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Bulletin 100

Soil Freezing

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Soil Freezing

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Contents

Freezing-and-Thawing Tests on Mixtures of Soil and Calcium Chloride

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Three soils, including a silt, silty clay, and clay-like soil, were used in this study. Specimens were molded using varying percentages of calcium chloride as an admixture and freezing-and-thawing tests were made at freezing temperatures of 24 F. and -18 F. The specimens were permitted to absorb water during the thawing cycles in some of the tests, while in other tests they were not. Further, several specimens were permitted to change in volume while others were not. Unconfined compression, and California bearing-ratio tests were made after various cycles of freezing and thawing.

The results of the study indicated that the greatest percentage loss in strength resulted after one cycle of freezing and thawing. It was found that calcium chloride reduced this loss in strength a small amount, due principally to its ability to lower the freezing temperature of the water in the soil. It was further found that restraining the specimens during the freezing cycles and thus keeping them from changing volume increased the effectiveness of the calcium chloride.

● THE problem of loss in soil bearing under highway and airport pavements in the spring has long been recognized. Research has been turned in that direction by pavement performance surveys made during the spring of the year (1, 2). In this connection, frost affects have been found to manifest themselves in two ways: (1) frost heave, an actual heaving of the soil due to growth of ice lenses, and (2) spring break-up, a condition brought about by the softening of the subgrade immediately under the pavement during the spring. Further, many soils become soft and lose considerable strength upon being subjected to alternate cycles of freezing and thawing.

Research dealing with frost action of soils has been varied and covers both laboratory and field investigations of the mechanics of soil freezing. Work has been done in the field, by means of load-bearing tests, on the loss in strength of the subgrade in the spring of the year. This type of work has been carried out extensively in connection with the Highway Research Board committee on Load Carrying Capacity of Roads as Affected by Frost Action (3).

Research has generally been directed towards the rate and amount of heaving resulting from freezing of soils. Work of this type includes that of Taber (4, 5), Winn and Rutledge (6), Haley and Kaplar (7) and others. Slate (8) reported on the prevention of frost heave by the addition of calcium chloride to soils.

A study was made by Johnson and Lovell (9) to determine the needs for future research in frost action of soils. The results of this study indicated a need for increasing present day knowledge of the technology of freezing and thawing processes in soils.

PURPOSE AND SCOPE

The purpose of this project was to determine (1) the effect of calcium chloride on loss in strength resulting from freezing and thawing soil and (2) the effect of calcium chloride on the rate and amount of moisture movement in soil during freezing and thawing. The scope included testing the three types of soils in the laboratory which are listed in Table 1.

Two of the soils were Illinoian drift materials, the silt representing the A horizon and the silty clay the transition to the B horizon. The third soil represents calcareous Wisconsin drift occurring in slight topographic depressions.

The effect of freezing and thawing on the strength of the soils was evaluated by means of unconfined-compression tests as well as California bearing-ratio tests. Calcium chloride was added to the soils in percentages up to two percent by weight.

Freezing and thawing conditions were limited to freezing and thawing of the sam-

ples from all directions. Two freezing temperatures were used: 24 F. and -18 F. In addition, several tests were made on samples kept at room temperature during the cycle in which other specimens were frozen. The variables of moisture and density were introduced by molding specimens under the following conditions: (1) 100 percent maximum Proctor density and 100 percent optimum moisture content; (2) 95 percent maximum Proctor density and 100 percent optimum moisture content; and (3) 100 percent maximum Proctor density and 75 percent optimum moisture content.

TABLE 1

SOILS TESTED

Designation and Derivation	L. L %	P. L %	Proctor Weight pcf.	C E	Frost Class
Vigo silty clay - Illinoian Drift	32	18	114 2		F_3
Brookston Clay - Wisconsin Drift	50	30	105 0		F_3
Vigo Silt - Illinoian Drift	29	25	103.4		F_4

In addition, several tests were made in which the samples were not permitted to absorb water during the thawing portion of the cycle and some in which the specimens were completely or partially restrained curing the cycles.

PROCEDURES

Since it was desirable to determine the rate and amount of moisture movement in soil, as well as the strength characteristics of the mixtures of soil and calcium chloride, procedures were devised whereby both determinations could be made on one soil specimen.

Molding

The procedures adopted for making the compressive-strength specimens were relatively simple and consisted of compacting the soil, in cylinders 2 inches in diameter by 4½ inches in height, to a predetermined density and at a known moisture content. This was done by weighing out a known amount of wet soil, placing the entire amount of soil in the 2-inch-diameter cylinder, and then compacting it until it was exactly 4½ inches in height. Figure 1 shows a view of the disassembled cylinder. The 2-by-4½-inch specimen was used to facilitate the performance of unconfined compression tests. The ratio of h/d for this size specimen is 2¼.

After molding, each specimen was encased in a thin rubber membrane, with both ends of the specimen left exposed to the air. Duplicate specimens were made, in most cases, and the results averaged to insure consistent results.

The California bearing-ratio specimens were molded at the standard Proctor optimum moisture content as determined by previous compaction tests. The height, weight, and number of blows of the compaction hammer were adjusted so that the energy imparted to the soil during compaction was approximately equal to that used in the standard Proctor test (12, 400 ft.-lb. per cu. ft.).

Freezing and Thawing

Two types of freezing-and-thawing tests were made. The first was one in which the specimens were frozen radially with the reduced temperature applied to all faces of the specimen. The specimens were allowed to absorb water from the bottom during the thawing period of the cycle.

Twenty-four hours of reduced temperatures and 24 hours of thawing constituted one cycle of freeze and thaw. After both the freezing and thawing portions of the cycle were completed, the specimens were measured and weighed. Following the completion of the desired number of cycles, the specimens were tested for their unconfined compressive strengths and then cut into ½-inch slices for moisture determinations.

The other type of test used was similar to the first, except that the specimens were completely restrained during the cycles (see Figure 2). Water absorption was again permitted during the freezing portion of the cycle.

In addition to the above tests, several specimens in both groups were tested with no free water available for capillary saturation.

To determine the effect of freezing temperatures on strength, tests were made on samples frozen at +24 F., -18 F., and on others which were not frozen at all, but kept at room temperature during the normal freezing cycle.

The C. B. R. specimens were exposed to weathering cycles much the same as were the unconfined-compression samples.

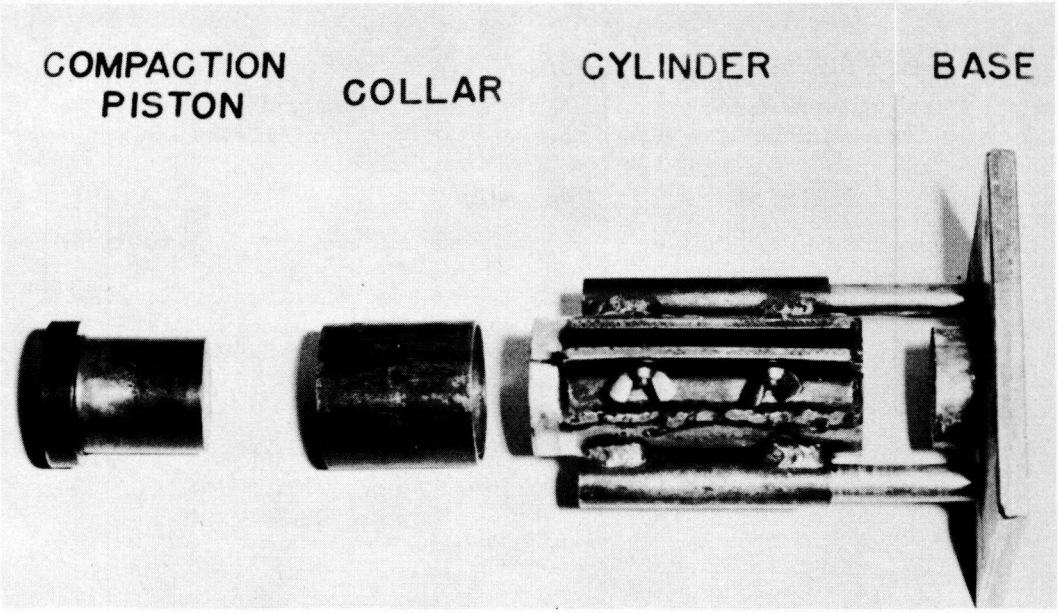

Figure 1. View of compaction mold. The base, cylinder, collar, and piston were machined so that when completely assembled and in contact, the length of the inside of the cylinder was exactly 4½ inches.

The specimens were, however, placed in water during the thawing portion of the sample and permitted to take up water from the top as well as the bottom. A 17-lb. surcharge weight was kept on these specimens at all times during the cycles.

Unconfined Compression Tests

After the desired number of freezing-and-thawing cycles were completed, the specimens were tested for unconfined compression. The specimens were loaded at a rate of 0.05 inches deformation per minute. The ultimate compressive strength was taken either as the peak unit load, or the unit load at 20 percent strain, whichever occurred first, divided by the corrected cross-sectional area.

Since the specimens were permitted to absorb water from the bottom during thawing, some of the specimens were quite soft

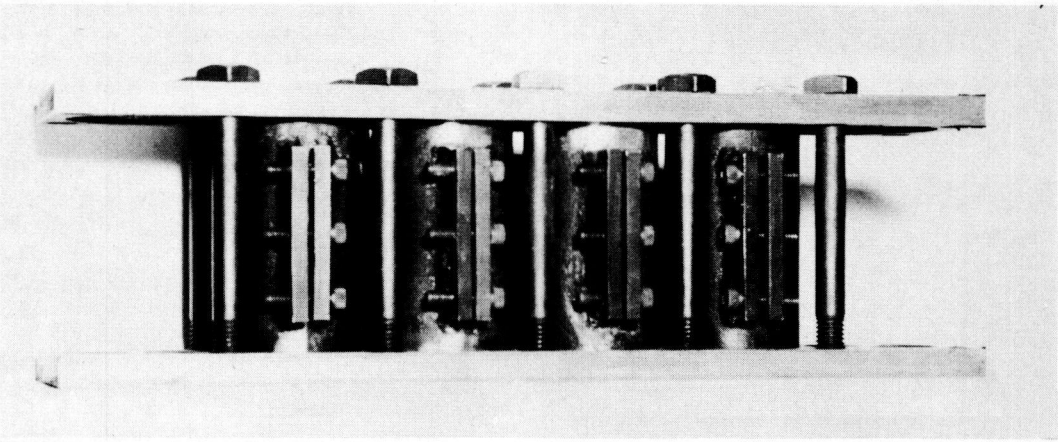

Figure 2. Set of molds used for weathering tests on restrained specimens.

TABLE 2

EFFECT OF SOIL TYPE AND CALCIUM CHLORIDE ON SOIL STRENGTH AFTER ONE CYCLE OF FREEZING AND THAWING

(Freezing temperature = 24 F., all specimens molded at optimum moisture and to 100 percent Proctor density)

Soil Type	$CaCl_2$	Moist	Density	Strength	Loss in Strength from initial	$CaCl_2$ Concentration before freeze	after thaw
	%	%	pcf.	psi.	%	% weight water	
	0	21.2	103.1	6.2	82	0	0
Brookston Clay	1	21.1	104.4	9.0	72	5.45	4.74
	2	19.6	104.6	16.0	50	10.10	10.20
Vigo Silty Clay	0	17.0	111.0	5.0	82	0	0
	1	16.1	112.0	10.5	67	7.10	6.20
	2	13.7	116.5	22.0	21	14.30	14.60
Vigo Silt	0	22.7	100.7	13.8	54	0	0
	2	21.5	101.5	17.0	37	12.3	9.30

in the lower half but relatively firm in the upper half. This was particularly true of those specimens treated with calcium chloride. As a result, failure generally resulted in bulging in the lower half of the specimen.

California Bearing-Ratio Test

The C. B. R. tests were made in the conventional manner using a piston having an end area of 3 sq. in. Loads were applied at a uniform rate of 0.05 inches per minute. The bearing ratio was calculated for each 0.1-inch penetration up to 0.5 inch of penetration; these values were then averaged. A surcharge weighing 17 lb. was used during the test.

Moisture Determinations

After both the unconfined-compression and C. B. R. tests, the specimens were cut in slices ½ inch high. A portion of each of these slices was then weighed and dried to constant weight and reweighed for moisture determinations. An attempt was made to make these slices exactly ½ inch in height so that the variation of dry density with depth, as well as moisture content, could be determined.

RESULTS

Most of the results presented in this section are averaged results. The interrelationship of all the variables was apparent from the start, and even though an attempt has been made to evaluate each variable independently, this interrelationship should be kept in mind.

Effect of Soil Type

The texture of a soil will influence its strength characteristics in a number of different ways: First, soil texture, density, and moisture holding capacity are all interrelated. A fine-grained soil will generally have a lower density (for a given compactive effort) than a coarse-grained soil with a resulting higher moisture content after saturation. Second, the rate at which water will rise in soil by capillary action will also vary with soil type. The amount of water which will freeze in a soil at a given temperature is largely unknown, but this too varies with soil type; or more basically with the amount of adsorbed water, grain shape, etc.

The results of the tests made on the three soils under conditions of 100 percent standard Proctor density and molded

at optimum moisture content are shown in Figure 3. These curves reveal that when a freezing temperature of 24 F. was used the calcium chloride was most effective when incorporated in the silty clay.

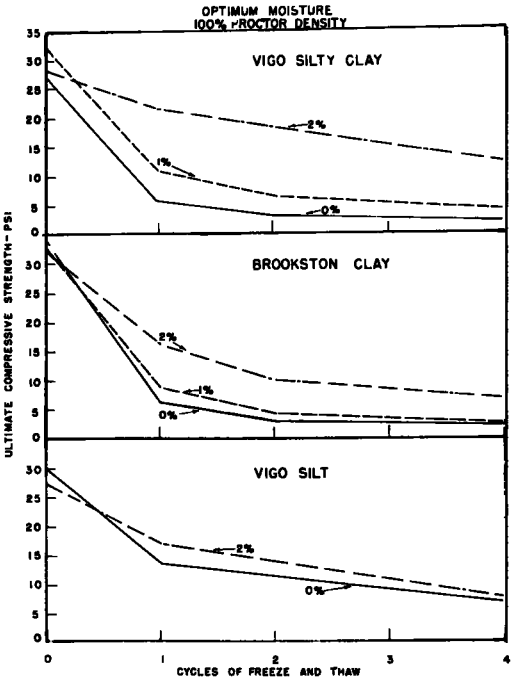

Figure 3. Effect of soil type on unconfined compressive strength.

The effect of calcium chloride at lower temperatures is discussed in the next section. Each of the soils had initial compressive strengths of comparable magnitude, ranging from 27 psi. to 34 psi. However at the end of the first cycle of freezing and thawing, the unconfined compressive strengths of all Brookston clay and Vigo silt samples were below 17 psi. In contrast, the unconfined compressive strength of the specimen of silty clay and 2 percent of calcium chloride was equal to 22 psi.

It should be kept in mind that each specimen was allowed to absorb water during the thawing cycle, thus the adverse effect of excess moisture was most apparent in those soils which absorbed water most readily. Furthermore, since the effect of the calcium chloride is principally that of lowering the freezing temperature of the soil water, the concentration of the salt in the soil water materially affected the soil strength after freezing.

A summary of the test data after one cycle of freeze and thaw is also shown in Table 2. Observation of this table brings out that a correlation exists between the concentration of the calcium chloride in the water and soil strength. The soils with the highest concentration of calcium chloride in the water (silty clay, 2 percent $CaCl_2$ mix; 14.6 percent calcium chloride in the water) lost the least amount of strength after freezing and thawing. This particular soil also had the highest initial density and lowest initial moisture content of all the soils. Other factors, such as mineralogical content of the soils, no doubt also affected the results, but to an unknown extent.

It was noted throughout the testing program that the greatest loss percentagewise in strength resulted after the first cycle of freeze and thaw. Additional losses in strength resulted after further cycles, but these were small compared to that after the first cycle. The loss in strength after the first cycle was generally 50 percent or more of the original strength.

TABLE 3

EFFECT OF FREEZING TEMPERATURE ON COMPRESSIVE STRENGTH AFTER ONE CYCLE OF FREEZING AND THAWING

Soil Type	$CaCl_2$	Freeze at 24 F		Freeze at -18 F.	
		Strength	Loss in Strength from Initial	Strength	Loss in Strength from Initial
	%	psi	%	psi	%
	0	6.2	82	9.0	74
Brookston Clay	1	9.0	72	9.0	72
	2	16.0	50	9.0	72
Vigo Silty Clay	0	5.0	82	0.5	98
	1	10 5	67	0.5	99
	2	22.0	21	0 5	98
Vigo Silt	0	13 8	54	14.0	53
	2	17 0	37	14.8	46

Effect of Moisture and Density

Figure 4 shows variation of compressive strength, average moisture content, and average dry density during freezing and thawing for the Vigo silty clay.

The data of this figure help explain the reason for the specimens of calcium chloride showing less loss of strength after freezing and thawing than the untreated specimens. Here is shown again in the upper curves, the strength curves for the Vigo silty clay as were shown in Figure 3. The lower and center curves indicate aver-

age dry density and average moisture content for the specimens. It will be seen that for this soil the treated specimens had the greatest initial dry densities, and that as the specimens progressed through the freezing-and-thawing cycles, the dry densities of the untreated specimens fell off rapidly with corresponding increases in moisture content. This was felt to be due chiefly to the chemical lowering of the freezing point of the soil water, thus minimizing expansion during freezing.

In this connection, several tests were made to determine the freezing point of the soil water with and without the calcium chloride admixture. These tests consisted of depressing the soil temperature and by taking readings at various intervals, determining the temperature at which the soil-water froze as indicated by the horizontal portion of the curve of temperature versus time. These tests indicated that when 2 percent of calcium chloride was incorporated in the soil, the freezing point of the soil was depressed below 24 F.

Since unconfined-compression testing techniques were used to evaluate the stability of the soils, several pertinent factors should be pointed out that affected the test results. First, at the time of test,

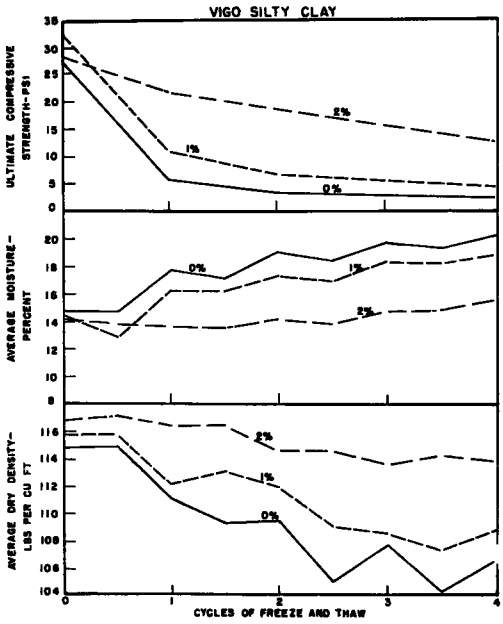

Figure 4. Variation of unconfined compressive strength, average moisture content, and average dry density with cycles of freezing and thawing.

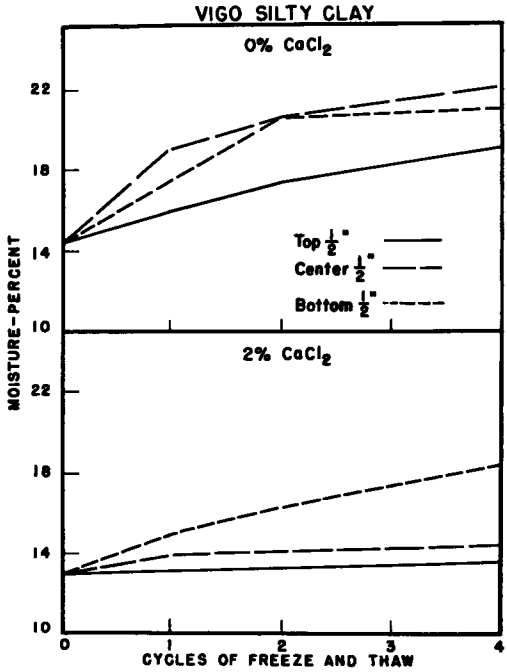

Figure 5. Variation of moisture content with depth of specimen.

after the appropriate cycle of freezing and thawing, the moisture by and large was concentrated in the lower portions of the specimens, as indicated in Figure 5. The density of the specimens likewise varied from bottom to top with lower densities being recorded at the bottom.

The curves of Figure 5 indicate variation of moisture content with depth in the Vigo silty clay samples after one cycle of freezing and thawing. These determinations were made by cutting the unconfined-compression samples into thin slices after the test, measuring and weighing them, and determining their moisture contents. This graph shows moisture content at only three locations: top, center, and bottom ½ inch. It will be noted that the moisture was by and large near the bottom of the specimens. However, it will be seen that the moisture content of the untreated specimen was more nearly the same throughout the height of the specimen than was the one treated with 2 percent of calcium chloride. In other words, not only was the average moisture content of the untreated specimen higher than the treated, but there was less variation from top to bottom in the untreated specimen.

Figure 6 shows the effect of initial moisture content and dry density on the test

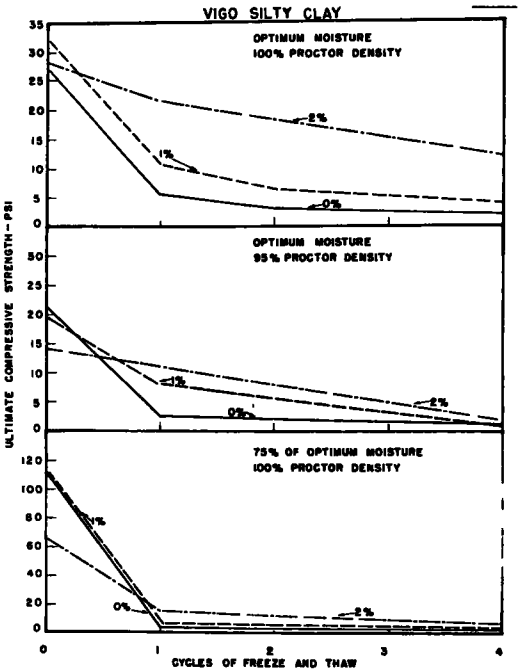

Figure 6. Effect of initial moisture content and dry density on unconfined compressive strength.

percent of the optimum. It can be seen that lowering either the initial density or moisture content almost obliterated the effect of the chemical. It is further seen that the specimens molded at low moisture contents lost practically all their strength after one cycle of freezing and thawing, even though the initial strengths were very high.

Effect of Freezing Temperatures

Figure 7 shows the results of tests made on the Vigo silty clay using a freezing temperature of -18 F., as well as 26 F. and for samples which were not frozen. Observation of the curves in Figure 7 and the data in Table 3 indicates that the calcium chloride was most effective when the specimens were frozen at relatively high temperatures. However, in the case of the silty soil, no great difference was apparent between strengths of the treated specimens after freezing at -18 F. and after freezing at +24 F. The silty clay soil showed greater loss in strength at

TABLE 4

EFFECT OF LENGTH OF CYCLE (BROOKSTON CLAY)

	$CaCl_2$ (%)	After One Cycle of One Day Freeze at +24 F and One Day Thaw on Porous Stone		After One Cycle of One Week Freeze at +24 F and One Week Thaw on Porous Stone		After One Cycle of One Day Freeze at -18 F. and One Day Thaw on Porous Stone	
		After Freeze	After Thaw	After Freeze	After Thaw	After Freeze	After Thaw
Density (#ft.^{3})	0	104.5	103.1	105.5	105.4	105 8	102.4
	1	105 0	104 4	106 6	103.0	106.5	103.5
	2	106 5	104.6	105.6	100 2	105.5	102.0
Moisture (%)	0	19 9	21 2	18.3	20 0	18.8	20.7
	1	18.8	21.1	18.1	21 4	18 5	20.0
	2	19.4	19 6	17.7	22.0	19 4	20.5
Strength (psi)	0	-	6 2	-	7 0	-	9.0
	1	-	9.0	-	7.5	-	9 0
	2	-	16.0	-	9.8	-	9.0
Loss in Strength from initial (%)	0	-	82	-	79	-	74
	1	-	72	-	77	-	72
	2	-	50	-	69	-	72

results. The upper curves are the same as those previously shown, while the lower and center curves indicate unconfined-compression strength versus cycles of freezing and thawing for specimens compacted at lower initial densities and moisture contents. In the center curve the specimens were compacted at optimum moisture content, but to just 95 percent of maximum Proctor density, while the specimens illustrated by the lower curves were compacted to 100 percent of maximum Proctor density, but at a moisture content of 75

-18 F., as did the clay-like soil.

The effect of soil texture on the effectiveness of the chemical was obliterated when a freezing temperature of -18 F. was used. In the previous section, it was brought out that the admixture was most effective when used in the silty clay material (depressed temperature 24 F.). However, when freezing temperatures of -18 F. were used, the unconfined compressive strengths were practically the same, for a particular soil, regardless of the quantity of admixture that was used.

TABLE 5

EFFECT OF WATER ABSORPTION (BROOKSTON CLAY)

Freezing Temp.	$CaCl_2$	Initial Strength	Loss in Unconfined Compressive Strength from Initial					
			After One Cycle			After Four Cycles		
			Water Absorp.	No Water Absorp	Diff *	Water Absorp.	No Water Absorp	Diff *
	(%)	psi	psi.	psi.	psi.	psi	psi	psi.
+ 24 F.	0	34.0	27.8	13 0	14 8	31 8	17 0	14 8
	2	32.0	16 0	2 0	14.0	25 0	2 2	22.8
-18 F.	0	34 0	25.0	17 0	8 0	31.0	26 2	4 8
	2	32.0	23.0	3 0	20.0	28 0	20 0	8.0

* Loss in strength that can be attributed to water absorption

Effect of Length of Freezing Time

Several tests were performed in which the freezing and the thawing phases were each increased to one week. The results of these tests are shown in Table 4. The effect of subjecting the soil-calcium chloride mixture to 24 F. for one week was practically the same as freezing it for a shorter period of time at -18 F. The treated specimens lost as much as 10 percent more of their initial strengths when freezing temperatures of 24 F. were used in the 1-week cycle as compared to the daily cycle.

A portion of this loss must be attributed to the longer period of thaw on the porous stones. At the end of one cycle the treated specimens in the 1-week cycle generally soaked up more water during the thawing cycle, and their densities were correspondingly lower after thawing than those whose cycle was 1-day freeze and 1-day thaw.

Effect of Water Absorption

During the thawing portion of the freeze-and-thaw cycles, water was permitted to flow into the majority of the specimens by capillary action. Since it was felt that a large portion of the decrease in soil strength was caused by this water absorption, an attempt was made to isolate this variable by sealing several specimens in paraffin, thereby not permitting them to lose or gain any moisture during the complete cycle. These specimens were first encased in thin rubber membranes, and then the ends were sealed with wax. Thus, expansion was permitted, even though no moisture changes took place.

Results of these tests are shown in Table 5. The data clearly illustrate that considerable loss in strength can be attributed to water absorption. When 24 F. was used with no water absorption, the compressive strength of the 2-percent-calcium-chloride specimen was 2.2 psi. less than the initial. It is seen also that when frozen at -18 F. these specimens showed appreciable losses in strength out to as far as four cycles. This is in contrast to the test results obtained when water absorption was permitted and by far the greatest reduction in strength resulted after the first cycle.

The data in this table also give a clue to the relative effects of soil freezing and

TABLE 6

EFFECT OF RESTRAINT ON COMPRESSIVE STRENGTH (BROOKSTON CLAY)

Freezing Temp	$CaCl_2$	Initial* Strength Restrained	Initial* Strength No Rest.	Loss in Compressive Strength from Initial			
				After One Cycle		After Four Cycles	
				Rest	No Rest.	Rest	No Rest.
F	(%)	psi	psi.	(%)	(%)	(%)	(%)
				With Water Absorption During Thawing			
+ 24 F	0	33 0	34 0	9	82	30	93
	2	27 0	32 0	0	50	0	78
-18 F	0	33 0	34.0	28	74	42	91
	2	27 0	32 0	22	72	37	87
				With No Water Absorption During Thawing			
+ 24 F	0	33 0	34 0	15	38	24	50
	2	27 0	32 0	-33*	6	-26**	7
-18 F.	0	33.0	34.0	0	50	0	77
	2	27 0	32 0	-4**	9	-26**	62

* Different size specimens were used accounting for difference in strength
** Minus sign indicates increase in strength

moisture changes. The losses in strength given in Columns 5 and 8 of Table 5 can be attributed to change in structure and density brought about by freezing, while those in Columns 6 and 9 are due to moisture absorption. It will be noted that the treated specimens lost little strength during freezing and thawing when no moisture was permitted to enter the specimen.

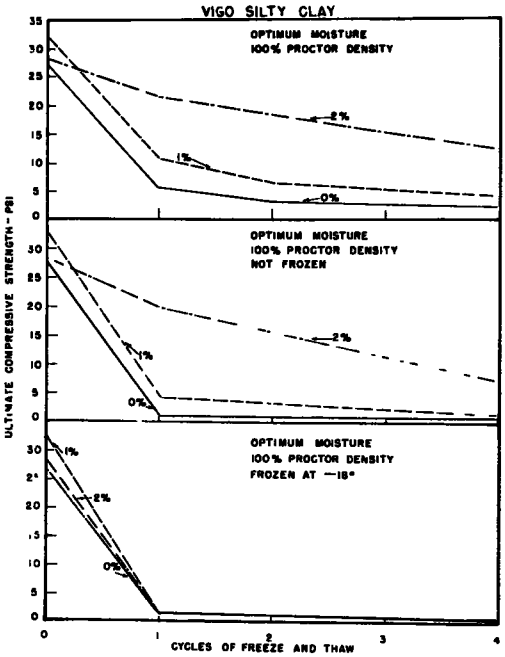

Figure 7. Effect of temperature on unconfined compressive strength.

Effect of Restraint During Freezing and Thawing

To further isolate the variable of moisture and density on strength, weathering tests were made on samples of the Brookston clay when in a completely confined condition. The specimens were encased in metal cylinders and held firmly by means of bolts. It was assumed that the plates on either end of the specimens were sufficiently rigid to make expansion negligible. The specimens were removed from these cylinders and tested in unconfined compression at the end of the weathering cycles.

The results of these tests are shown in Table 6. The results were slightly erratic, inasmuch as the treated specimens (which were in a restrained condition and not permitted to absorb water during the weathering cycles) showed increases in strength with cycles of freeze and thaw.

In general, when the specimens were permitted to absorb water they picked up just sufficient water to completely saturate the specimens. None of the specimens lost a great amount of strength during the cycles, the average being about 25 percent of the initial.

It will be noted in Table 6 that the effectiveness of the calcium chloride was materially increased by restraining the specimens. It is felt, however, that more tests need be made before any definite conclusions can be reached.

Effect of Calcium Chloride and Freezing and Thawing on C. B. R.

Only a limited number of C. B. R. tests were made, the results of which are shown in Figure 8. These tests were made using standard procedures utilizing a 17-lb. surcharge weight. The end point was to determine the effect of partial restraint during freezing and thawing. When the specimens were tested in a soaked condition, the calcium chloride increased the soil-penetration resistance after freezing and thawing, but not to any great extent.

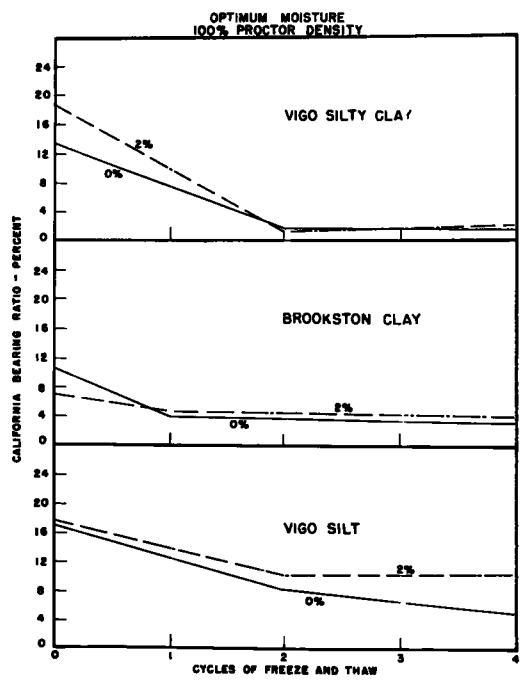

Figure 8. Effect of soil type on California Bearing Ratio.

The results indicated that the calcium chloride was effective in decreasing loss in C. B. R. in some cases, but that partial restraint of the surcharge did not have the effect of full restraint.

SUMMARY

The effects of freezing and thawing on soil strength are variable, depending on soil texture, density and changes in density, moisture content and changes in moisture content, duration of freezing, freezing temperature, and degree of confinement. The degree to which calcium chloride affected the strength of the soils differed for all of the above-mentioned variables and by restricting or controlling any one of the variables the effectiveness of the calcium chloride was increased.

The effectiveness of the calcium chloride was found to be principally due to its property of lowering the freezing point of the soil moisture. When the specimens were frozen at very low temperatures the salt was ineffective. Likewise, increasing the freezing time decreased the effectiveness of the calcium chloride.

The results and conclusions of the study are summarized below. They apply only to the conditions of test imposed in this study. The variable of permanence of the chloride, formation of ice lenses, and prevention of frost heaving, were not included in the study.

1. Soil texture influenced the effectiveness of the calcium chloride in preventing loss in strength after freezing and thawing. Moisture content and density played a very important role in this as did the actual concentration of the chloride in the soil water. For the three soils tested the calcium chloride was most effective in the silty clay soil.

2. When the specimens were molded at optimum moisture content and 100 percent of Proctor standard density, the calcium chloride reduced the loss in unconfined compressive strength resulting from the weathering cycles. This reduction was as much as 60 percent of the soil's initial strength.

3. Calcium chloride was found to be most effective at relatively high freezing temperatures. The principal effect of the chloride was that brought about by lowering the freezing point of the soil water.

When the specimens of soil and calcium chloride were frozen at relatively high temperatures for long periods of time, the effect was similar to that of freezing them at low temperatures for a short period of time.

5. Water absorption effected the test results appreciably. A larger portion of the reduction in strength of the specimens was attributed to water absorption rather than to the freezing temperature. However, in the case of the raw soil, the actual freezing of the soil had about the same effect as the water absorption.

6. Restraining the soil specimens, and thus preventing expansion, increased the effectiveness of the calcium chloride materially.

7. The calcium chloride was effective in decreasing loss in C. B. R. after freezing and thawing. The effect of partial restraint was not nearly as pronounced as that of total restraint on soil strength.

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Discussion

ROBERT E. PYNE, Assistant Maintenance Engineer, Massachusetts Department of Public Works—In the fall of 1947 as a part of a reconstruction project on Route 116 in the town of South Hadley, an experimental section using calcium chloride as a deterrent to frost heaves was incorporated in the design. This research project was under the supervision of J. E. Lawrence, former maintenance engineer of the Massachusetts Department of Public Works and member of the Committee on Frost Heave and Frost Action in Soil of the Highway Research Board.

A plan and cross-section of the test section is attached. The northerly side of the road was the location of the former roadway which was completely removed to subgrade and a foot of permeable gravel was placed on the subgrade. The pavement consisted of 3½ inches of crushed stone bound with sand under a wearing course consisting of 2½ inches of crushed stone penetrated with bitumen, bound with keystone and sealed with bitumen and peastone. On the southerly side of the road new fill was placed to permit widening.

On top of the compacted fill calcium chloride was placed between: Station 3+0 and 4+0, 2¼ lb. $CaCl_2$ per sq. yd.; Station 4+0 and 5+0, 9 lb. $CaCl_2$ per sq. yd.; Station 5+0 and 6+0, 36 lb. $CaCl_2$ per sq. yd.; Station 6+0 and 7+0, 18 lb. $CaCl_2$ per sq. yd.; and Station 7+0 and 8+0, 4½ lb. $CaCl_2$ per sq. yd.

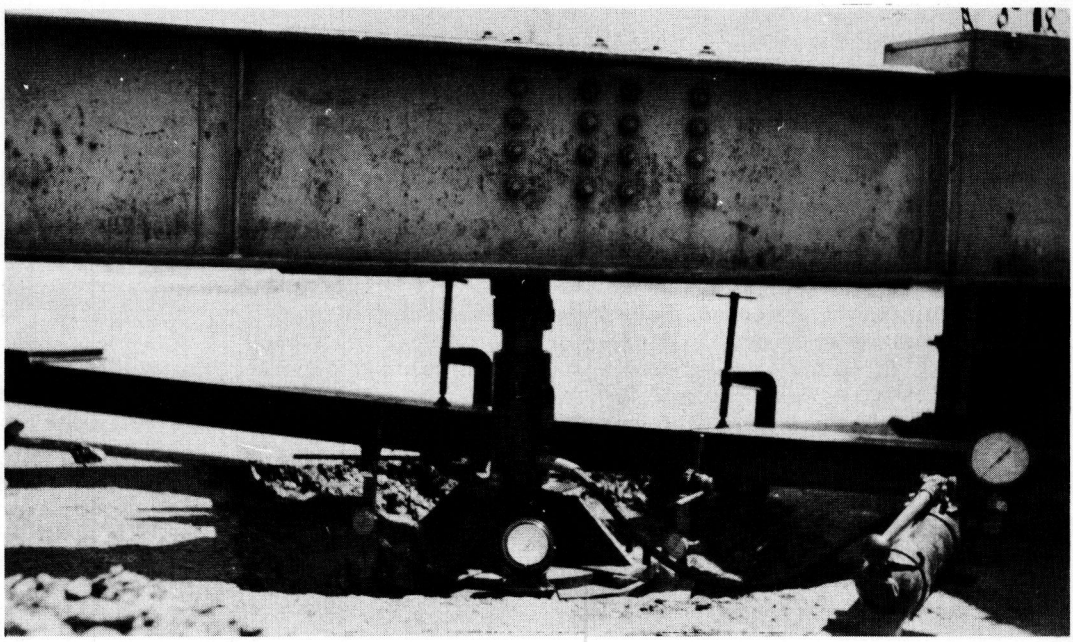


Figure A. Closeup of laboratory bearing-power equipment on South Hadley test.

The location of the project was selected because of its proximity to the Connecticut River and the resultant large volume of silt deposits which would tend to cause a maximum frost action. Generally, the subgrade soils consisted of sandy silts in the A-3 class.

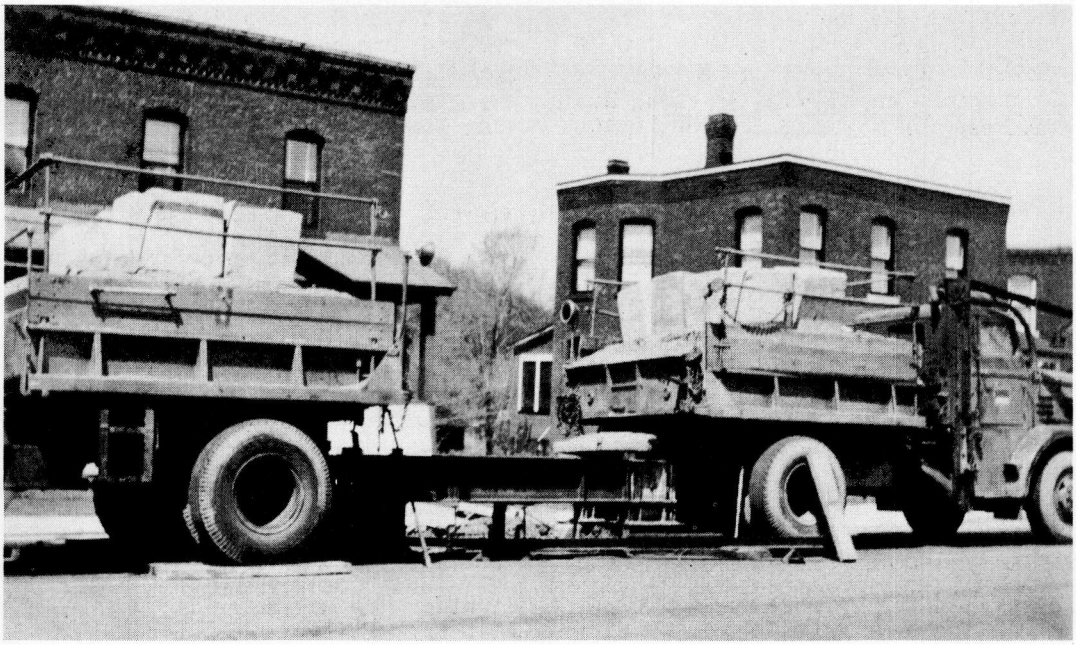


Figure B. Laboratory bearing-power equipment, showing position of loaded trucks.

On top of this treated subgrade the same construction was used as on the northerly side; that is, 1 foot of gravel, 3½ inches of sand-bound stone and 2½ inches of penetrated stone.

The project was completed very late in the fall and the top course was penetrated during a cold spell which caused considerable ravelling in the winter months, necessitating a surface treatment in January of 1948.

During the first winter following construction, two sets of levels were taken during a protracted cold spell from January 8 to February 3, there being but two cycles during this period. The third set of levels was taken on March 24 in more-moderate weather with a total of 22 cycles during the 7-week period. As indicated in the attached chart of frost movements, there was considerable variation between Station 1+50 and Station 3+0 on the southerly side which was untreated as compared with the treated section from Station 3+0 to Station 8+0.

It was noted on February 25, 1948, that a longitudinal crack appeared between Stations 3+0 and 8+0, approximately 1.5 feet to the left of the treated area. Between Station 3+0 and 7+0 on the treated

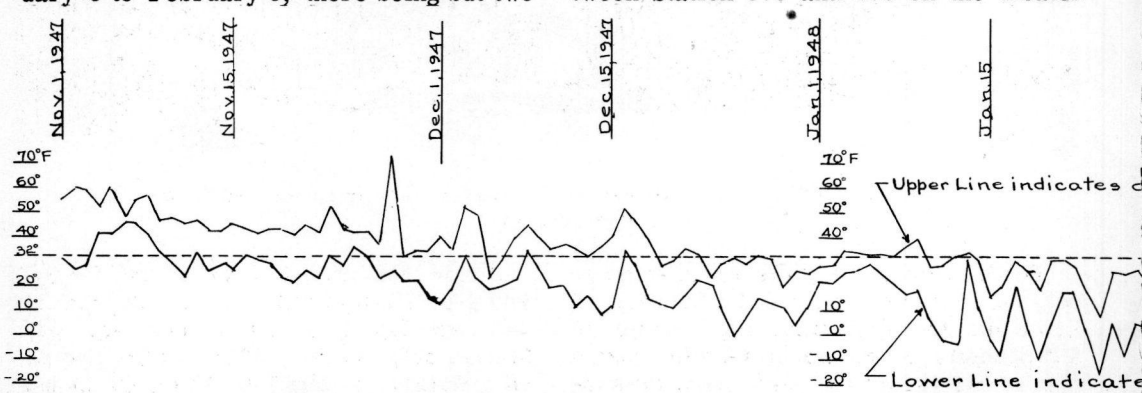


Figure C. Daily temperature chart for Newton Str

TABLE A

GRADATION AND LIMITS FOR SOILS FROM NEWTON STREET, SO. HADLEY, MASS.

(Samples taken from 2.0 to 4.0 feet down at each station)

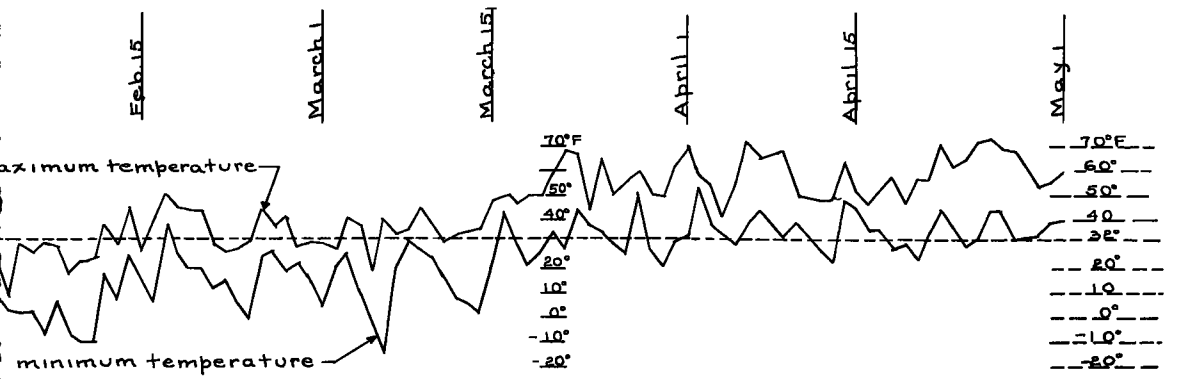
			Station				
			7+50	6+50	3+50	4+50	5+50
Gradation							
		Ret. 1½-in.	-	25.4	-	0.0	0.0
Pass.	1½-in.	1-in.	-	0.0	19.7	13.5	0.0
	1-in.	¾-in.	-	5.3	4.0	0.0	0.0
	¾-in.	½-in.	-	0.8	1.1	0.0	0.0
	½-in.	⅜-in.	-	-	3.5	-	-
	½-in.	No. 4	0.0	3.6	-	0.0	1.1
	⅜-in.	No. 4	-	-	0.9	-	-
	No. 4	No. 10	0.4	2.3	6.8	0.0	0.4
	No. 10	No. 20	1.2	8.5	6.1	0.6	0.4
	No. 20	No. 40	0.6	11.8	10.9	1.0	0.3
	No. 40	No. 80	3.4	15.1	9.5	3.1	0.6
	No. 80	No. 200	3.4	18.1	1.0	5.7	1.1
	No. 200	Pan	91.0	9.1	36.5	76.1	96.1
Limits							
Plastic Limit			26.9	None	11.7	18.3	21.2
Plastic Index			9.5	N.P.	4.3	5.3	7.1
Liquid Limit			17.4	15.6	16.0	23.6	28.3

side, the frost action was approximately 25 percent of that on the untreated area. The results between Station 7+0 and 8+0 were somewhat spotty but generally showed less frost action on the treated side.

This first winter proved to be an abnormally cold one with more than average snow fall. The frost depths as noted by public utility companies varied from 3.5 to 5.0 feet in depth under the roadway, whereas in the areas adjacent to the highway (covered with snow the entire winter) there was an average of 1 foot of frost. Attached to this report is a chart of Meteorological observations for the period from December 1947 through March 1948.

Field Bearing Tests

These tests were run with a 30-inch-diameter plate on the surface of the 12-inch gravel layer. The attached tabulation indicates bearing power at 0.20-inch Deflection in pounds per square inch. The average bearing power of all stations for an individual series of tests, or of any station for all series of tests, indicate that the treated area has higher bearing power than the untreated areas.



h Hadley, from November 1, 1947, to May 1, 1948.

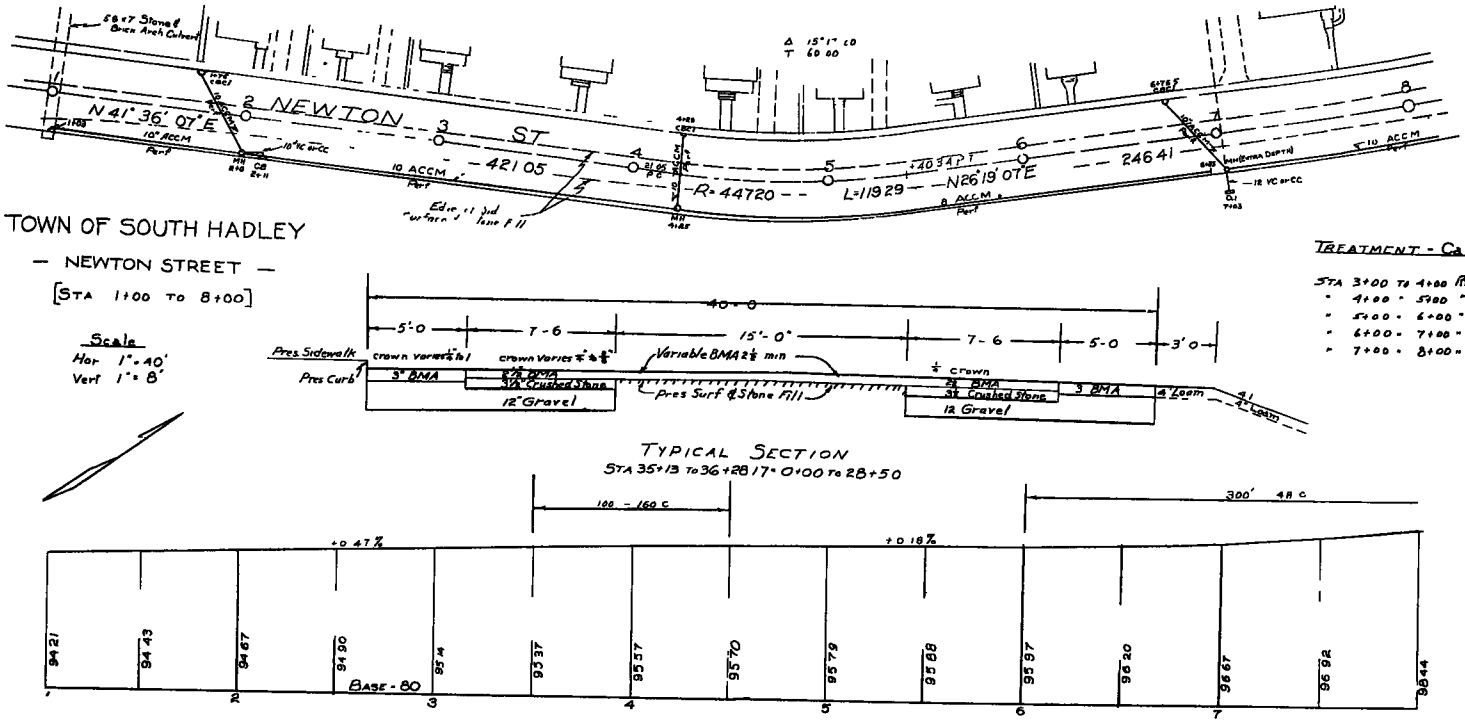

TOWN OF SOUTH HADLEY
— NEWTON STREET —
[STA 1+00 TO 8+00]
Scale
Hor 1" = 40'
Vert 1" = 8'
NEWTON ST
N41° 36' 01" E
N26° 19' 01" E
R = 44720
L = 11929
Δ 15° 17' 00"
T 60 00
10 ACCM
12 VC or CC
Pres Sidewalk
Pres Curb
3" BMA
12" Gravel
Variable BMA 2½ min
Pres Surf & Stone Fill
3 BMA
4" Loam
TYPICAL SECTION
STA 35+13 TO 36+28 17" 0+00 TO 28+50
+0.47%
+0.18%
BASE - 80
TREATMENT - CaCl₂
STA 3+00 TO 4+00 PER -2 5/8
" 4+00 " 5+00 " - 9 5/54
" 5+00 " 6+00 " - 36 5/54
" 6+00 " 7+00 " - 18 5/54
" 7+00 " 8+00 " - 4 5/54

Figure D.

TABLE B

EXPERIMENTAL SECTION

SOUTH HADLEY, MASS. - AUTO ROUTE 116

Determination of Cl computed to $CaCl_2$

Sta. #3 - 3+50 R 2.25 lbs. $CaCl_2$ per sq. yd.				Sta. #2 - 4+50 R 9 lbs. $CaCl_2$ per sq. yd.				Sta. #1 - 5+50 R 36 lbs. $CaCl_2$ per sq. yd.			
7/2/48		1/23/50		7/2/48		1/23/50		7/2/48		1/23/50	
Depth	% $CaCl_2$	Depth	% $CaCl_2$	Depth	% $CaCl_2$	Depth	% $CaCl_2$	Depth	% $CaCl_2$	Depth	% $CaCl_2$
2.33'	0.00196	3.0'	0.023	2.33	0.00246	2.5'	0.018	2.33	0.00222	1.5'	0.009
		4.0'	0.016			3.5'	0.018			2.75'	0.023
		5.0'	0.022			4.5'	0.020			3.5'	0.040
		6.0'	0.011			5.5'	0.022			4.5'	0.031
		7.0'	0.011			6.5'	0.024			5.5'	0.201
						7.5'	0.002			6.5'	0.944
						8.5'	0.002			7.5'	0.363
						9.5'	0.003			8.5'	0.142
						10.5'	0.002			9.5'	0.027
						11.5'	0.004			10.5'	0.020
						12.5'	0.003			11.5'	0.011
						13.5'	0.002			12.5'	0.004
										13.5'	0.006
										14.5'	0.011

Calcium-Chloride Content

Tests of the amount of calcium chloride in the soil samples gave variable results. Initial soil samples tested for chloride content were taken in July 1948 at the edge of the pavement and at a depth of 28 inches. The results showed that considerable leaching action had taken place at the southerly edge of the pavement.

Samples taken in July 1949; January 1950; July 1951; and January 1954 were from the test pits for bearing power tests located in the center of the treated pavement. Test results are attached.

All test data was taken and compiled under the supervision of A. V. Bratt, testing engineer.

Conclusions

Based upon a study of data accumulated over the past six years during which this project has been under observation, the following general conclusions are made:

Calcium chloride, when used to reduce frost heave of pavements by treatment of the subgrade soil, does have beneficial results as long as the chemical is not dissipated.

The effective life of a calcium chloride treatment as used on this project depends upon permeability of the subgrade soil, drainage conditions, and tightness of the

TABLE C

SUMMARY OF FIELD BEARING LISTS

TEST DATE	STATION 3+50		STATION 4+50		STATION 5+50	
	Left 11 25' Untreated Subgrade	Right 11 25' 2¼# $CaCl_2$ per sq. yd.	Left 11.25' Untreated Subgrade	Right 11 25' 9# $CaCl_2$ per sq yd	Left 11 25' Untreated Subgrade	Right 11.25' 36# $CaCl_2$ per sq. yd.
November 1948	51 0# sq in	68 5# sq. in.	48 0# sq. in	64 0# sq. in	48.5# sq in.	65.5# sq in
April 1949	51.8# sq. in.	59.5# sq in	28 0# sq. in.	53.3# sq in.	47 0# sq. in.	71 5# sq. in.
June 1950	66 5# sq. in.	111 0# sq in.	59.0# sq. in.	100.5# sq. in	54 5# sq in	89.5# sq. in.

TABLE D

CHART OF MOVEMENTS IN NEWTON STREET, SOUTH HADLEY, SURFACE IN RELATION TO ELEVATIONS TAKEN MAY 26th 1948

	January 8					February 3					March 24				
Station	20	10	0	10	20	20	10	0	10	20	20	10	0	10	20
1+50	+ 03	-.01	-.01	+.06	+ 15	+.07	+ 05	+ 05	+.12	+.14	-.01	+.04	00	+ 10	+.06
2+00	00	+ 03	+ 03	+ 07	+.08	+ 10	+.07	+ 05	+ 07	+ 04	+ 04	+ 08	+.02	+.06	+ 07
2+50	00	00	+.01	+.05	+ 12	+ 01	+ 09	+ 06	+ 06	+ 18	+ 03	+ 10	+ 02	+ 08	+ 21
3+00	+ 01	- 02	+ 01	+ 07	+ 01	+ 07	+ 07	+ 08	+.09	- 01	+.01	+ 07	+ 02	+ 09	+ 09
				2¼ lb											
3+50	- 03	+.34	+ 01	+ 01	-.01	- 02	+ 11	+ 09	+ 05	+ 04	.00	+.05	+ 01	+ 01	- 03
				Sq Yd.											
4+00	+ 06	+ 02	+ 04	+.03	00	+ 03	+ 08	+.08	+ 09	+.11	+.04	+.04	+ 02	+ 03	+ 13
				9 lb											
4+50	+ 06	- 07	+.01	+ 05	+ 03	+ 04	+.02	+ 12	+ 05	+ 02	+ 06	- 04	+.11	+ 02	+.08
				Sq Yd.											
5+00	-.01	- 03	+ 10	+ 03	- 02	+ 03	+.09	+.18	+ 04	00	+ 04	+ 03	+ 16	+.01	+.02
				36 lb											
5+50	+.05	+ 05	+ 01	+.02	+ 02	+ 04	+ 26	+ 08	+ 08	+.06	+.05	+ 08	+ 05	+ 01	+ 06
				Sq Yd											
6+00	+ 04	+.04	+ 06	+.04	00	+ 01	+ 27	+ 13	+.01	00	+.06	+.06	+ 08	+ 04	+.08
				18 lb.											
6+50	+ 06	+.16	+ 02	+ 01	+ 01	- 01	+ 18	+ 06	00	+.03	+ 10	+ 06	+ 08	+ 03	+ 02
				Sq Yd.											
7+00	+ 07	+.05	+ 02	+.07	+ 01	+.12	+.22	+ 08	+ 04	+ 02	+ 05	+ 06	+ 08	+ 05	+ 02
				4½ lb											
7+50	-.02	+.01	- 02	+.02	+ 02	+ 01	+ 05	+ 05	+ 06	+.02	+.03	+.05	+ 04	+.04	+ 02
				Sq. Yd											
8+00	.00	+ 03	- 01	00	+ 05	- 01	.00	+ 04	+ 07	-.02	+ 02	-.02	+ 04	+ 07	+ 08
8+50	+ 01	+.04	-.01	+.03	+ 02	- 01	+.01	+ 03	+.05	.00	+.02	+.01	+.02	+ 09	+.02
9+00	+.02	+.03	+.04	+ 08	-.01	-.01	+.09	+.08	+.03	- 06	+ 03	+.02	+.06	+ 09	-.01
9+50	+.04	+.03	+.05	+.06	+.06	-.02	+.04	+.01	+.04	-.01	+.06	+.04	+.04	+.05	+.05

Treated Area

pavement against percolation of surface water. On this project, not only was the penetration pavement sufficiently pervious to permit surface water to percolate through into the subgrade, but there was also a horizontal flow of water which caused additional leaching of the calcium chloride from the subgrade, thereby destroying its effectiveness. The effective life on this project was about 3 years. No calcium chloride remained in the subgrade after 6 years.

The proper application of calcium chloride for most-effective results varies with the individual project conditions. On this project 9 lb. of calcium chloride per square yard of subgrade surface seemed most effective.

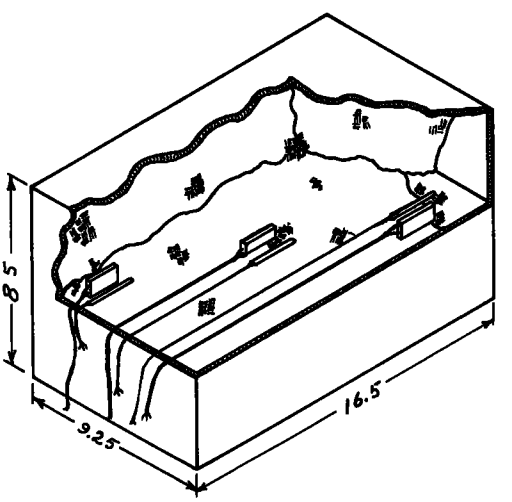
Frost Determination by Electrical Resistance

ELMER F. ROWLAND, Engineering Aide,
LEWIS H. STOLZY, Soil Conservationist, and
GEORGE A. CRABB, Jr., Hydraulic Engineer
Division of Soils Research, U. S. Department of Agriculture

A preliminary report on a new method of determination of frost in the soil profile in situ. The method involves interpretation of electrical resistances obtained with a soil-moisture block and their associated soil temperatures, a method developed at the Michigan Hydrologic Research Station as a result of correlations noted from daily field records of soil moisture and temperatures. A hypothesis relating these phenomena was formulated and was substantiated by field and laboratory studies. Under certain conditions an abrupt 200-ohm increase in the electrical resistance of the plaster-of-paris moisture unit was a satisfactory indicator of the commencement of crystallization of moisture. Test instrumentation and graphic results of the laboratory tests are illustrated.

● THIS paper presents a simple method for determining the commencement of initial freezing, or water crystallization, of soil moisture in situ. The method was developed at the Michigan Hydrologic Research Station as a result of correlations noted between field observations of frost penetration depths and soil-moisture readings made with moisture blocks. This method of frost determination, which utilizes principles of soil-moisture measurement by variations in the electrical resistance of porous units buried in the profile, is discussed in preliminary form, because of the wide interest evidenced by agronomists and by highway and agricultural engineers.

The electrical-resistance method of determining the moisture content of soil utilizes a porous unit (containing a pair of equidistant electrodes) buried in the soil profile. The moisture content of the unit varies with that of the soil in which it is embedded, and the electrical resistance between the electrodes varies with the moisture. Temperature also affects the resistance of the unit, but this effect may be corrected if the temperature is known. A number of types of such moisture-sensitive units are in use today, primarily differentiated by the type of dielectric used in construction. The most-widely used units utilize plaster-of-paris, nylon, fiberglass, and similar materials as a dielectric. The plaster-of-paris unit developed by Bouyoucos and Mick[1] is used at the Michigan Hydrologic Research Station for the daily recording of soil moisture at several locations and depths. This method has given indications that it might be used to tell when initial freezing of the soil moisture takes place. Various authors have indicated that frozen soil causes abnormally high resistance, but none of them



FROM LEFT TO RIGHT
BLOCK 128 TEMPERATURE UNIT PLACED DIRECTLY BELOW THE MOISTURE BLOCK.
BLOCK 129 TEMPERATURE UNIT AND BLOCK IN THE CENTER. UNITS PLACED SIDE-BY-SIDE.
BLOCK 130 TEMPERATURE UNIT PLACED DIRECTLY ABOVE THE MOISTURE BLOCK.
NOTE: DIMENSIONS ARE OF THE SOIL SAMPLE. THERMOMETERS ARE 0.5 INCH FROM THE MOISTURE BLOCKS

Figure 1. Schematic drawing of soil sample used in second freezing experiment, showing instrumentation. Notice that measuring units are so placed as to show progress of freezing from all sides towards the center.

[1] Bouyoucos, G. J, and Mick, A. M, an electrical method for the continuous measurement of soil moisture under field conditions Michigan Agricultural Experiment Station Technical Bulletin 172, 1940.

have presented data to pinpoint the commencement of the freezing process.

Comparison of resistance readings from the daily soil moisture studies with manually determined frost-penetration depths in the cultivated watersheds indicated that there was a probable correlation between soil temperatures, moisture-block resistances, and presence of frost in the profile. A preliminary field study correlating these factors was carried on during the winter of 1951—52, and showed a high degree of correlation. Data used in this preliminary field study consisted of the daily soil temperature and moisture readings from the hydrologic investigations, supplemented by actual frost-penetration data obtained by probing with a Veihmeyer soil-sampler. Analysis of this data provided the basis for an hypothesis regarding the determination of the commencement of crystallization.

This hypothesis predicated that an abrupt 200-ohm increase in the electrical resistance of a soil-moisture block under stable temperature conditions in the range of 30 to 32 F., followed by an increase in resistance and a slowly decreasing or static soil temperature, indicates the commencement of crystallization of soil moisture. An elementary laboratory study was designed to substantiate this hypothesis.

In this study, seven similar samples of soil from the cultivated watersheds were placed in baskets of hardware cloth and frozen in a conventional ice-cream-storage cabinet. The cabinet was thermostatically controlled to maintain a box temperature ranging from 8 to 11 F. Plaster-of-paris moisture units were centrally placed in three of the soil samples, while resistance thermometers were similarly placed in three other samples. A seventh sample was not instrumented, but was utilized for physical determination of frost penetration by probing. The seven samples were installed in the freezer and the cooling process commenced. Temperatures and resistances of the soil samples, box temperatures, and the probed depth of frost penetration were noted at frequent intervals throughout several freezing-and-thawing cycles.

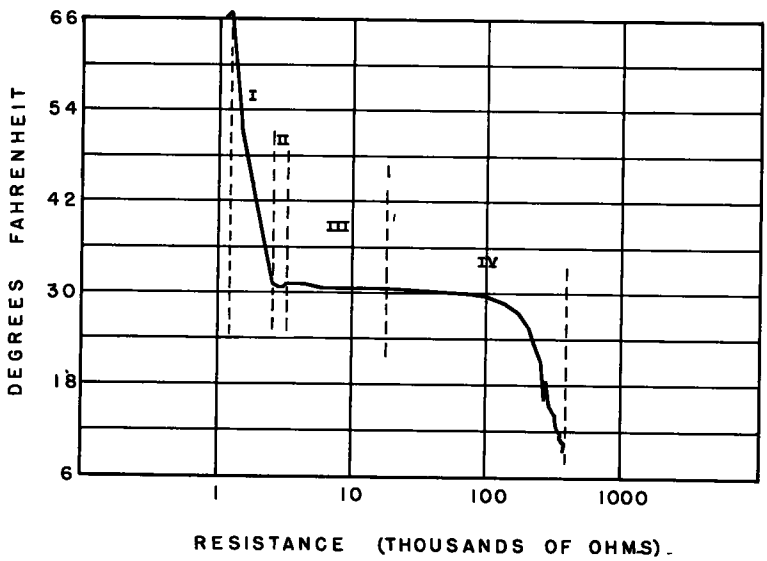


Figure 2. Temperature-resistance relationships from the preliminary freezing experiment. Note particularly those relationships around 31 F., characterized by a bulge. This is considered to indicate the commencement of crystallization, and is followed by static temperatures and sharply increasing resistances until the completion of crystallization.

The resulting data, when plotted and analyzed, clearly supported the basic hypothesis but also indicated several difficulties in experiment design. A more-elaborate study was devised to offset these objections.

The second laboratory study entailed observation of temperature and resistance

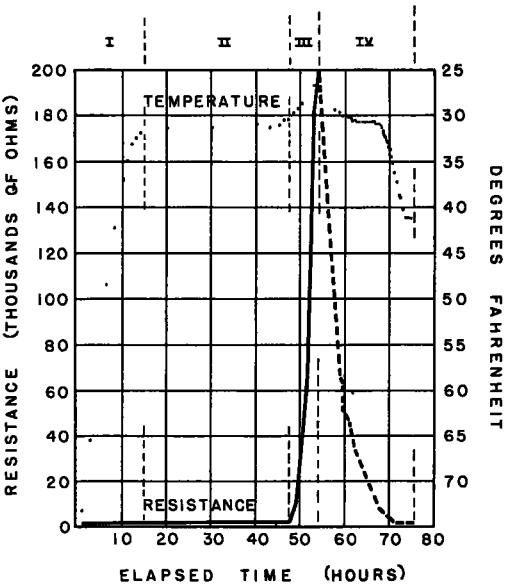


Figure 3. Resistance and temperature variations noted in one complete freezing-thawing cycle, utilizing a single, large soil sample. Temperature and resistance have each been plotted against elapsed time, and in opposition to each other, to illustrate the decisive point of commencement of crystallization, and show their relationship to each other at any given time. It will be noted that temperature and resistance values are quite similar before and after freezing.

at 15-minute intervals throughout the experiment, with 1-minute observations during periods of critical change. In order to accomplish the difficult task of making rapid and accurate instrumental readings, especially during the periods when observations were made at 1-minute intervals, a multi-switch selector panel was designed. This device permitted switching from one instrument to another without opening and closing the freezer box, thereby eliminating a major source of error in uncontrolled heat-exchange. Physical probing of the sample was necessarily omitted in the second experiment. The seven small soil samples of the first experiment were replaced by a single, large sample, instrumented with three plaster-of-paris resistance blocks and three resistance thermometers arranged so as to show penetration of cold from above and below, and to the center of the sample (see Figure 1). Regular observations were maintained throughout one freezing-thawing cycle in the second experiment, covering an elapsed time of 72 hours.

Data from the second study were carefully tabulated and plotted so as to show all possible relationships. Resistance was plotted against temperature, resistance against time, temperature against time, and resistance and temperature were plot-

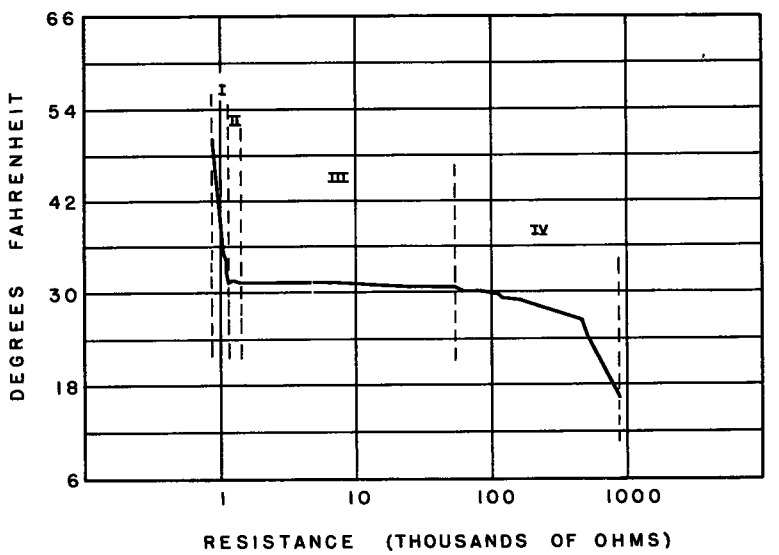


Figure 4. Temperature-resistance relationships from the final freezing experiment. Note the marked similarity between relationships found in the preliminary and final freezing experiments. Early difference in resistance noted in the two experiments may be ascribed to differences in moisture-content of the samples.

ted against time and in opposition to each other. The graphic correlations which resulted (see Figures 2, 3, and 4) clearly illustrate the decisive period of the freezing process and verify the basic hypothesis that an abrupt 200-ohm increase in resistance, under certain circumstances, indicates the commencement of crystallization of soil moisture.

Review and analysis of the tabular and graphic data resulting from field and laboratory observations lead to the following conclusions: (1) A characteristic 200-ohm increase in resistance indicates the commencement of the freezing process. (2) A period of stable temperature and resistance conditions always preceded freezing. (3) Similar results were obtained in field and laboratory studies. (4) After thawing, field and laboratory measurements showed values similar to those noted before the freezing process. (5) Increases in resistance at temperatures below 32 F. are caused by the dehydration of soil moisture resulting from freezing and indicate the degree of completion of crystallization. (6) Freezing of mineral soils within the available moisture range commences at temperatures of between 31.00 and 31.75 F. (7) Field and laboratory studies fully substantiate the original hypothesis that, under specified conditions, an abrupt 200-ohm increase of resistance of a plaster-of-paris soil-moisture unit indicates the commencement of crystallization of soil moisture.

Discussion

CARL B. CRAWFORD, Division of Building Research, National Research Council, Ottawa, Canada—Advancements in instrumentation in recent years have encouraged large-scale studies of the variation in soil temperatures and soil moisture content. Considerable interest has been shown in attempts to predict frost penetration in various soils and efforts have been made to establish the frost line by nondestructive methods.

Correspondingly, attempts have been made to simplify the determination of the frost line by direct measurement. A simple boring tool for this purpose is described in detail by Goodell (1). Another method (2), developed in France, employs a series of capillary tubes which are installed in the ground and may be withdrawn periodically to obtain a direct observation on the region of freezing.

Many observers are content to determine the position of the frost line by interpolation of soil temperatures measured in a vertical profile. It has been established by the authors and others, notably the Corps of Engineers (3), that it is reasonably accurate to assume that the frost line occurs at the 32-F. isotherm. The fact that the position of the frost line can be estimated at any time by interpolation of temperatures appears to be an advantage over the use of moisture meters for this purpose. A further disadvantage to the use of moisture meters for locating the frost line is that continuous readings are required to establish the time of initial crystallization. Consequently, the electrical-resistance method appears to have no advantage over temperature-sensing elements in field installations which are read periodically. It is probably, however, that electrical-resistance units could be used to advantage for detailed studies of freezing phenomena.

The detection of freezing by means of the change in dielectric constant was reported by Powers and Brownyard in 1947 (4). This test is based on the fact that the dielectric constant for water is about 80 and that of ice about 4.

A good review of work on the freezing point of soil moisture is given in a paper by Richards and Campbell (5) in which it is pointed out that the preparation of samples for laboratory tests may have an appreciable effect on results. For this reason it is important that the exact procedure be described in reporting work of this nature.

Ground-temperature records are now being collected at ten locations in Canada by the Division of Building Research of the National Research Council. Most of this work is being done jointly with other organizations. In all cases the depth of frost is determined by interpolation of the temperature profile assuming that the frost line and the 32-F. isotherm coincide.

In addition to the temperature observations, an attempt is now being made to collect depth of frost information at 13 locations across Canada. In order to simplify the field work, records include only date, location, depth of frost, soil type, and surface cover conditions. It is

hoped that a comparison of these records with air temperature records will assist in the empirical approach to the prediction of frost penetration. If preliminary work is successful, this frost survey will be extended to include more regional variations in climate.

Closely linked with the work on ground temperatures is a study of soil moisture fundamentals. The first stage of this study was a review of literature on nondestructive methods of measuring soil moisture content (6). Apparatus is now being assembled for the evaluation of electrical resistance suction meters.

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Frost Action on Small Footings

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Hydro Electric Power Company, Toronto, Ontario, Canada

Frost heave of small footings is a periodic trouble at transformer stations of the Ontario Hydro Electric Power Commission. Warped transformer pads, opened switches and distorted service boxes are among the disturbances arising during the winter months. This paper summarizes the results of laboratory and field investigations, made during the last 4 years, to determine ways of controlling footing heave.

Initial laboratory tests indicated that the grip between frozen soil and concrete was of the order of 400 psi. at - 10 F. These results confirmed measurements made in Siberia approximately 18 years ago. During the last two winters various proposals for heave control were tried on an installation of small footings, placed in a frost susceptible soil 40 miles north of Toronto. Low temperature grease, polyethylene covers, gravel backfill, and soil treatment with a waste product of the pulp and paper industry were among the measures used to reduce the grip between the frozen soil and concrete. Reaction to the heaving forces was also provided by subjecting the footings to various loads or by equipping them with enlarged pads below the frost line.

During the winter of 1951–52, untreated footings having a buoyed weight of approximately 150 lb. heaved as much as 4 inches; greased footings moved about this amount. Frost penetration that year was 12 inches. The following winter was mild and the maximum heave of untreated footings was less than 2 inches. Less movement was obtained under higher loads, however, and footings equipped with concrete pads did not move at all. The greased footings under loads of 350 lb. and above did not move.

An analysis of the frost heave problem, based on these observations, resulted in the preparation of an approximate chart which indicates the bearing pressure required to overcome footing heave. It suggests that the cross-section of footings within the zone of freezing should be made as small as possible, consistent with the structural strength of the concrete. Below this depth the footings should be enlarged to transmit the footing load to the soil safely. The use of low temperature grease was also recommended for special applications such as conduits etc. where a reaction force is not obtainable. Field observations are continuing to assess the resistance of these footings under more-severe winter conditions.

● THIS paper deals with the problem of frost action on and associated heave of small footings founded below the zone of freezing. Footing heave is a nuisance problem that besets many of the transformer and distributing stations of the Ontario Hydro and other utilities each winter, causing switches to be opened, the distortion of service boxes and other minor disturbances.

To date measures for controlling these movements include the provision of footings with smooth, tapered sides to reduce the vertical component of the heaving force and gravel backfill to discourage the formation of ice layers. A satisfactory solution for meter houses has been obtained by placing each footing in a grease-packed sleeve. However, most of these frost-control proposals have been ineffective, and a more-thorough study of the problem was begun early in 1950.

INITIAL STUDIES

In order to ascertain the magnitude of the forces associated with footing heave, the first step in the investigation was to measure the grip which develops between frozen soil and concrete. This was done using the simple arrangement shown in Figure 1. Saturated soil was placed inside the hollow concrete prism and frozen for 24 hours at - 10 F. The specimen was then placed in a compression machine and the force required to free the frozen plug

was measured. The sample was loaded rapidly to avoid excessive plastic flow. Values obtained ranged from 304 to 495 psi., indicating that the grip of frozen soil is quite considerable.

When forces of this magnitude are involved, the practice of tapering footings would be relatively ineffective against heaving and actually appears to be based on faulty reasoning. Even though some reduction is obtained by inclined sides, this advantage is offset by the greater surface area exposed to the frozen ground.

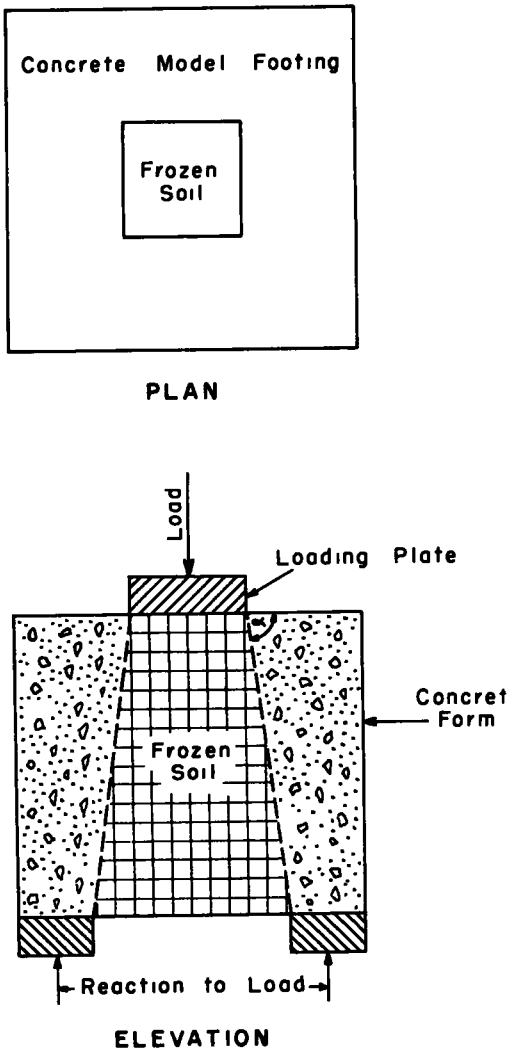


Figure 1. Apparatus used to determine the bond between frozen soil and concrete.

FIELD TESTS

During the past 2 years, experiments designed to test several proposals for controlling heave were conducted on a footing model installation in soil known to be susceptible to frost action.

The treatments used in the test program were made on footings 8 inches square and 4 feet deep. In one set of tests, the elimination of the rigid grip of the frozen soil was sought by the application of low-temperature grease to the sides of the footings or by wrapping the footings with polyethylene sheet. In other tests the formation of ice layers in the soil was discouraged by backfilling with gravel or by treating the soil with waste liquor from the pulp and paper industry. Various loads or methods of anchorage were also applied to the models to determine the resistance which must be provided to prevent their movement. A general layout of the site and sketches of the various treatments are shown in Figures 2 to 4.

During the winter of 1951—52 untreated footings in the installation were heaved as much as 4 inches, although the maximum frost penetration did not exceed 12 inches. Last winter was even milder and ground movement was only half the previous value. Ground temperatures in the zone of freezing did not fall below 32 F. Under these moderate freezing conditions, complete protection was obtained for the greased footings under a load of about 350 lb. In general, other treatments provided some relief but appeared to be inadequate for permanent protection. Figure 5 presents the maximum heave of each footing under various loads during the winter of 1952—53.

The most-promising and most-practical arrangement for controlling frost heave, as indicated by these tests, is a footing constructed with an enlarged base below the frost line. In this installation, a base approximately 3½ times the footing width gave the desired anchorage against heave. Its effectiveness is well illustrated in the inset of Figure 5.

ANALYSIS OF FOOTING HEAVE

The resistance to frost heaving, provided by the addition of sufficient footing load, suggests that a rational approach to heave control can be found. Therefore, the following analysis of the problem was made in the hope that it might lead, at least, to an approximate method of design against frost action. The process of heaving probably occurs in a manner similar to that

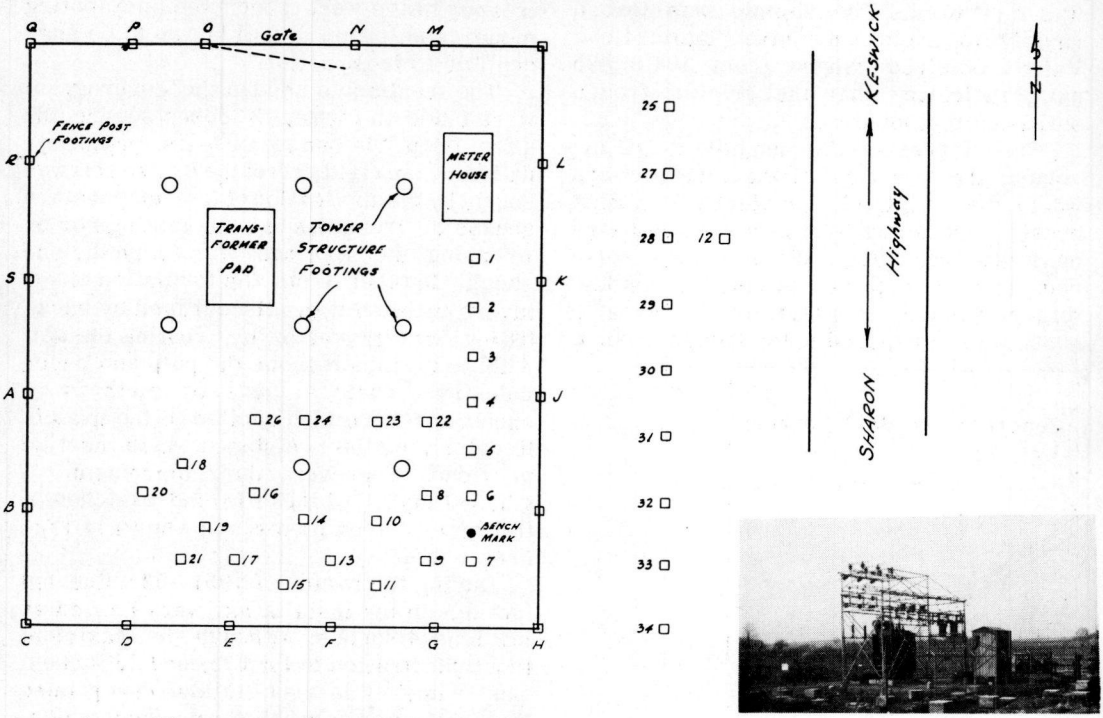


Figure 2. Sketch showing the location of the footing models.

outlined in Figure 6. This sectional sketch illustrates a footing rigidly gripped along XY by a layer of frozen soil, t inches thick. The heaving force, pushing up on the frozen layer and therefore on the entrapped footing, is generated in the thin zone of freezing as pore water expands into ice. In this example it is assumed that the footing is loaded sufficiently to resist these forces and that consequently the frozen soil is held down in its vicinity. The magnitude of the heaving forces is probably proportional to the restraint applied to the frozen soil layer; therefore, the upward

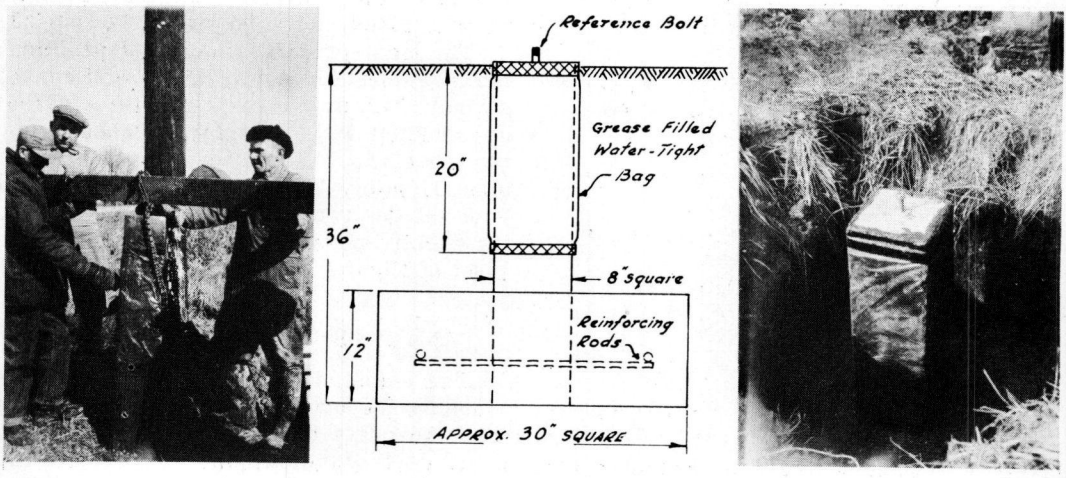


Figure 3. Sketch and photograph (right inset) showing a footing with an enlarged base and protected against frost action by a grease-filled water-tight polyethylene bag. Left inset - Lowering a greased treated footing into position - 200 lb. pedestal added after installation.

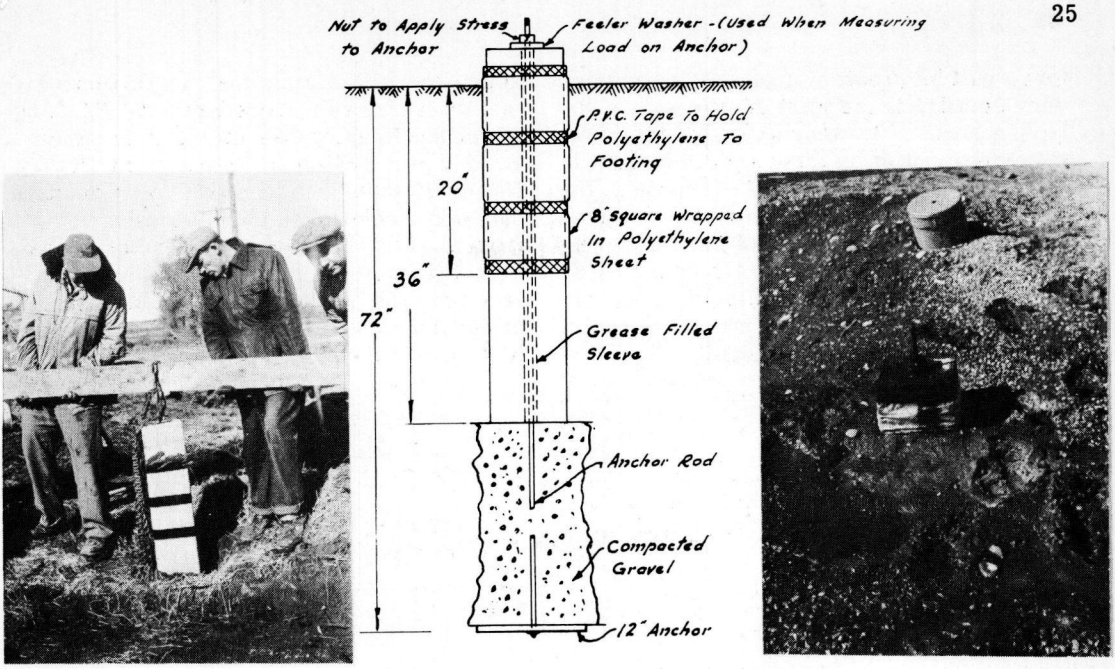


Figure 4. Sketch showing an anchored footing with a polyethylene cover. Left inset - Lowering a polyethylene covered footing into position. Right inset - Anchored footing showing wrinkling of polyethylene cover as ground heaved.

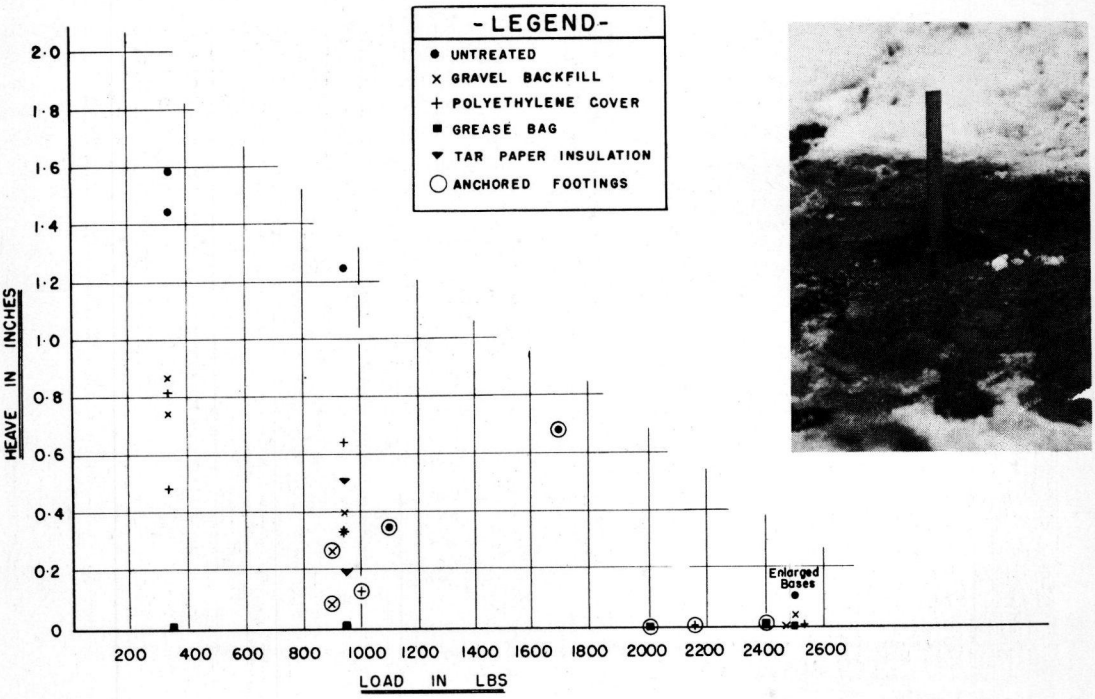


Figure 5. Chart showing how footing heave was reduced by load (All loads estimated for anchored and enlarged base footings). Inset - Photograph of the heave resistance of enlarged base footings. Surrounding ground heaved up approximately 2 inches.

force will be greatest near the footing and very small near the point of inflection of the frozen layer. By assuming that the pressure distribution is triangular and equal to the uplift resistance, W, of the footing, the maximum bending moment of the ice layer can be calculated readily. When a footing is continuous, the resultant heaving force will act at a third of the distance, L, between the side of the footing and the point of inflection of the frozen soil.

suggest that this distance, L, is approximately 10 times the frost depth, t. ("Lifting Force and Bearing Capacity of an Ice Sheet" B. Lofquist Technical Translation TT-164 National Research Council of Canada). The same relationship can be assumed for frozen soil, which can reasonably be expected to deflect in the same manner as lake ice; the bending moment, M, exerted at the contact surface xy of a continuous footing will then be

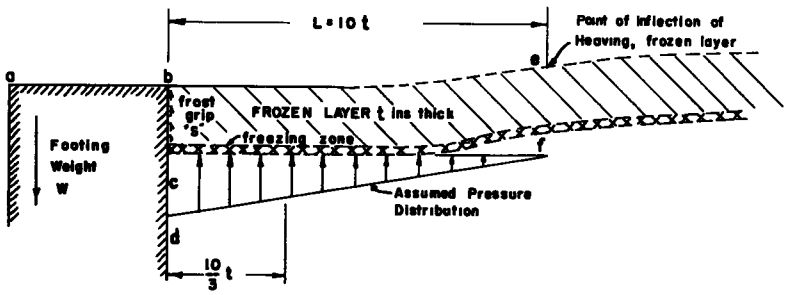


Figure 6. Sketch showing mechanics of heave of a continuous footing.

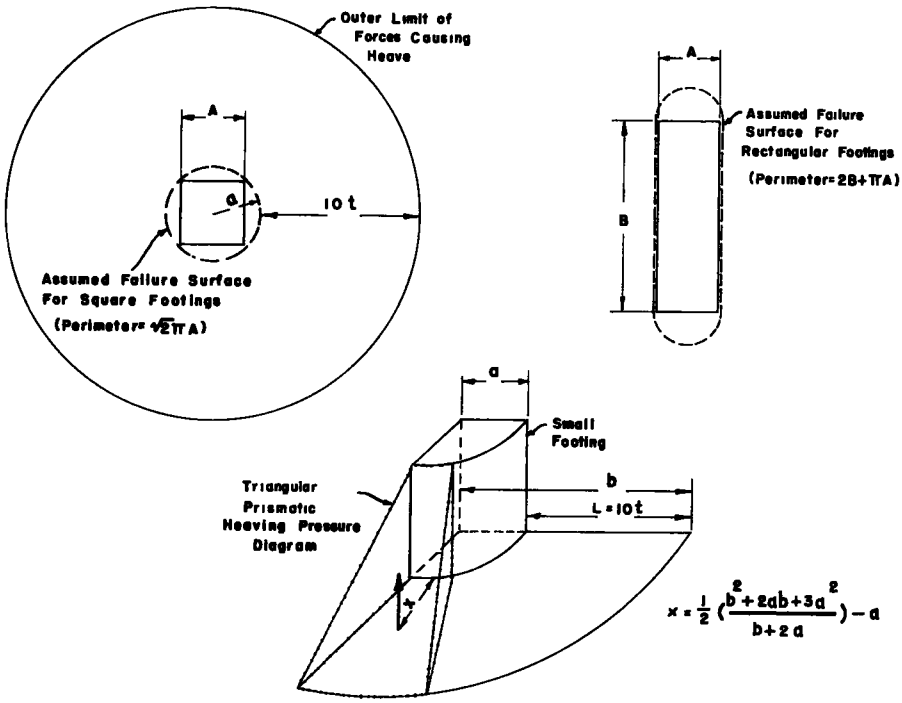


Figure 7. Sketches showing assumed failure surfaces and pressure distribution for square and rectangular footings.

Measurements obtained by Lofquist, of Sweden, of the deflection of lake ice around piers during periods of rising lake levels,

$$\frac{10}{3} \text{ Wt. in in.-lb.}$$

The moment of resistance of the frozen

soil at this surface is equal to the product of the section modulus of the frozen ground and the frost grip, f, along xy. Using the value f = 400 psi., the ultimate bending resistance, M_r, along a footing of perimeter P becomes 66.7 Pt^2 in.-lb. Hence the load, W, required to prevent heaving of a continuous footing is equal to 20 Pt lb.

$$\frac{A}{\sqrt{2}}$$

Any segment of the heaving-pressure diagram bounding this circle will have the shape of a triangular prism with a center of gravity located at a distance from the circle:

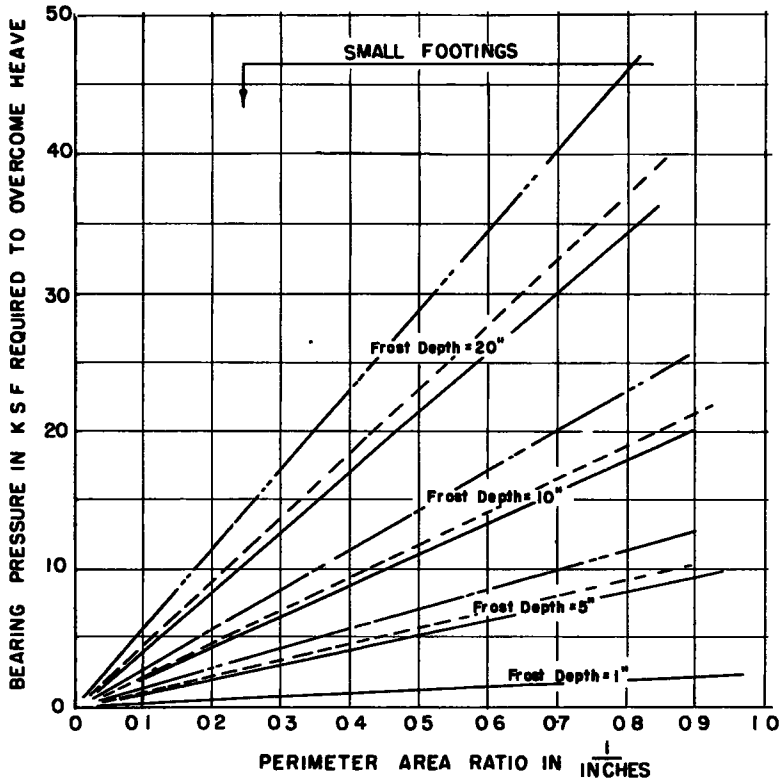


Figure 8. Chart indicating the bearing pressure required to overcome footing heave, (frost grip S = 400 psi.).

Unfortunately, because the distribution of the heaving forces becomes quite complex, this simple method cannot be used for the accurate calculation of the loading required to resist heaving of small square and circular footings. An approximate estimate however, can be obtained by slightly modifying the analysis. As indicated in Figure 7 the zone in which the heaving forces are confined about a small footing probably comprises a ring L = 10t inches wide. The bending moment is assumed to be most critical along the surface indicated by the broken lines; for a square footing of width A the length of this surface is the circumference of a circle of radius

$$X = \frac{1}{2}\,\frac{b^2 + 2ab + 3a^2}{b + 2a} - a$$

where $a = \frac{A}{\sqrt{2}}$, and $b = a + 10t$

The bending resistance developed around the perimeter, P, of this critical section is again approximately 66.7 Pt^2 in.-lb. Therefore the footing load, W, required to resist heave will be

$$\frac{66.7\ Pt^2}{X}$$

The chart in Figure 8 indicates, for various depths of frost penetration, the bearing pressure necessary to overcome

footing heave. The size of the footings is expressed by the ratio of their perimeter to area, a quantity which becomes larger as the size of the footings decreases. It is immediately apparent from the chart that the bearing pressure required for small footings is much higher than can be applied safely to most soils. Since it is unlikely that small footings would be designed to withstand such high bearing pressures, they require some special treatment or modification to make them invulnerable to frost action.

PROPOSED FROST-HEAVE CONTROL MEASURES

Although small footings are susceptible to frost heave when they conform to orthodox design, a simple modification should provide the necessary resistance. To achieve this resistance the cross-section of the portion of the footing within the zone of frost penetration should be as small as possible; reference to Figure 8 will assist the selection of proper footing size. Below the frost zone the base of the footing can be enlarged to transmit the load safely to the soil.

For such applications as ground-entry point of cables and conduits, where adequate heave resistance cannot be obtained by design or loading, grease-packed sleeves should provide immunity from frost action. Sleeves can also be used to protect the benchmarks and monuments established by surveyors. In many instances these reference points have approximately the same dimensions as the footing models used in the present experiments and therefore are highly vulnerable to frost action.

Discussion

HAMILTON GRAY, Soils Engineer, Maine Highway Department—The subject of Trow's investigation is one which has also interested the writer for a number of years, primarily because it seems to offer a relatively inexpensive way of accumulating fundamental data on frost action for the purpose of obtaining a more-complete understanding of this phenomenon. This is not to imply that the investigation lacks considerable practical value, since although the actual cost of the type of installation studied by Trow is relatively small, the total investment in all similar installations represents a large sum of money. Similarly, when such an installation requires maintenance or some form of rehabilitation as a result of frost action, the cost is not excessively great. However, such maintenance is required on many similar installations. Therefore, in the aggregate, the necessary expenditures total a large sum.

Figure A illustrates the principles studied by Trow. The upper part of the pedestal or pier in this figure is subject to what has been concisely termed "adfreezing" of soil to its lateral surface, and as the frozen ground is moved upward by the expansion of ice or by the development of ice layers at the freezing line, a force termed by Trow "frost grip" tends to raise this portion of the pier upward also. The frost grip creates a reaction in the soil adjoining the pier below the zone of freezing, in the form of a vertical compressive stress in excess of that normally created by the earth overburden about the pier. The nature of the variation of this stress is unimportant here. It is designated by "excess soil pressure" in Figure A.

If the load P applied to the top of the pier is insufficient to overcome the lifting tendency of the frost grip, tension represented by the force T in the diagram will develop in cross-sections below the frost line. In other words, that portion of the pier which is being elevated by the rising soil tends to pull along with it the lower portion. This tendency to move the lower portion is resisted by the weight, W, of this lower portion and by soil pressure on the upper surface of the footing.

That the frost grip usually proves more than capable of moving the lower portion of the pier and its footing seems to indicate that the unfrozen soil between the frost line and the upper surface of the footing is compressed under the action of the excess soil pressures. The amount of compression will depend upon the magnitude of the excess pressures, as well as upon the inherent compressibility of the intervening soil.

Upon the release of the frost grip during thaws, the excess compressive stress in the soil diminishes at an unknown rate. The footing, however, having been elevated through a distance approximately equal to

the compression of the soil, often does not return to its original position. This is attributed to the fact that soil may squeeze or fall inward from the sides of the cavity which the raising of the footing creates. This intruding soil serves to prevent the bottom of the footing from returning to its original position, hence, there is a net annual increase in elevation, never greater than the maximum displacement during the winter, which accumulates from year to year so that the elevation of the pier progressively increases. Of course, if the pier fractures under the action of the force T, its upward movement will be accelerated in future years.

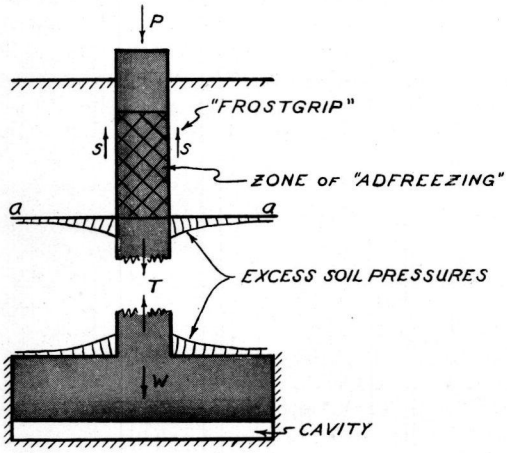


Figure A.

There does not appear to be any limit to the displacement of such a pier until it has been almost wholly ejected from the ground. Stone bounds and similar monuments have been found lying prone upon the ground. Others defining street boundaries have been uplifted unevenly and acquired a resultant tilt displacing their tops by nearly a foot. Invariably this displacement is away from the street and is attributed to the more-energetic frost action which accompanies regular removal of snow insulation by street clearing.

An entirely similar process would seem to explain the annual spring crop of stones with which farmers have to contend in cultivating soil derived from glacial till. No matter how completely the stones are removed from the ground surface, additional ones will appear within a year or two, having been raised by frost action from various depths beneath the ground surface.

Examples of the type of behavior discussed herein are numerous in any locality where the frost penetrates normally to depths exceeding a few inches. Figures B, C, and D illustrate culvert head walls which have been displaced or broken by frost action either upon the head walls themselves or upon the culverts which in turn move the head wall. In Figure D the culvert has broken through the surface of the shoulder and part of the pavement, as well as having moved the head wall entirely out of position. Rupturing of head walls just above the culvert crown is common and suggests that the culvert pipe is moved more energetically than would be the head wall itself.

Figure E illustrates a door stoop, the outer edge of which was supported on concrete cylinders embedded in the ground. This photograph was taken during the first winter after construction. The posts did not return in the spring to their original position; consequently, the owner will be ultimately faced with the necessity of removing at least the top portions of these posts in order to permit him to open his front door. