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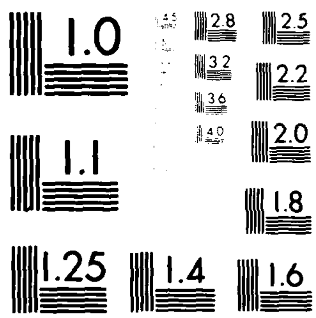

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NEW HAMPSHIRE FIELD STUDIES OF MEMBRANE ENCAPSULATED SOIL LAYERS WITH ADDITIVES

Robert A. Eaton and Richard L. Berg

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This report describes the construction, instrumentation, and performance of membrane encapsulated soil layer (MESL) pavement test sections at the U.S. Army Cold Regions Research and Engineering Laboratory in Hanover, New Hampshire, from 1973 to 1978. Membrane encapsulated soil layer construction involves using a waterproof membrane to protect low grade soils from absorbing moisture, especially during the freezing process. Most of these lower grade soils are frost-susceptible; in these soils water can be drawn to the freezing zone to form ice lenses, which in turn cause heaving of the surface. Lime, flyash, → next page

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and sodium chloride were added to a silt material prior to encapsulation. These additives were incorporated to add strength to the silt, absorb excess moisture, and increase its load-supporting capabilities. Results show that 1) the moisture content within the MESL sections remained relatively constant over the five years of testing, 2) a nonencapsulated lime-flyash-stabilized silt material heaved 8.8 times as much as the identical material which was encapsulated, 3) the lime-flyash-stabilized MESL had twice the strength of the plain or salt-stabilized MESL, 4) the silt with the additives had less frost heave within the MESL than the untreated silt. In summary, MESL's can be constructed to perform well in cold regions, thereby replacing high quality aggregates which are being depleted.

PREFACE

This report was prepared by Robert A. Eaton and Dr. Richard L. Berg, Research Civil Engineers, of the Geotechnical Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. John M. Sayward, Research Chemist of the Earth Sciences Branch, Research Division, authored Appendix B and Jonathan Ingersoll, Civil Engineering Technician of the Geotechnical Research Branch, Experimental Engineering Division, authored Appendix C.

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North Smith of CRREL technically reviewed the manuscript of this report.

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NEW HAMPSHIRE FIELD STUDIES OF MEMBRANE ENCAPSULATED SOIL LAYERS WITH ADDITIVES

Robert A. Eaton and Richard L. Berg

OBJECTIVES

In the period from August to October 1973, an access road consisting of nine highway pavement test sections was constructed at CRREL (Berg 1974). This test road was designed, in part, to evaluate the performance of additives (sodium chloride, lime and lime with flyash) and their effects within MESL sections using a highly frost-susceptible silt. Observations included air temperatures, subsurface temperatures, surface elevations for frost heave and/or settlements, moisture contents in the base and subgrade, strain measurements of the subsurface materials, and pavement deflections under a loaded truck as measured with the Benkelman beam. The test sections were located on an access road subject to very low volume, low weight vehicular traffic.

SITE CONDITIONS

The test area is located on the west side of the CRREL property (Fig. 1). CRREL is located on the lacustrine deposits of Lake Hitchcock, a former glacial lake of the Pleistocene epoch. The climate is subcontinental or woodland of the cool temperate zone (Landsberg et al. 1965). The design air freezing index (based on the average of the 3 coldest years in 30) for Hanover, New Hampshire, is 1820°F-days. The mean air freezing index is 1060°F-days (Bilello 1966).

TEST SECTIONS

The road containing the MESL test sections is 970 ft long with a 16-ft-wide traffic surface and 4-ft-wide paved shoulders on each side. The natural surficial soil in the test area is primarily a nonplastic frost-susceptible sandy silt classified as ML in the Unified Soil Classification System (U.S. Department of Defense 1968). Prior to construction, the silt ranged in thickness from less than 1 ft to approximately 7 ft, and the in-situ moisture content ranged from 25% to 35% by dry weight. These moisture contents were 10% to 20% wetter than desired for adequate compaction; therefore, the segments of the road in cut areas were aerated to induce drying.

Underlying the sandy silt is a varved clay (CL) having a plasticity index of 7, a liquid limit of 31, and an in-situ moisture content ranging from 30% to 40% by weight. (The gradation curve for this clay is shown in Appendix Fig. A1.) Varved clay was encountered from stations 6+10 to 6+50. To facilitate operation of the construction equipment and to provide both a more stable platform and a more homogeneous subgrade for the subsequent roadway, 2 ft of the varved clay was removed and replaced by gravel borrow (see Fig. A1 for the gradation curve). An adequate thickness (approximately 4 ft) of sandy silt overlies the clay from station 7+10 to 9+70.

Three of the access road test sections (section F, station 4+90 to 6+10; section G, station 6+10 to 7+30; and section H, station 7+30 to 8+50) are of MESL construction (Fig. 2) as developed and refined by the U.S. Army Engineer Waterways Experiment Station (Sale et al. 1973). CRREL has installed and evaluated similar MESL test sections in Alaska (Smith and Karalius 1973, Shaefer 1973, and Smith and Pazsint 1975).

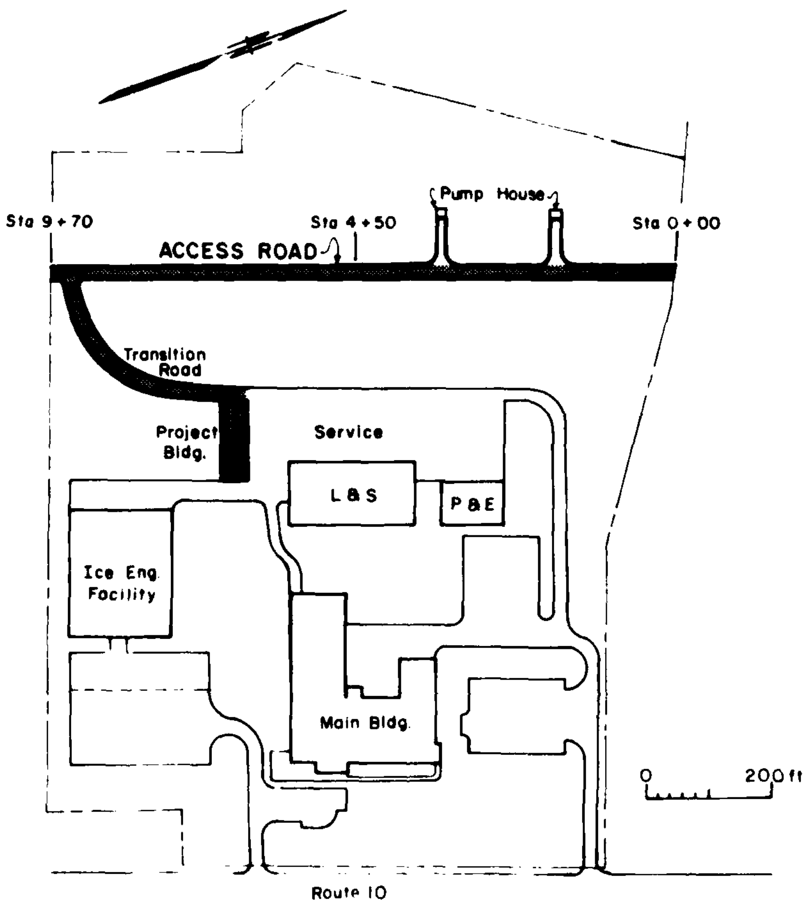


Figure 1. Site plan.

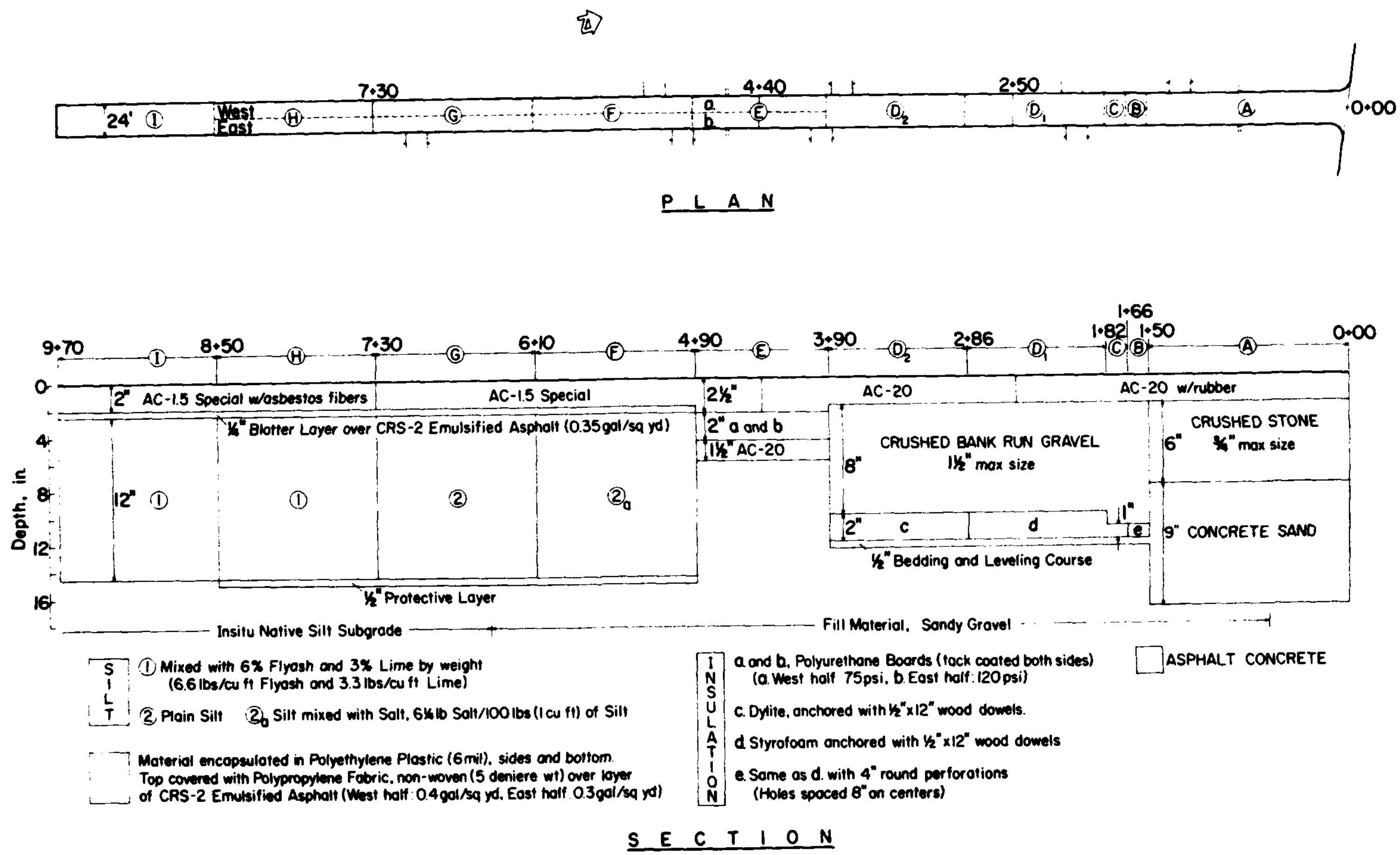


Figure 2. Plan and profile views of access road.

The basic difference between the conventional road design and the three MESL test sections on the access road is that the material encapsulated is a highly frost-susceptible silt (90% minimum passing a no. 200 sieve). In section F, approximately 6-1/4 lb of deicer grade sodium chloride was mixed with each cubic foot of silt in order to lower the freezing point of the silt below the minimum temperature anticipated to occur in the layer (see App. B). This quantity of sodium chloride should have lowered the freezing point to about -6°F (the eutectic temperature for sodium chloride); however, samples obtained in February 1974 froze in the laboratory at approximately -10°F. This indicated other salts having lower eutectic temperatures were probably present.

