of 110 mm dia by 15 to 12 mm high. The specimens are flanked with semipermeable (dialysis) membranes in a cell that is then placed in a tank containing polyethylene glycol (carbowax) 6000 (see Figure 5). Water passes through the membrane so that the moisture content of the specimen equilibrates with the osmotic pressure of the carbowax (which has been determined by previous calibration). Seven days are allowed for equilibrium. A series of tests at different concentrations of carbowax enables the suction-moisture content curve to be determined over a range of pF 1.5 to 4.4 (4 to 2500 kPa). Capillary rise methods are used at lower suctions.

Relationship Between Suction Characteristics and Frost Heave

Inspection of Table 3 and the suction-moisture-content curves (Figure 6) revealed that the volumetric moisture content at a suction of pF 2.5 (31 kPa), $\theta_{2.5}$, ranked most of the materials in the same order as the frost heave. For $\theta_{2.5}$ less than 20 percent H_{96} increased with $\theta_{2.5}$, but there was a decrease thereafter (see Figure 7). It has been suggested *(24)* that for aggregates with an optimum moisture content of less than 10 percent, $\theta_{2.5}$ is associated with a characteristic suction in the frozen fringe. However, both this suggestion and any frost-susceptibility criterion developed from it, must be regarded as tentative until they are confirmed by tests on many more materials.

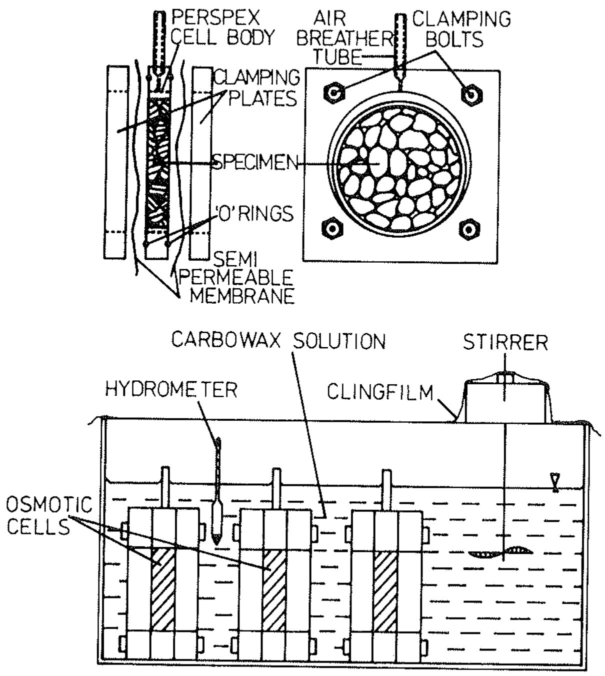

FIGURE 5 Osmotic suction apparatus.

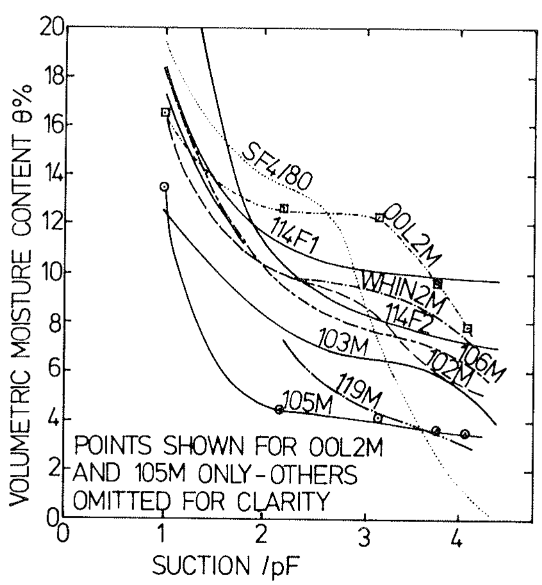

FIGURE 6 Suction characteristics.

Nevertheless, the relationship between $\theta_{2.5}$ and H_{96} is consistent with the applicability of the capillary theory to subbase aggregates.

The apparatus to measure $\theta_{2.5}$ is simpler, cheaper, and more easily duplicated than that for direct frost heave tests. The suction method could be a useful supplement, especially where only small quantities of material are available or where a large number of materials need to be processed simultaneously, for example at the preliminary investigation or tendering stages.

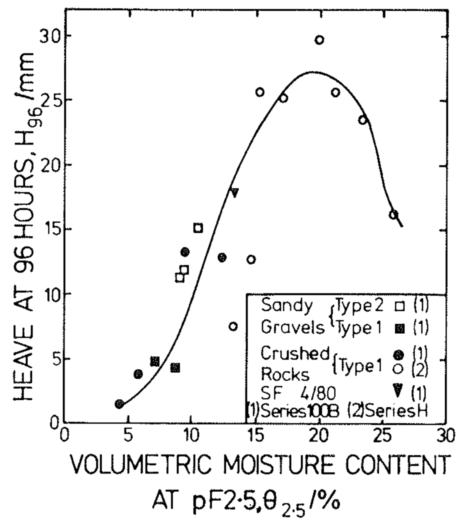

FIGURE 7 Correlation of frost susceptibility and aggregate suction $\theta_{2.5}$.

OPTIMIZING THE USE OF BORDERLINE MATERIALS

A borderline material is most likely to satisfy the frost-susceptibility criterion if it is produced at the coarsest permitted grading (this minimizes the effect of the between-particle pores). The contribution of within-particle pores may be reduced either by excluding suspect horizons at source or by mixing with better material. Experience suggests that careful testing is required to optimize any proposed mixing procedure.

Following the discussion in the earlier part of the paper, another use for borderline materials is in locations where the zero isotherm is unlikely to penetrate or where water is absent.

CONCLUSIONS

The following conclusions were reached:

1. Detailed changes to the TRRL test have improved reproducibility;
2. The design depth to which frost protection is required could safely be reduced below the presently specified 450 mm in the milder parts of lowland Britain, although further research is needed to quantify the depth appropriate to a particular site;
3. Even where relatively frost-susceptible aggregates are used within the depth of frost penetration, the surface heave will be much reduced if the supply of water is restricted by drainage or an impermeable subgrade. However, the potential improvement may be nullified if a perched water table exists;
4. Materials classified as frost susceptible in the United Kingdom are likely to be in the high frost-susceptibility categories in the United States and France;
5. Suction characteristics appear to be better indicators of frost susceptibility than grading or saturation moisture content. In particular, for a range of granular materials, frost heave and $\theta_{2.5}$ (the volumetric water content at a suction of pF 2.5,

equivalent to 31 kPa) were related by a single curve. However, any frost-susceptibility criterion based on suction characteristics must be regarded as tentative until supported by considerably more data; and

6. An understanding of the basic factors affecting frost heave enables the best use to be made of borderline materials. Lower-quality material may prove satisfactory if used at a coarse grading or placed where the probability of frost penetration or freely available water is low. There may also be some scope for improvement by mixing with better-quality materials.

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