Section G is the "control" or untreated MESL section. The encapsulated silt in section H was mixed with 3% by weight of agricultural lime and 6% by weight of flyash. The design curves used to establish proportions of lime and flyash are shown in Appendix C.

Section I consists of the same lime-flyash treated silt as section H, but it was not encapsulated to determine the benefits of isolation by membranes.

The construction procedure for all three MESL test sections was essentially the same. First, crushed bank run gravel berms were constructed on each side of the road to confine the encapsulated material. A concrete sand cushion approximately 1/2 in. thick was placed on the subgrade and 6-mil-thick polyethylene film, 24 ft wide, was placed on the sand. Transverse joints were overlapped 12 in. and sealed with CRS-2 (catonic, rapid-setting emulsified asphalt). The material to be encapsulated was placed by end dumping and compacted in two 6-in.-thick layers to 98% of the laboratory attained density. The ends of polyethylene film were brought to the surface at stations 4+90 and 8+50. The test sections were then sprayed with CRS-2 and allowed to cure for approximately 15 min prior to placing a polypropylene non-woven fabric on the surface. The material used was "Petromat" (Phillips Fiber Corp., a subsidiary of Phillips Petroleum Co.), weighing 4 oz/yd^2. The individual fibers were 5 denier (which means that 9000 m weighs 5 g) or about 27 μm (about 1.1 mil) in diameter. The fabric came in 12-ft wide sheets, necessitating use of two strips with a 12-in. overlap longitudinal joint along the centerline of the road. Again the joints were made by applying CRS-2 on one layer of membrane and then overlapping the other membrane. After placement of the polypropylene fabric, CRS-2 was sprayed on the surface. The CRS-2 was allowed to cure about 30 minutes before a sand "blotter" layer was applied with a truck-mounted sander. The excess sand was removed a few days later with a mechanical sweeper and the 2-in.-thick asphalt concrete pavement was placed.

It should be pointed out that a period of at least nine days was required to aerate and dry the silt to below its optimum moisture content as required for MESL construction in seasonal frost areas. Also, the material had to be covered with plastic at night and over weekends to protect it from rain, overnight dew and fog.

Between station 4+40 and 9+70, a high viscosity, high penetration asphalt cement graded at approximately AC 1.5 was used in the asphalt concrete mix. To reduce the extent of probable rutting in this soft material during the warmer months, asbestos fibers were added to the mixture between stations 7+30 and 9+70.

Pertinent properties of the soils and paving materials are shown in Appendix A.

INSTRUMENTATION

Thermocouples

Copper-constantan thermocouples were installed throughout the access road to monitor:

1) the rate of frost penetration and recession,

2) the maximum depth of frost penetration,

3) the number of freeze-thaw cycles, and

4) various isotherm locations.

Figure 3 shows the plan view locations of the thermocouple assemblies and a cross-sectional view giving depth locations beneath the surface.

The thermocouple assemblies are made from 20-gage copper-constantan wire (PVC covering, premium grade, ± 3/4°F accuracy within an operating range of -75°F to + 200°F). Wire pairs of individual thermocouples lead to the edge of the section in which they are located and to a connector box where they are connected to ± 1-1/2°F accuracy extension wire in 6, 8, 12, or 24 pair cables. The extension cables run to the instrument shelter located 40 ft left of centerline at station 4+90, where they are read by a digital data collection system. The data system is a 300 channel capacity Joseph Kaye Model 8000 with an accuracy of ± 0.1°F. A duplexer was built by the CRREL Electronics Section to allow the data system to read 480 thermocouples with its 300 channels. Temperature readings are recorded on punched paper tape at two-hour intervals for computer analysis.

Soil Strain Gages

Soil strain gages were installed to measure static long-term deformations and dynamic strains in the silt material (sections F, G & H). The basic components used to make these measurements are induction coil strain sensors which utilize the principle of electromagnetic coupling between sensor pairs. The sensors used were Bison Instruments Models 4011 and 4044. This type of instrument requires no

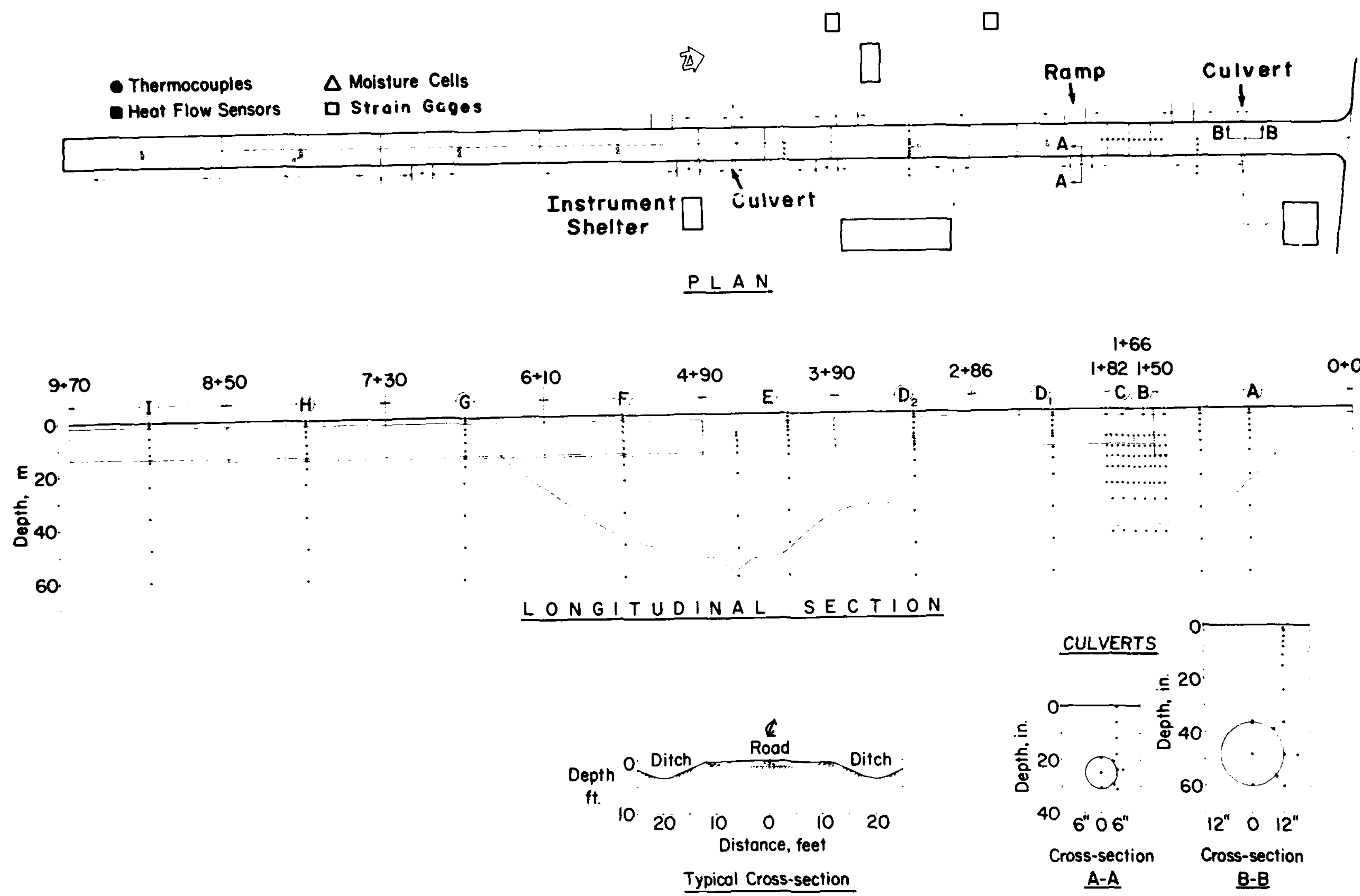


Figure 3. Plan and profile view showing instrumentation locations.

mechanical linkage between the sensors as do conventional displacement transducers, thereby minimizing the disturbance of the material to be measured. A sensor pair consists of two disk-shaped coils, which are placed coaxially either vertically or horizontally depending on the strain measurement desired. The diameters of the sensors are 1 in. (Model 4011) and 4 in. (Model 4044) and the thicknesses are 0.1 in. and 0.4 in. respectively. The sensor cables run from the point of installation to a protective box adjacent to the test section. Readings are taken by connecting the cables to an external instrumentation package (Bison Strain Gage Model 4101A), consisting of driving, amplification, balancing, readout, and calibration controls and a self-contained power supply. According to the manufacturer, the sensor pairs are capable of measuring dynamic strains of 0.00005 in./in. and long-term static strains of 0.005 in./in. with an accuracy of 1%. The locations of the sensors are shown in Figure 3 and listed in Table I. A pair of 4-in.-diam. sensors is located within the MESL sections (F, G & H), the top sensor being 5 in. below the pavement surface and the other 14 in. below. This 9-in. spacing between sensors is about twice the 4-in. diameter, as is required for proper operation. All sensors are 2 ft east of the centerline. The alignment pattern is designed to measure the vertical strain in the various silt mixtures.

Table I. Locations of Bison strain gages.

Section	Station	Location	Depth below pvt surface (in.)	Size (in.)
F	5+50	2 ft east of centerline	7 14	4 4
G	6+70	2 ft east of centerline	5 14	4 4
H	7+90	2 ft east of centerline	5 14	4 4
I	9+10	2 ft east of centerline	5 14	4 4

Benkelman Beam Tests and Level Surveys

Static pavement rebound measurements and level surveys of established points on the pavement surface were conducted on the sections. There were two Benkelman beam test points per section (Table II). Level surveys were conducted at every 25-ft station on the inside and outside edges of the shoulders and at the centerline. The level surveys monitored frost heave and permanent deformation.

Table II. Benkelman beam test point locations.

Test point	Section	Station	Location from Centerline
11	F	5+30	4' Lt
12	F	5+70	4' Rt
13	G	6+50	4' Lt
14	G	6+90	4' Rt
15	H	7+70	4' Lt
16	H	8+10	4' Rt
17	I	8+90	4' Lt
18	I	9+30	4' Rt

Meteorological observations

Air temperatures and other climatological observations have been made at a field site located on CRREL property approximately 200 ft east of the road and at an elevation approximately 30 ft higher. The observations have been made by the CRREL Detachment of the Maynard, Massachusetts, Meteorological Team, U.S. Army Atmospheric Sciences Laboratory, which provides monthly tabulated reports.

TEST RESULTS AND OBSERVATIONS, 1973-1978

Air Temperatures

Air temperatures at the access road site may be slightly lower than those measured by the meteorological team due to the slightly lower elevation. Figure 4 illustrates average daily air temperatures for the 1973-1974 winter. Also shown in the figure are the long-term maximum, minimum and average monthly temperatures for the National Oceanic and Atmospheric Administration meteorological station in Hanover, New Hampshire. This station is at Dartmouth College and is approximately one mile south of the CRREL facility.

The mean air freezing index for the Dartmouth meteorological station is 1060°F-days and the design air freezing index is 1820°F-days, on the basis of the average of the 3 coldest years in 30 (Gilman 1964). Following is a list of air freezing indices for each year:

1973-1974:	1,021°F-days
1974-1975:	955°F-days
1975-1976:	1,408°F-days
1976-1977:	1,636°F-days
1977-1978:	1,513°F-days.

A plot of cumulative freezing degree-days is given in Figure 5.

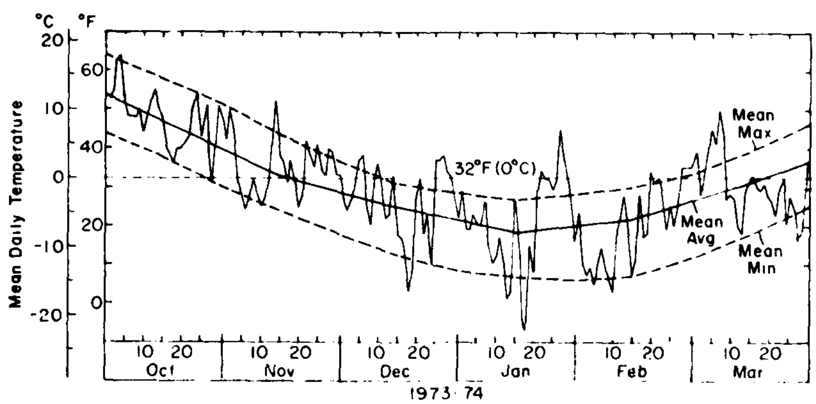


Figure 4. Average daily air temperatures, 1973-74.

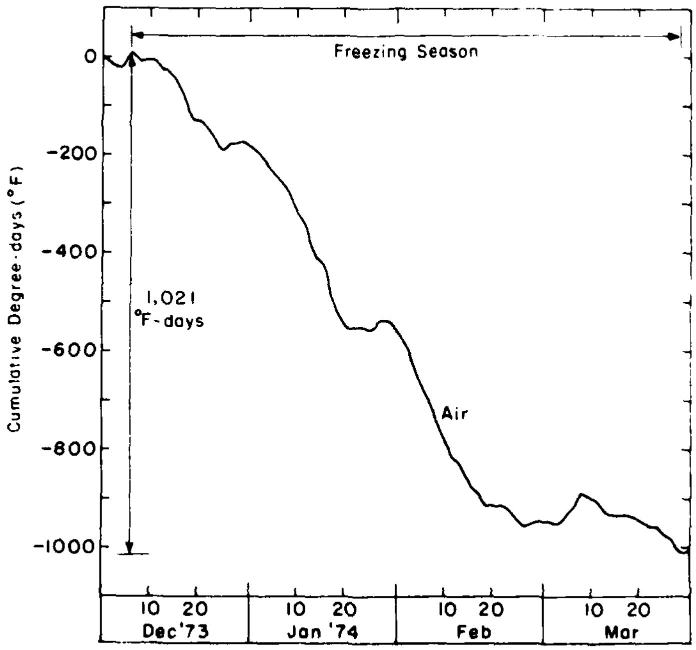


Figure 5. Freezing index at CRREL for winter of 1973-1974.

Frost Penetration

Ground temperature data are available from late January 1974, when the monitoring system was installed. Temperature sensors in all of the sections were monitored once every two hours.

Figure 6 illustrates measured penetration of the 32°F isotherm in sections F, G and H. Frost depths were calculated for each section using the modified Berggren equation and the air freezing index for the 1973-74 winter. Calculations were also made using the mean air freezing index and the design air freezing index. Calculated depths are shown in Table III. When using air freezing index values to calculate frost penetration depths, a surface transfer coefficient "n-factor" is used to provide a surface freezing index. The n-factor for winter conditions recommended by the U.S. Army (1966a) is 0.90 for paved surfaces.

Some surface temperatures at the access road were not measured for the entire winter; it was therefore not possible to determine n-factors for these test sections. However, results from other test sections at CRREL indicate that the n-factors for the 1973-74 winter were approximately 0.45 for asphalt concrete pavements (Eaton 1976). N-factors of both 0.90 and 0.45 were used in the calculations and are compared with measured data for the 1973-74 winter in Table III. The n-factor of 0.45 provided results in much closer agreement than those calculated with an n-factor of 0.9 for the salt and lime-flyash stabilized sections. The computations assumed that no pore water froze in the salt-stabilized MESL test section. Soils data used in the computations are shown in Table IV. Discrepancies between the measured and calculated frost depths are due to three primary uncertainties or inaccuracies: 1) inaccurate surface freezing indexes, 2) wrong values used for soil properties, and 3) an inadequate thermal model. Thermal conductivities for the salt-silt and lime-flyash mixture were approximated by using values for the soil without additives.

Frost Heave

General

Table V contains a list of the soils used in the construction of the CRREL access road. Each of the soils is categorized according to the frost groupings developed by the U.S. Army (1966b). It should be noted that the classifications listed in Table V are based entirely on the Unified Soil Classification System and the grain percentage finer than 0.02 mm. Frost susceptibility tests have not been conducted on any of these soils except the CRREL sandy silt which was classified as very highly frost-susceptible.

Level Surveys

Pavement surface elevations were obtained approximately every week throughout the winter. However, in early March 1974 it was discovered that the manhole being used as a benchmark was also heaving. It was, therefore, not possible to determine accurately the frost heave at any given time throughout the first winter until 15 March when a frost-free benchmark was installed in the area.

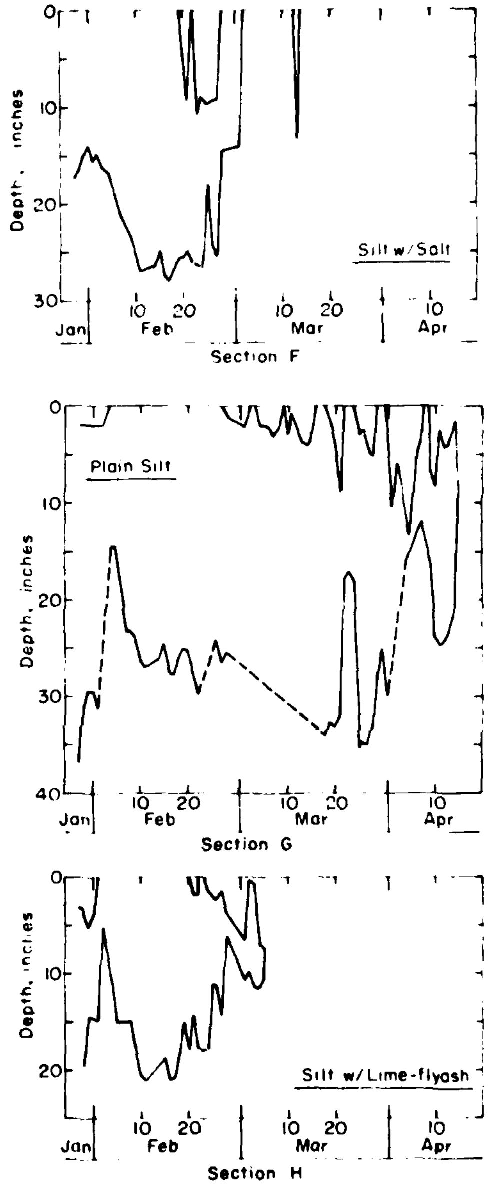


Figure 6. Penetration of 32°F isotherm, 1973-1974 winter.

Table III. Maximum frost penetration (ft), CRREL access road 1973-1974 winter.

Test section	Calculated using design freezing index* n = 0.90	n = 0.45	Calculated using mean freezing index n = 0.90	n = 0.45	Calculated using 73-74 freezing index n = 0.90	n = 0.45	Measured 73-74
F	4.2	3.2	3.5	2.1	3.4	2.1	2.3
G	4.2	3.0	3.3	2.1	3.3	2.0	2.9
H	3.6	2.5	2.8	1.9	2.7	1.8	1.8

* Frost depths are calculated with modified Berggren equation.

Table IV. Soils data used for modified Berggren equation solutions.

	Dry density (lb/ft^3)	Moisture content (%)	Layer thickness (ft)	Volumetric heat cap. ($Btu/ft^3 °F$)	Thermal conductivity (Btu/ft hr)	Volumetric latent ht. (Btu/ft^3)
			Section F			
Pavement	145	0	0.17	28.	0.86	0
Silt MESL	100	20	1.0	32.	0.90	0
Granular fill	110	10	2.2	26.95	1.20	1584
Silt subgrade	85	27	2.5	31.66	0.81	3305
Silt subgrade	80	40	5.0	37.6	0.97	4608
			Section G			
Pavement	145	0	0.17	28.	0.86	0
Silt MESL	95	20	1.0	30.4	0.82	2736
Granular fill	110	10	1.5	26.95	1.20	1584
Silt subgrade	85	20	2.5	27.2	0.66	2448
Silt subgrade	80	40	5.0	37.6	0.97	4608
			Section H			
Pavement	145	0	0.17	28.	0.86	0
Silt lime-flyash	87	16	1.0	25.23	0.59	2004
Silt subgrade	105	10	1.0	25.72	0.66	1512
Silt subgrade	85	20	2.5	27.20	0.66	2448
Silt subgrade	80	40	5.0	37.6	0.97	4608

Table V. Classification of soils used in construction of the CRREL access road.

Material	Unified soil classification system	% finer than 0.02 mm	Frost group
Concrete sand	SP	0	NFS
Gravelly sand subbase	SW	2.5	NFS
Moulton pit silt (MESL material)	ML	83.0	F4
CRREL sandy silt	ML	14.0	F4

Figure 7a illustrates elevations at the centerline of the road on 29 November 1973 and 15 March 1974. At the time of the November survey no frost penetration had occurred. Results from the full depth and reduced subgrade strength test sections at CRREL (Eaton 1976) indicate that the maximum frost heave on these test sections occurred on 10 March 1974. From subsurface temperature data, summer 1974 survey data and the frost free-benchmark, Figure 7a was drawn which represents the maximum frost heave profile for the 1973-74 winter. Figures 7b - 7e show centerline elevations for 1974-75, 1975-76, 1976-77 and 1977-78 respectively.

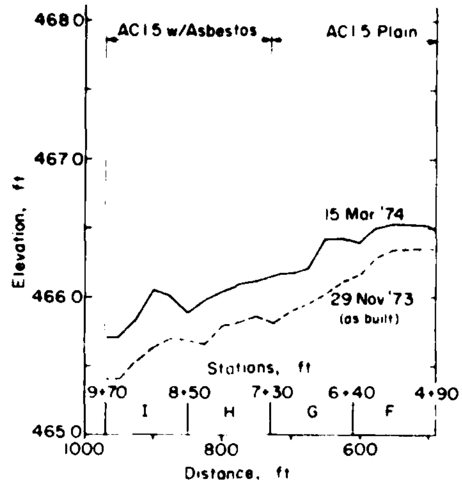
Subsurface Strain Gages

Bison soil strain gages developed by Selig and Grangoard (1970) were installed in each of the MESL sections (F, G and H) and in unencapsulated section I. The U.S. Army Engineer Waterways Experiment Station (WES) had used these gages in some of their studies on pavements. The electronic device for monitoring these strain gages was loaned to CRREL by WES, and each pair of gages was calibrated by CRREL personnel. One reason for installing the gages in the MESL sections was to determine the amount of frost heaving caused by the freezing of the encapsulated layers. Data in Table VI indicate that there was some frost heave within the encapsulated material in each of the encapsulated sections.

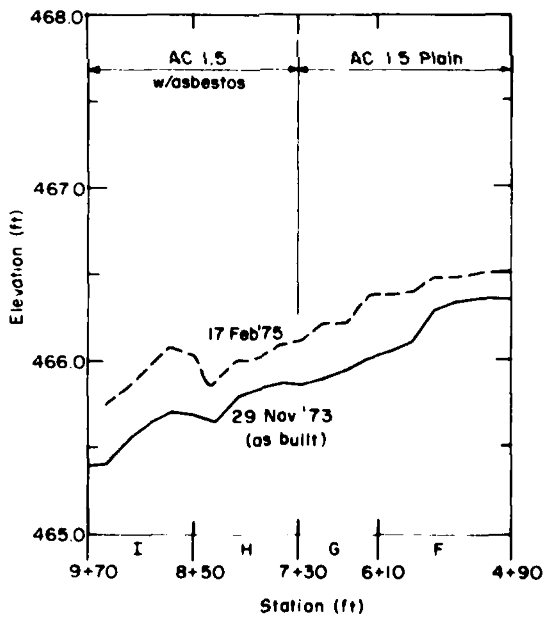
The recorded frost heave was as follows:

1. 0.13 in. within the lime-flyash-stabilized MESL
2. 0.22 in. within the salt-stabilized MESL
3. 0.37 in. within the plain MESL
4. 1.14 in. within the nonencapsulated lime-flyash-stabilized silt.

Comparing the above maximum frost heaves within the MESL material to the total maximum heave measured on the pavement surface at the gage

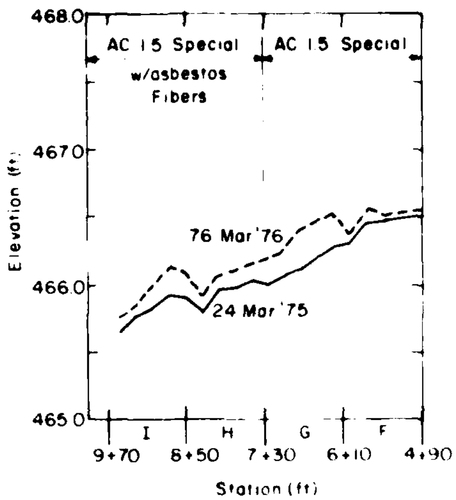


a. 1973-1974

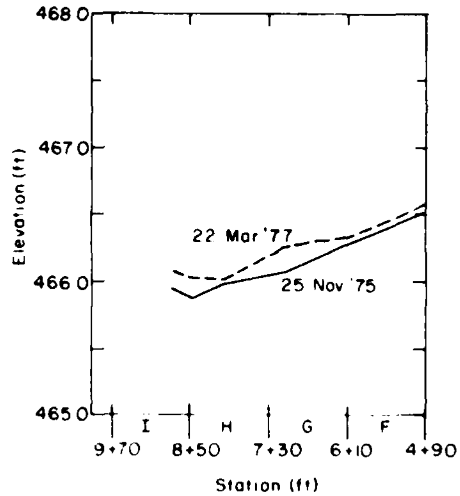


b. 1974-1975

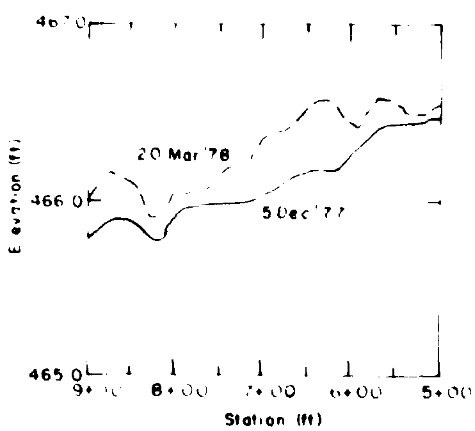
Figure 7. Centerline elevations of CRREL access road, 1973-1978.



c. 1975–1976



d. 1976–1977



e. 1977–1978

locations yields the following ratios and percentages:

1. $\frac{0.13 \text{ in.}}{3.00 \text{ in.}} = 4.3\%$

2. $\frac{0.22 \text{ in.}}{2.28 \text{ in.}} = 9.6\%$

3. $\frac{0.37 \text{ in.}}{2.88 \text{ in.}} = 12.8\%$

4. $\frac{1.14 \text{ in.}}{3.96 \text{ in.}} = 28.8\%$

These results show that by wrapping the silt at optimum moisture content, frost heave within the material itself was more than reduced by half. The silt material which was treated with lime and flyash, or salt, heaved only one-third or one-seventh as much.

This proves the benefit of encapsulation upon frost heave.

Tests were not run in the following years due to equipment malfunction and gage destruction.

Table VI. Soil strain gage measurements (in.), not temperature corrected.

Section/Date	F	G	H	I
25 Jan 74	7.74	9.10	9.60	9.65
29 Jan 74	7.70	9.20	9.57	9.79
31 Jan 74	7.64	-	-	-
12 Feb 74	7.67	9.20	9.60	9.66
15 Feb 74	7.84	9.36	9.60	9.78
19 Feb 74	7.95	9.35	9.52	9.70
21 Feb 74	7.64	9.21	9.59	9.65
26 Feb 74	7.73	8.99	9.47	9.32
17 Apr 74	7.73	8.99	9.50	8.64

Frozen Soil Cores

In February and March 1974, samples were obtained by coring the frozen materials in sections F, G, H and I. In all of the MESL sections care was taken not to penetrate the bottom membrane; however, it was accidently penetrated in Section F. Subsequently, the material in section F was cored to a depth of approximately 70 in. below the pavement surface. At that depth the water table was encountered but samples of the saturated material could not be obtained.

Figure 8 illustrates the coring equipment used in this project. All the coring in February 1974 was accomplished with the small drill

Table VII. Average moisture contents (% by weight).

Section F (salt MESL)

Depth (in.)	7 Mar. '74 Sta. 5 + 80 (3 ft lt)	19 Feb. '75 7 + 75 (2 1/2 ft rt)	15 Mar. '76 5 + 26 (3 ft rt)	17 Feb. 77 5 + 40 (2 ft rt)	8 Mar. '78 5 + 23	8 Mar. '78 5 + 50
1 - 12	20	19	18	19	19	27*
12 - 24	15	-	11	13	18	-
24 - 36	28	-	16	23	-	-
36 - 48	25	-	20	14	-	-
48 - 60	29	-	-	9	-	-
60 - 72	30	-	-	-	-	-

Section G (plain MESL)

Depth (in.)	21 Feb. 6 + 40 (4 ft rt)	14 Feb. 7 + 01 (3 ft rt)	15 Mar. 6 + 62 (Center)	17 Feb. 6 + 85 (4 ft rt)
1 - 12	19	23	21	23
12 - 24	-	-	13	24
24 - 36	-	-	42	45
36 - 48	-	-	-	41
48 - 60	-	-	-	26

Section H (lime-flyash MESL)

Depth (in.)	21 Feb. 7 + 77 (5 ft rt)	19 Feb. 8 + 25 (5 ft lt)	15 Mar. 8 + 12 (Center)	17 Feb. 7 + 57 (5 ft rt)	8 Mar. 8 + 00
1 - 12	16	17	17	19	28.3*
12 - 24	-	-	18	24	-
24 - 36	-	-	23	24	-
36 - 48	-	-	-	15	-
48 - 60	-	-	-	18	-

Section I (unencapsulated lime-flyash)

Depth (in.)	7 Mar. 8 + 70 (5 ft lt)	19 Feb. 8 + 90 (5 ft rt)	15 Mar. 9 + 20 (Center)	17 Feb. 8 + 90 (4 ft rt)	8 Mar. 8 + 75
1 - 12	40	41	38	42	47
12 - 24	33	41	33	29	-
24 - 36	26	-	21	20	-
36 - 48	19	-	-	18	-
48 - 60	26	-	-	14	-
60 - 72	40	-	-	-	-

* Near prior hole where moisture entered MESL

Figure 8. Coring equipment.

normally used for coring portland cement concrete. For these tests a core barrel with a carbide tipped bit was used to core the pavement and the frozen material. Compressed air was used as the drilling fluid. The cores were approximately 3 in. in diameter.

The coring procedure used in March 1974 was slightly different from that used in February. The carbide-tipped core barrel was used to core only the pavement layers and a modified CRREL auger was used to obtain cores of the frozen and unfrozen materials below the pavement.

The primary reason for coring the MESL sections was to determine the extent of moisture redistribution that took place during the freezing process. Table VII contains moisture content data obtained from the cores. It appears that very little moisture redistribution had occurred within the encapsulated materials.

Benkelman Beam Deflections

Benkelman beam tests are used by various agencies to indicate the strength of the pavement and base materials (ASTM 1970). There is some variation in the readings due to the temperature influence on the stiffness of the asphalt cement, and various agencies have developed methods of correcting deflections to a standard temperature, normally 70°F (Asphalt Institute 1969). The correction factors or procedures used by various agencies differ; however, there are no factors for temperatures below about 35°F. The correction procedures also differ for various pavement profiles, paving materials, and asphalt cements. The Benkelman beam deflections are also a function of whether the support layers beneath the pavement are frozen or thawed. Frozen support layers naturally decrease the Benkelman beam deflections significantly. Temperature correction factors were not employed in this study because of the various grades of non-standard asphalt cements used in the pavements.

Table VIII and Figure 9 show the Benkelman beam deflections for sections F, G, H, and I for all five years (1973-1978).

The lime-flyash-stabilized material which was placed in section I did not have time to cure properly before winter. The extra fine particles made the highly frost-susceptible silt even more so. The ensuing winter and freezing process caused this material to draw excess moisture upward and the section heaved approximately 4 in. in a manner similar to a box rising out of the ground.

When the material thawed in the spring, it was saturated and deflected under the Benkelman beam up to 0.2 in.

Over the course of the 1974 summer, the material had a chance to cure and gain in strength. This strength increase is vividly shown by the next year's performance. Even though the total frost heave was approximately the same, the Benkelman beam deflections were a maximum 0.086 in., or less than half the first year deflections.

The deflections for the remaining years follow the same general trends with section H (lime-flyash-stabilized silt MESL) the strongest, followed by section F (salt-stabilized silt MESL), section I (non-encapsulated lime-flyash-stabilized silt), with section G (plain silt MESL) being the weakest.

Lime-Flyash Design

MacMurdo and Barenberg (1973) developed curves relating compressive strengths and curing degree-days for soil-lime-flyash mixtures. They show one relationship using a 40°F base temperature for curing temperatures below 70°F, and another with a 65°F base for curing temperatures above 70°F. The effects on strength of cyclic temperature fluctuations having a small amplitude, i.e. less than about 5°F at a mean temperature of about 50°F, were found to be minor when compared to those displayed by specimens that had been cured at a constant 50°F. They cited a British study which showed that strength gains in high amplitude cyclic curing conditions (temperature range 50° to 86°F) were nearly double the gains obtained for specimens cured for the same period of time at 68°F. MacMurdo and Barenberg attribute the added strength to accelerated

Table VIII. Average Benkelman beam deflections (in.).

1973-1974 Winter

Section	29 Jan	15 Feb	19 Feb	27 Feb	4 Mar	6 Mar	7 Mar	27 Mar	3 Apr	12 Apr	17 Apr	14 May
F	0.0880	0.0475	0.0455	0.0543	0.0768	0.0783	0.0720	0.0773	0.0745	0.0810	0.0665	0 0920
G	0.0410	0.0033	0.0050	0.0145	0.0400	0.0453	0.0463	0.0553	0.0845	0.1345	0.0763	0.0718
H	0.0220	0.0050	0.0065	0.0105	0.0215	0.0265	0.0320	0.0498	0.0220	0.0383	0.0455	0.0227
I	0.0955	0.0013	0.0013	0.0053	0.0753	--	0.1207	too spongy	too spongy	0.2003	0.2030	0.0835

1974-1975 Winter

Section	8 Nov	8 Jan	1 Feb	24 Feb	28 Feb	6 Mar	18 Mar	24 Mar	11 Apr	15 Apr	16 Apr	23 Apr	5 May
F	0.0667	0.0545	0.0270	0.0590	0.0623	0.0413	0.0694	0.0554	0.0710	0.0688	0.0706	0.0693	0.0755
G	0.0660	0.0128	0.0068	0.0510	0.0493	0.0440	0.0450	0.0874	0.0895	0.0872	0.0848	0.0720	0.0880
H	0.0223	0.0105	0.0065	0.0200	0.0280	0.0147	0.0134	0.0312	0.0338	0.0338	0.0330	0.0308	0.0325
I	0.0120	0.0060	0.0005	0.0525	0.0313	0.0160	0.0086	0.0680	0.0864	0.0586	0.0468	0.0600	0.0525

1975-1976 Winter

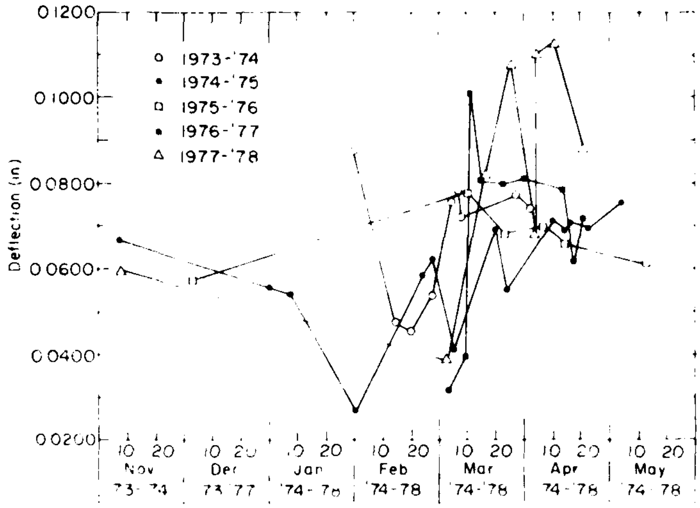
Section	11 Aug	3 Dec	10 Mar	23 Mar	7 Apr	16 Apr	11 May
F	0.0650	0.0575	0.0783	0.0680	0.0698	0.0658	0.0612
G	0.0620	0.0630	0.0583	0.0853	0.0988	0.1005	0.0710
H	0.0480	0.0275	0.0175	0.0305	0.0323	0.0283	0.0198
I	0.0265	0.0395	0.0375	0.0660	0.07[illegible]	0.0678	0.0483

1976-1977 Winter

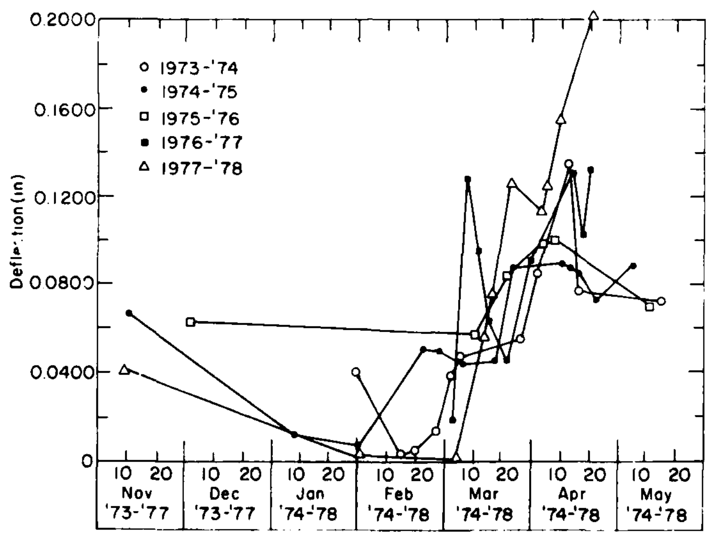
Section	3 Mar	9 Mar	11 Mar	15 Mar	22 Mar	30 Mar	13 Apr	18 Apr	20 Apr	23 Apr
F	0.0320	0.0400	0.1015	0.0805	0.0800	0.0810	0.0785	0.0615	0.0715	0.0648
G	0.0190	0.1284	0.0948	0.0632	0.0473	0.0918	0.1315	0.1025	0.1330	0.0683
H	0.0080	0.0195	0.0170	0.0267	0.0223	0.0358	0.0485	0.0300	0.0710	0.0155
I	0.0120	0.1280	0.1320	0.0625	0.0410	0.0410	-	0.0510	0.0540	0.0380

1977-1978 Winter

Section	8 Nov	31 Mar	3 Mar	16 Mar	23 Mar	4 Apr	5 Apr	10 Apr	19 Apr
F	0.0595	0.0560	0.0390	0.0820	0.1083	0.0683	0.1105	0.1143	0.0875
G	0.0425	0.0033	0.0013	0.0750	0.1270	0.1135	0.1255	0.1560	0.2035
H	0.0230	0.0055	0.0013	0.0028	0.0165	0.0328	0.0283	0.0458	0.0225
I	0.0357	0.0250	0.0010	0.0720	0.0870	0.0530	0.0315	0.1430	0.0370

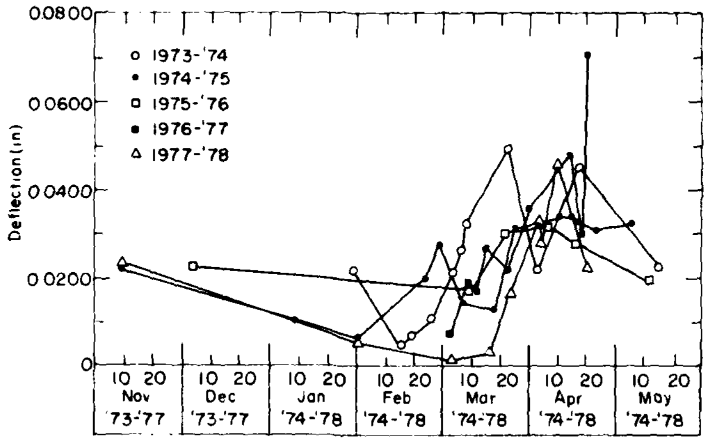


a. Section F (salt MESL).

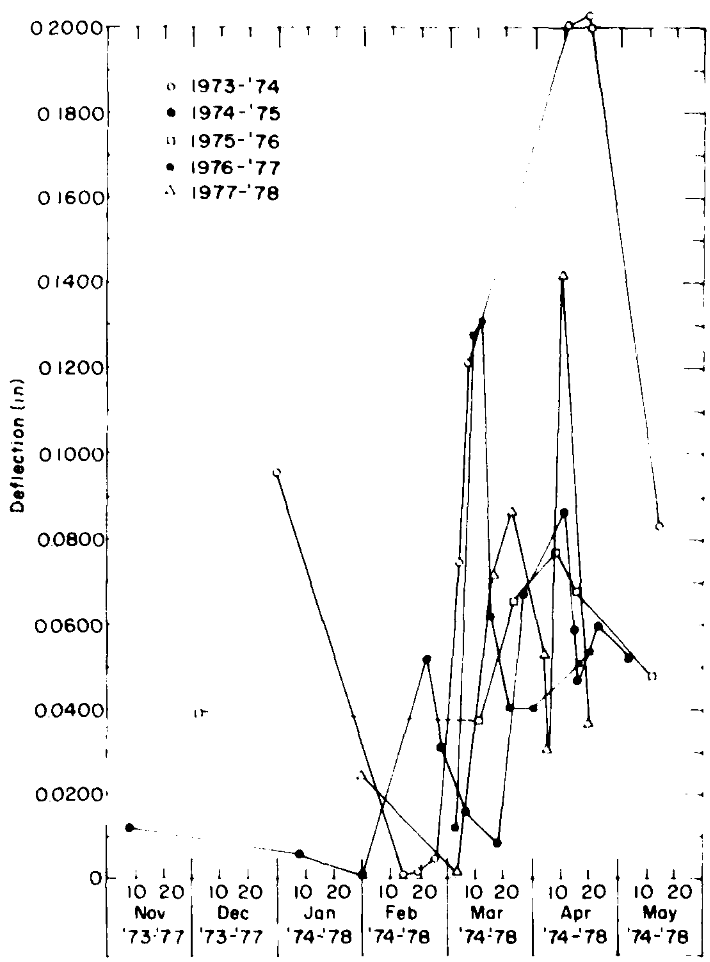


b. Section G (plain MESL).

Figure 9. Benkleman beam test results.



c. Section H (lime-flyash MESL).



d. Section I (unencapsulated lime-flyash).

curing at the higher temperatures in the cycle. These results reinforce the requirement for different base temperatures for various curing conditions.

Placement of the lime-flyash treated silt on the access road was completed on 29 September 1973. Applying the procedure suggested by MacMurdo and Barenberg (1973) to air temperatures subsequent to placement of the material and prior to freezing the mixture, 256°F-days of curing occurred in the material. A base temperature of 40°F was used for these computations. Temperatures within the stabilized layer were not measured during this period because the readout device was not available.

A laboratory program was initiated to determine 1) the extent of curing likely to have occurred prior to freezing in 1973, 2) the degradation of strength due to freeze-thaw cycles, and 3) the extent of autogenous healing of the silt-lime-flyash mixture (Thompson and Dempsey 1969).

Figure 10 shows the compressive strength for various degrees of curing at 120°F. In computing the degree-days of curing, a base of 65°F was used because MacMurdo and Barenberg (1973) recommended the 65°F base for temperatures greater than 70°F. MacMurdo and Barenberg's data indicated that, within a limited range, approximately 6°F-days of curing below 70°F provided strengths equivalent to 1°F-day of curing above 70°F.

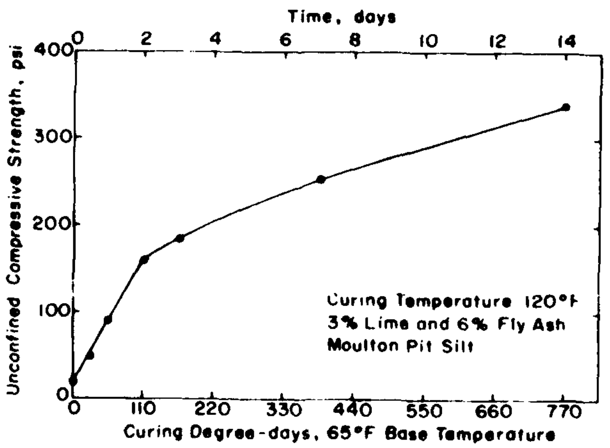


Figure 10. Compressive strength of lime-flyash silt mixture.

Assuming that an equivalent relationship of 6 to 1 is valid for the mixture used on the access road, the 256°F-days of curing below 70°F is approximately equivalent to 43°F-days of curing above 70°F. From Figure 10 it appears that a compressive strength of about 80 psi would have been developed prior to onset of freezing temperatures in December 1973. This value is far below the 200-to 400-psi strength that Thompson (1973) suggests is necessary to preclude frost effects. The MESL construction protected the encapsulated soil from highly adverse ground water conditions. The stabilized encapsulated soil did not require the high strengths needed by a nonprotected soil before freezing and was able to cure the following summer.

Following suggestions by Dempsey and Thompson* additional cores were obtained from section H in June 1974. Since the pH of the in-situ material in section H averaged 12.4, it appeared that residual lime remained in the mixture and some autogenous healing could have taken place. Split-tensile tests were conducted on core sample obtained from station 8+10 of section H in June, following the procedure used by Thompson (1965). Results from the tests showed that the sample had a tensile strength of 3.12 psi and an approximate compressive strength of 24.0 psi. The compressive strength value was determined from the relationship by Thompson (1965) that the split-tensile strength was approximately 13% of the unconfined compressive strength.

Salt Design

In a MESL of fine-grained soil in a cold climate, limited frost action may still be possible. By adding a solute of salt, the freezing point of the soil water is lowered.

Lab studies were conducted to determine the optimum amount and type of salt that could be added to Hanover silt to minimize this frost action.

The lab study and analysis is described in Appendix B.

SUMMARY AND CONCLUSIONS

Current supplies of high quality aggregates for pavement construction in frost areas are being rapidly depleted. Environmental laws are becoming more stringent.

Methods of using lower quality or currently unacceptable soils are needed and membrane-encapsulated soil layer (MESL) construction appears to be an acceptable practice.

MESL test sections were constructed in the fall of 1973 at CRREL

* B.J. Dempsey and M.R. Thompson, personal communication, 1974.

in Hanover, New Hampshire, and their field performance was evaluated for the following five years.

A highly frost-susceptible silt was used as the low quality soil. It was used without additives in one section, mixed with salt in another, and mixed with a lime-flyash mixture in the last section. A fourth section was built using the same lime-flyash silt mixture to determine the benefits of encapsulation.

Polyethylene plastic of 6-mil thickness was used as the bottom membrane and a nonwoven fabric as the surfacing which was made waterproof by spraying top and bottom with emulsified asphalt. These materials are readily available and relatively inexpensive.

As evidenced by soil cores taken in the sections each year, the moisture content within the MESL sections remained relatively constant. The nonencapsulated section, however, showed an almost constant moisture content of over 40% by weight in the top 12 in., even though it was built at the same 17-19% as the MESL sections.

The frost heave within each section was as follows:

0.13 in. within the lime-flyash-stabilized silt MESL
0.22 in. within the salt-stabilized silt MESL
0.37 in. within the plain silt MESL
1.14 in. within the 12-in. lift of nonencapsulated lime-flyash-stabilized silt.

Following are the percentages of the heave occurring within the MESL compared to the total heave caused by the subgrade beneath the MESL:

1.	Lime-flyash MESL	4.3%
2.	Salt MESL	9.6%
3.	Plain MESL	12.8%
4.	Nonencapsulated lime-flyash stabilized silt	28.8%

Benkelman beam deflections showed that:

1. The lime-flyash-stabilized MESL section was the strongest with deflections of 0.02 to 0.04 in.

2. After deflecting up to 0.2 in. the first year, the nonencapsulated lime-flyash section gained strength and was the second strongest section.

3. The salt-stabilized MESL deflected more than any of the other sections during the winter because it did not freeze. During the spring it was weaker than both lime-flyash-stabilized sections.

4. The plain silt MESL section was the weakest section during the spring thaw-weakening period with deflections of up to 0.2 in.

Results from this study showed that

1. The encapsulation kept the soil at a constant moisture content and prevented moisture intrusion.

2. The silt with additives had less frost heave within the MESL than the untreated silt.

3. The nonencapsulated lime-flyash treated layer of silt heaved 8.8 times as much as the lime flyash MESL.

4. Additives increased the strength of the highly frost susceptible material as measured with the Benkelman beam.

In summary, MESL can be constructed and should perform will in cold regions.

The latest information concerning use of MESL in cold regions may be obtained by contacting the Commander and Director of the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), P.O. Box 282, Hanover, N.H. 03755, Autovon 684-3400. Personnel from CRREL are available to provide advice to engineers constructing MESL facilities in cold regions.

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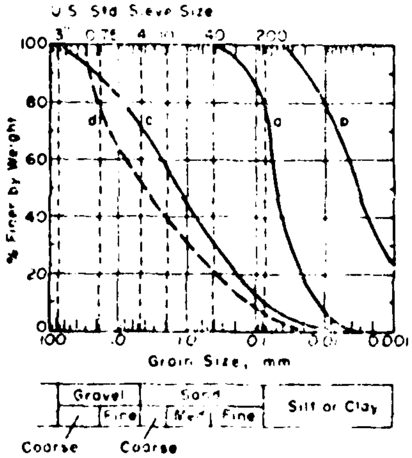
APPENDIX A

MATERIALS GRADATIONS AND CHARACTERISTICS

Access Road



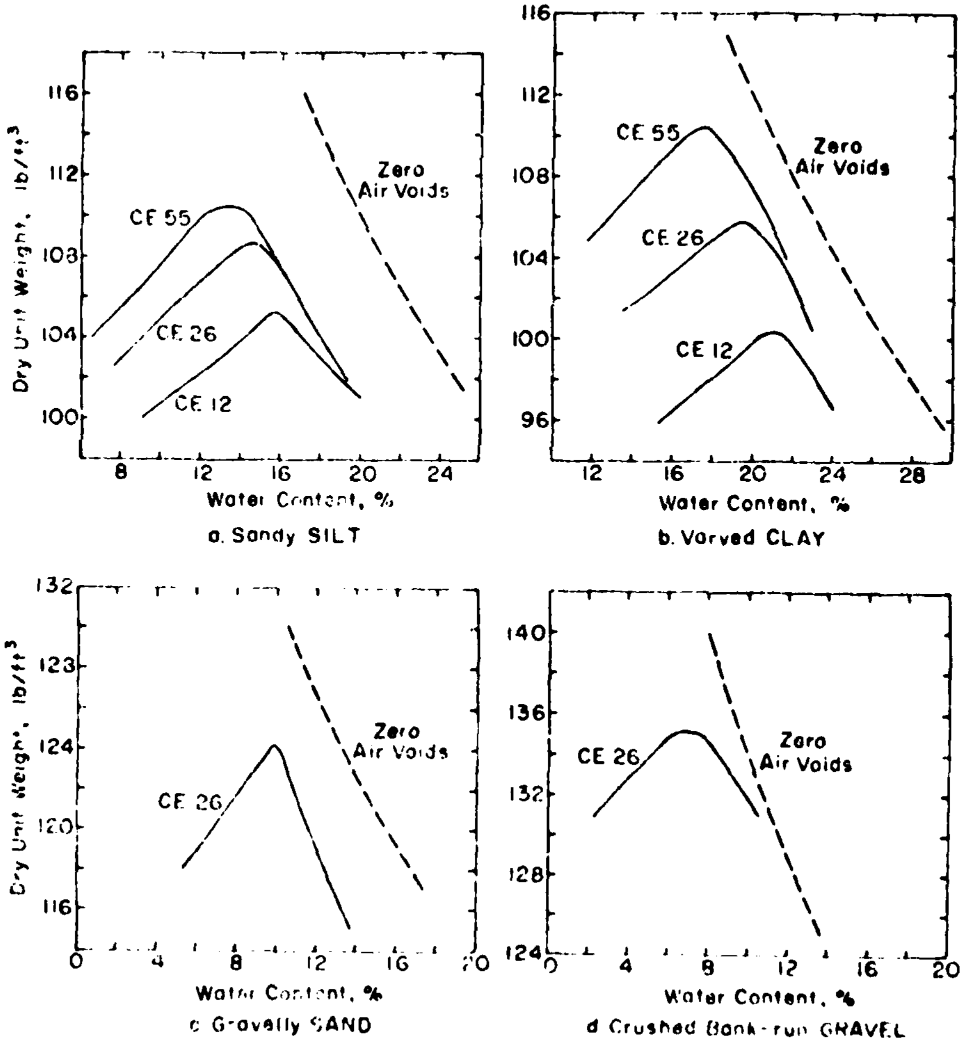


Figure A1. Grain size distribution and compaction curves for subgrade, subbase, and base.

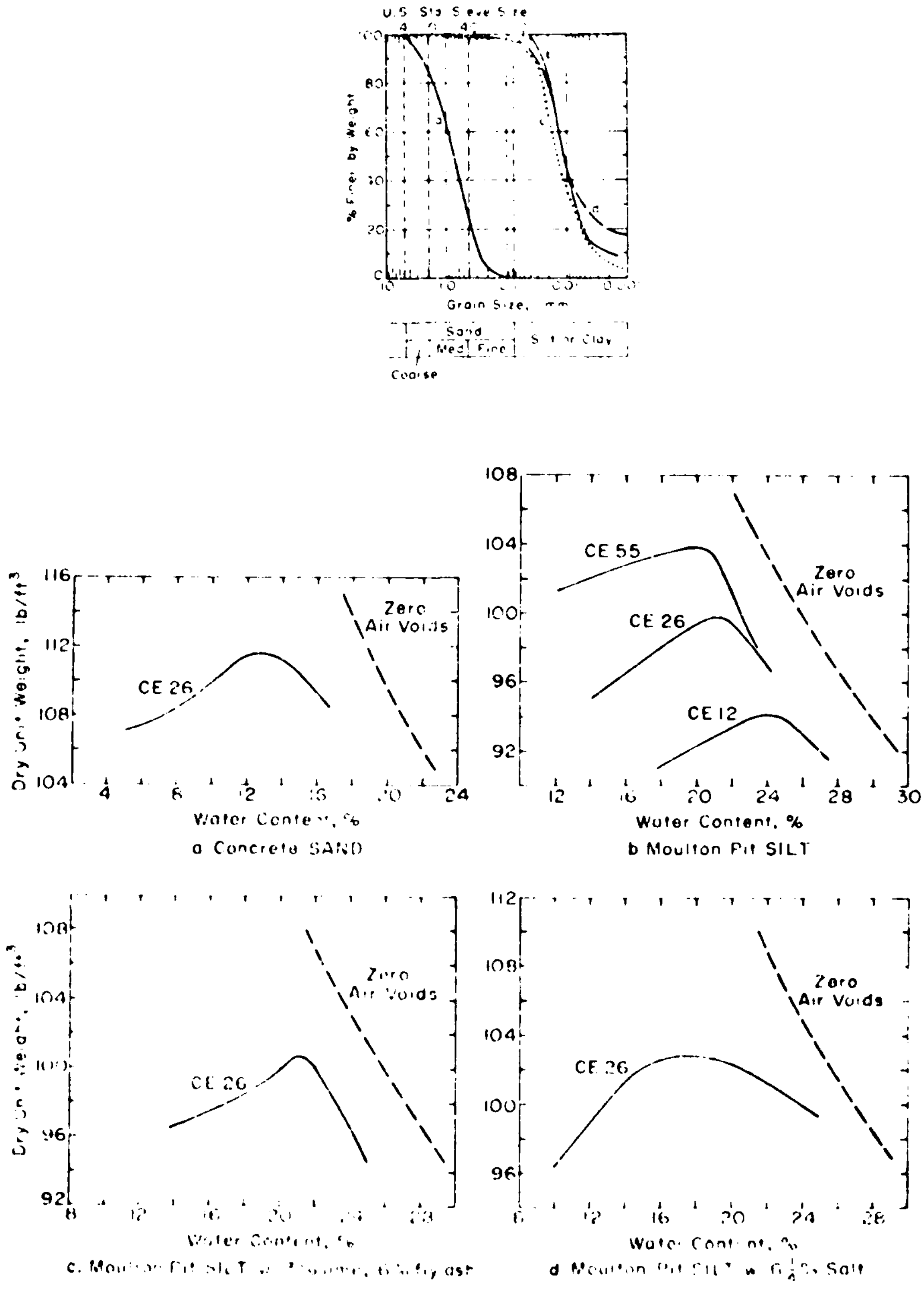


Figure A2. Grain size distribution and compaction curves for concrete sand and encapsulated silt materials.

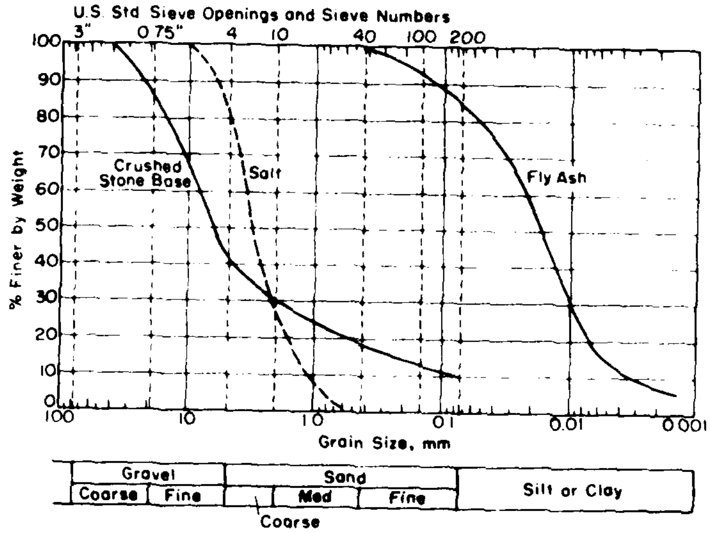


Figure A3. Grain size distribution curves for the crushed stone base, salt and flyash used in the study.

Table AI. Material characteristics.

Soil	Specific gravity	Liquid limit	Plasticity index
CRREL sandy silt	2.68	27	NP
Moulton pit silt	2.75	27	NP
Gravelly sand subbase	2.68	–	–
Concrete sand	2.72	–	–
Sandy gravel	2.73	–	–
Varved clay	2.78	31	7

Table AII. Field CBR values on the subgrade.

Section	Station	Subgrade	CBR
E	4 + 40	fill	38.5
H	8 + 00	cut	24.7

Table AIII. CRREL access road pavement aggregate gradations and asphalt contents.*

Material % passing	16 October 1973: Extr (field) AC 1.5 w/ asbestos†	16 October 1973: Extr (field) AC 1.5	16 October 1973: Extr (field) AC 1.5	16 October 1973: Extr (NED) AC 1.5	16 October 1973: Extr (field) AC 20	16 October 1973: Extr (NED) AC 20	16 October 1973: BMS AC 20	24 October 1973: Bins AC 20	24 October 1973: Extr (field) AC 20	24 October 1973: Extr (field) AC 20/ rubber**	Job-mix requirement
1/2 in.	99.7	99.0	99.5	99.5	99.8	99.8	99.8	99.6	99.9	99.8	99-100
3/8 in.	88.8	91.0	91.9	88.3	90.5	87.5	89.8	89.6	91.1	89.9	87-95
#4	65.8	69.3	70.5	68.5	67.2	65.5	66.8	62.5	65.3	65.8	60-70
#10	45.4	47.9	48.5	47.2	46.6	47.4	47.3	46.5	47.7	45.7	40-48
#20	30.3	33.1	31.6	32.5	31.5	32.6	34.7	35.8	33.4	30.5	26-32
#40	15.6	16.3	16.5	16.8	15.2	15.5	17.2	18.7	17.8	16.5	14-20
#80	7.1	7.4	8.3	8.7	6.2	6.6	6.9	7.4	7.3	7.4	6-11
#200	4.4	4.6	5.6	5.6	3.6	3.6	3.1	3.2	3.7	4.2	2-5
Asphalt content (% by weight)		6.0	6.3	6.4	6.0	6.2	-	-	6.1	6.3††	***

* Tests performed by personnel from the New England Division, Corps of Engineers

† Johns-Manville Asbaltic asbestos fiber (2.5% by wt).

** Goodyear L-170 liquid latex (0.23% by wt).

†† Unable to determine if rubber particles in asphalt or aggregate.

*** Job mix requirements for bituminous material

AC 1.5 w/asbestos	7.15-7.65
AC 1.5	5.45-5.95
AC 20	5.55-6.05
AC 20 w/rubber	5.75-6.25

Job mix requirements for AC 1.5 based on mix design for AC 3.

Table AIV. CRREL access road density and Marshall stability tests*

Date	16 Oct 73	16 Oct. 73	16 Oct. 73	24 Oct. 73	24 Oct. 73
Material	AC 1.5 w/Asb.	AC 1.5	AC 20	AC 20	AC 20 w/rubber
No. of samples	3	2	2	2	2
Stations	7+30 to 9+70	4+40 to 7+30	3+90 to 4+40	2+70 to 3+90	0+00 to 2+70
Temp mold time (°F)	290°	250°	250°	250°	290°
Density (lb/ft^3)	144.1	146.4	144.3	146.8	146.2
Marshall stab. (lb)	744 (1080)+	1060 (1080)	1435 (1940)	2016 (1940)	2204 (2100)
Flow (0.01 in.)	52 (12-27)	7.0 (4.5-10)	6.0 (9-16)	6.5 (9-16)	7.8 (10-16)
Voids (%)	3.4 (1.3-2.3)	3.7 (3-5)	5.4 (3-5)	3.8 (3-5)	4.0 (3-5)
Voids filled with asphalt cement (%)	83.2 (86-94)	78.9 (75-85)	71.1 (75-85)	78.2 (75-85)	77.5 (75-85)
Asphalt Content (%)	7.4** (7.15-7.65)	6.0 (5.45-5.95)	5.9 (5.55-6.05)	5.9 (5.55-6.05)	6.0 (5.75-6.25)

* Tests conducted by personnel from the New England Division, Corps of Engineers

** Asphalt concent of the AC 1.5 with asbestos was assumed because an extraction test could not be performed

\+ Numbers in parenthesis are the job mix requirements.

Appendix B: Salt Use In MESL Construction

by John M. Sayward

Fine, silty soils are usually not suitable for use as a base course or subbase course in roadways because they absorb excessive moisture, have poor bearing strength, and may heave significantly when frozen. They are generally left in place as a subgrade material beneath roads and are overlain by several inches of higher strength granular materials.

Granular soils are often not readily available and are becoming more scarce and expensive. Soils which were previously unsuitable can serve as a base course for light duty roads or as subbase layers in airfields by using the MESL (membrane encapsulated soil layer) system. Before enclosure, such soil is adjusted to 2-4% on the dry side of optimum moisture content (to minimize effects of frost action) and then compacted.

In a MESL layer composed of a fine-grained soil in a cold climate, limited frost action may still be possible. Investigation of the possibility of internal migration of moisture and consequent frost action in a MESL and of added salt as a remedy are among the objectives of the present work.

Frost action in a MESL might be prevented by adding a solute to lower the freezing point of the soil water. Most expedient for this purpose are sodium chloride (NaCl), and calcium chloride ($CaCl_2$). Other salts like magnesium chloride ($MgCl_2$), and ferric chloride ($FeCl_3$), and organic solutes like glycol and alcohol would serve the same purpose but are more expensive. The concept of adding salts to lower the freezing point of soil-water to prevent frost action is not new, but in time salts are leached out of soil. Placing the salted soil in a MESL eliminates most of the problems due to leaching.

The phase equilibrium data in Figure B1 show the freezing point composition relationships for NaCl, $CaCl_2$, and $MgCl_2$. The minimum freezing or "eutectic" points are seen to be

NaCl	-21°C	(-6°F)	at 23% (29.8 lb/100 lb-water)
$CaCl_2$	-51°C	(-59°F)	at 29.8% (42.5 lb/100 lb water)
$MgCl_2$	-34°F	(-28°F)	at 22% (28 lb/100 lb water).

Up to 20% concentration, NaCl and $CaCl_2$ exhibit similar freezing point lowering (although the latter would be a more effective in deicing situations because it has a positive heat of solution, i.e. it evolves heat on contact with water). At 23% (the NaCl eutectic) $CaCl_2$ solution freezes only 3°C (5°F) lower than NaCl solution. Since $CaCl_2$ is about three times as costly, the more available NaCl is obviously preferred (1973 prices were about 3¢ and 1¢/lb, respectively).

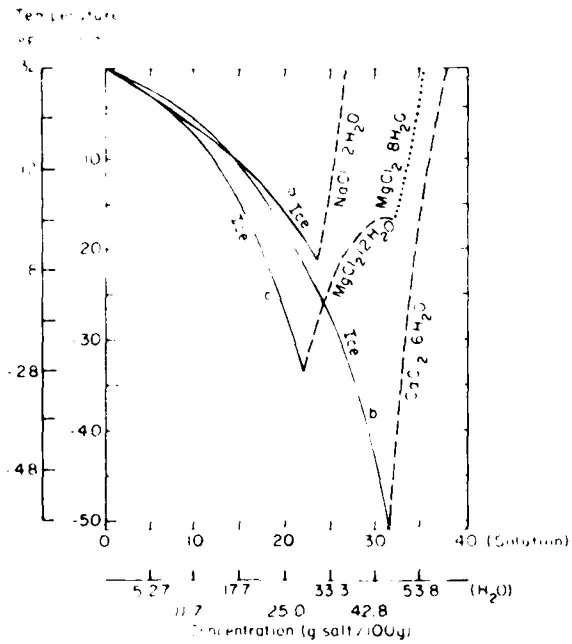


Figure B1. Refrigerating brine phase equilibrium diagram for NaCl (a), $CaCl_2$ (b), and $MgCl_2$ (c). Based on figure in International Critical Tables, Vol. II, p. 327*.

For temperatures lower than -21°C (-6°F), however, $CaCl_2$ or $MgCl_2$ would be the choice to prevent freezing. $MgCl_2$ is seen not only to have a lower eutectic than NaCl but to have a more effective weight ratio than either of the others. Thus, to achieve the NaCl eutectic of -21°C, requires:

NaCl	23.0% (29.8 lb/100 lb water)
$CaCl_2$	21.8% (27.8 lb/100 lb water)
$MgCl_2$	17.5% (26.2 lb/100 lb water)

However, the cost of $MgCl_2$ is appreciably greater than the others, swinging the choice to $CaCl_2$ for lower temperatures.

* Published 1927, McGraw-Hill Book Co., New York, for National Research Council.

Protecting the soil moisture to the temperature of the NaCl eutectic (-21°C, -6°F) was deemed sufficient for this study. The salt requirement here was determined as follows.

For the CE 26 compactive effort, as shown in Figure A2, the Moulton Pit silt had a water content of 18% of dry soil, or 18.5 lb ft^{-3}, for a density of 102.5 lb ft^{-3}. To produce the eutectic in this soil water would require NaCl at 5.37 lb/100 lb soil or 5.52 lb ft^{-3}.

Actually, the compacted soil had a water content of 19% and dry density of 100 lb ft^{-3}. For the NaCl eutectic this requires 5.67 lb/100 lb soil or 5.67 lb ft^{-3}. Adding 10% as a safety factor gives about 6 1/4 lb NaCl per ft^{-3}, the proportion added in the field experiment.

Applying the salt in layers with the soil was considered but was deemed unwise, as diffusion of soil water to produce uniform concentration would likely be very slow. Applying the salt as coarse granules would be undesirable for similar reasons. Actual observations in the lab (by Alan Greatorex of CRREL) showed that large grains of salt compacted with the soil left cavities as the salt dissolved and diffused into the soil water. Such effect was more notable with $CaCl_2$, owing to its greater hygroscopicity and solubility; particles up to 3/8 in. were dissolved and drawn into the soil in a few hours leaving cavities. With NaCl, only particles <1 mm in size left cavities so soon; larger ones dissolved very slowly. Thus, large grains of NaCl might not fully dissolve and diffuse to protect the soil uniformly until considerable time elapsed. Cavities or planes where salt had dissolved away could also make points or planes of instability in the soil. So, use of fine grain salt is preferable.

The present report indicates that samples taken from the salt MESL in February 1974 froze at -10°F rather than the expected -6°F NaCl eutectic point. In absence of analysis or a freezing test before construction, the speculation that another salt contaminant was present seems justified. This probably was present in the salt as received, for technical and decier grade NaCl may contain impurities as shown in Table B1.

Table B1. Commercial salt data (Kaufmann 1960).+

	Percentage of NaCl					
	From salt beds				Oceanic	Solubility at
Component	NY state	Detroit	Louisiana	Kansas	salt	RT (g/100 g H_2O)
NaCl	96.7-98.7	96.4-98.1	99.0-99.2	95.7-97.3	78.8	35.7
$MgCl_2$	to 0.01	0.05	to 0.01	0.25	10.9	52.8
$MgSO_4$				to 0.09	4.7	26.2
$CaSO_4$+	0.5-0.9	0.6-0.9	0.17-0.24	2.2-3.4	3.6	0.18
K_2SO_4					2.5	7.35
MgBr					0.2	9.1
$CaCO_3$+					0.3	0.001
$CaCl_2$	0.2-0.04	0.04-0.05	to 0.01			59.5

+ Note low solubility

As mixtures of chemicals have lower melting points than do the pure components, so solutions of salt mixtures may exhibit lower ice points than similar concentrations of pure salt solutions. Conceivably, capillary effects in silt soil might also have contributed to apparent freezing point lowering.

Appendix C: Design of Lime-Flyash-Silt Mixture Access Road

by

J. Ingersoll

This appendix contains data from unconfined compressive strength tests performed on various lime-flyash-silt mixtures for the CRREL access road. Most of the tests were conducted to develop design percentages of lime and flyash in the silt. Other tests, conducted after the road was constructed, determined the strength gain with increased time of curing at 120°F. The soil used in the road and in these tests is a uniform nonplastic silt obtained from the Moulton pit near the Lebanon Regional Airport in West Lebanon, New Hampshire. The lime-flyash silt mixture was used as an encapsulated base material in section H of the access road and as a nonencapsulated base material in section I.

A mixture of 3% lime and 6% flyash was selected based on an apparent optimum strength determined by CBR tests conducted prior to construction represented by the dashed lines in Figure C1. Unconfined compressive strengths of specimens with various lime-flyash ratios are also plotted in Figure C1. The selected mixture was further tested by obtaining CBR values from 24-hour oven cured (120°F CE-26 moisture-density) samples; Figure C2 shows that values increased from approximately 20% before curing to over 100% after curing.

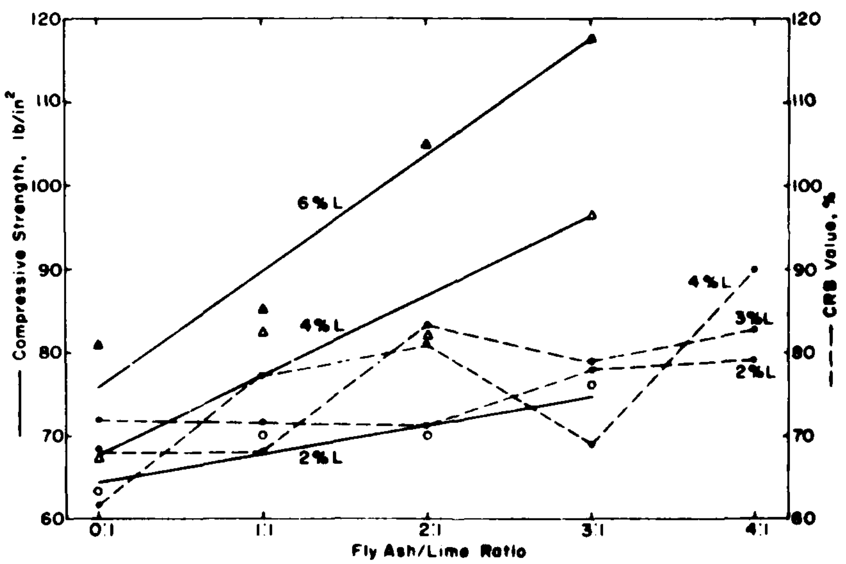


Figure C1. Unconfined compressive strength and CBR values of various lime-flyash ratios with Moulton pit silt.

Another series of unconfined compression tests was run prior to construction using various percentages of lime-flyash molded in the standard Proctor compaction mold. The diameter of these samples was 4.00 in. and the length was 4.59 in. The applicability of results of these tests was questionable because of the less than 2:1 ratio of length to diameter; however, the results are shown in Figure C3.

The second test series was conducted to develop a relationship between the compressive strength and the time of sample curing at 120°F. This relationship should assist in determining the state of the curing process actually occurring in the roadway. Only that portion of the soil which passed the no. 10 sieve was used in test mixtures. The percentages of lime and flyash mixed with the soil were based on the dry weight of the soil. The required amounts of lime and flyash were initially mixed into the wet soil (W = 18%) by hand for approximately 4-5 min, ensuring uniform distribution. After mixing, the lime-flyash and soil were covered to prevent moisture loss and allowed to mellow for one hour before compaction.

The mold used for sample preparation was a split Lucite mold in which four specimens could be compacted simultaneously. The dimensions of the specimens were 2.0 in. in diameter and approximately 4.75 in. in length.

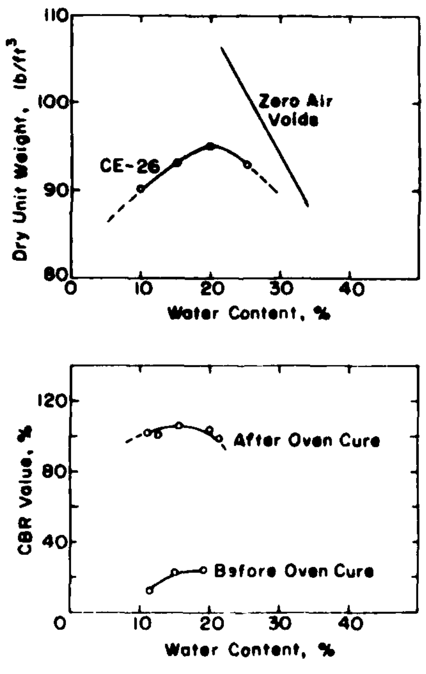


Figure C2. Compaction curve and CBR values for Moulton pit silt with 3% lime and 6% flyash.

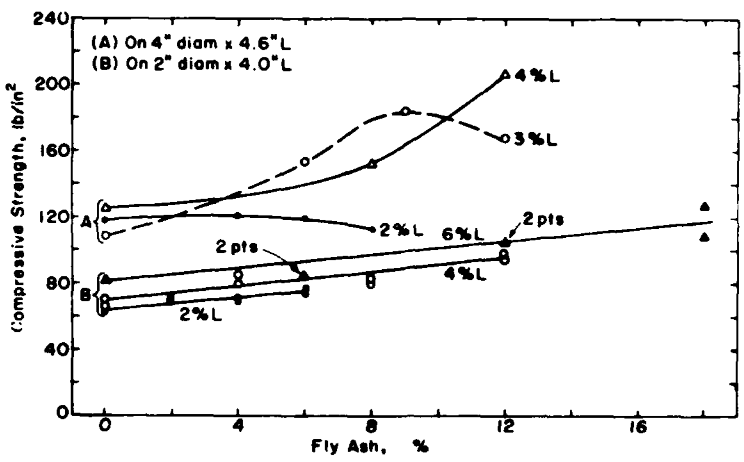


Figure C3. Unconfined compressive strength after curing for 24-hours at 120°F.

Compaction was accomplished using a Harvard miniature compaction hammer with a 40-lb+ spring load. To ensure a uniform density each specimen was compacted in 16 layers, with the number of blows increasing from 35 blows on the bottom to 70 blows on the top. After each second layer the number of blows was increased by 5. The surface of each layer was scarified before placing additional material.

Prior to compaction, the base of the mold was covered with a plastic strip to prevent the soil from adhering. The molds were lightly lubricated with silicone grease to ease extraction. Extraction was accomplished by pressing a piston against the base of the samples using a hydraulic jack. The ejected sample were encased in the split 2.0-in.-diam. mold and the top was trimmed square after removing approximately 1/2 in. of the irregular molded surface. At this point, the lengths of the samples were measured and initial weights were determined. The completed samples were wrapped tightly in individual plastic bags and placed in a 120°F oven for the required curing time. Duplicate samples were molded for each test point and the average strength of the two used. Curing time for these specimens was 24 hours at 120°F.

All samples were tested on a Materials Testing System (MTS) machine at a constant strain rate of 0.05 in. per minute. The machine provided a stress-strain curve for each sample; however, the compressive stress was not corrected for axial strain (except in the unstabilized tests A and B), as most all samples broke between 1.0 and 1.5% strain, introducing only insignificant errors for the purpose of these tests. At the conclusion of testing the samples were weighed for final moisture determination and the failure planes were sketched.

Table CI lists the soil properties, lime-flyash percentage and unconfined compressive strength of each test. Figures C1 and C3 show the relationships of CBR values and compressive strength test results on 4.0-in.-diam x 4.5-in.-long samples and compressive strength results on the conventional test samples with various lime-flyash ratios.

Table CII lists the sample properties for the time-cure vs strength tests (Fig. 10 contains a plot of time of cure vs compressive strength). Figures C2 and C4 contain moisture density curves and CBR values, and grain size distribution curves, respectively.

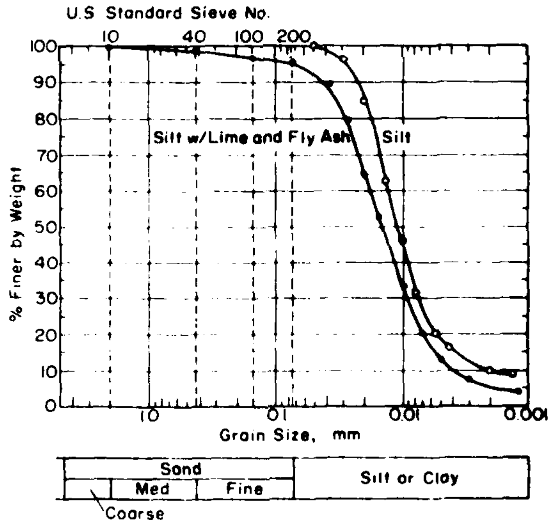


Figure C4. Grain size distribution curves for silt and silt with 3% lime and 6% flyash.

Table CI. First series of compression tests.
(All samples were cured for 24 hours at 120°F.)

Sample	Lime (%)	Flyash (%)	Dry density (lb/ft^3)	Orig % water	Final % water	Comp. strength (psi)	Avg. comp. strength (psi)	Remarks
A	0	0	94.9	19.4	19.3	19.2	18.8	
B	0	0	94.9	19.3	19.3	18.3		
1	-	-	--	--	--	--	--	spoiled
2	2	0	97.1	17.2	15.9	63.8	63.8	
3	4	0	98.2	17.2	16.4	69.6	67.7	
4	4	0	98.0	17.2	16.2	65.7		
5*	6	0	96.7	17.6	15.7	80.6	80.9	
6*	6	0	96.4	17.6	16.1	81.1		
7	2	2	96.3	18.3	16.5	71.4	69.9	
8	2	2	96.4	18.3	16.6	68.3		
9	4	4	94.8	17.4	15.0	85.3	82.3	
10	4	4	94.2	17.4	15.1	79.3		
11*	6	6	94.9	16.6	15.1	84.0	85.0	
12*	6	6	94.6	16.6	15.3	85.9		
13*	2	4	95.3	17.4	16.3	71.7	70.0	
14*	2	4	95.5	17.4	16.1	68.3		
15*	4	8	94.4	16.4	15.5	80.1	81.7	
16*	4	8	94.3	16.4	15.2	83.2		
17	6	12	92.4	15.9	14.5	105.0	105.0	
18	6	12	92.7	15.9	14.5	105.0		
19	2	6	94.8	17.4	16.1	73.5	75.8	
20	2	6	94.7	17.4	15.7	78.0		
21	4	12	92.8	16.1	14.4	94.5	96.5	
22	4	12	93.0	16.1	14.4	98.5		
23	6	18	91.5	14.7	13.2	127.3	117.5	
24	6	18	91.6	14.7	13.3	107.7		

*Indicates test re-run because of curing problems.

Table CII. Second series of compression tests. (Relationship between compressive strength and time of curing at 120°F.)

Test	Dry density (lb/ft^3)	Orig. Water %	Final Water %	Days cured	Comp. strength (psi)	Avg. comp. strength (psi)
12-1	95.4	16.5	15.9	0.5	46.8	47.8
12-2	95.1	16.6	15.9	0.5	48.7	
24-1	95.2	17.4	15.7	1.0	88.2	88.6
24-2	94.4	17.4	15.9	1.0	89.0	
48-1	95.1	16.6	14.9	2.0	157.2	157.9
48-2	94.8	16.6	14.9	2.0	158.6	
72-1	95.4	16.7	15.3	3.0	178.1	183.4
72-2	95.0	16.7	15.3	3.0	188.6	
7-1	96.0	16.7	14.4	7.0	244.0	247.5
7-2	95.3	16.7	14.0	7.0	250.9	
14-1	96.3	16.7	13.0	14.0	341.4	338.8
14-2	96.2	16.7	12.9	14.0	361.1	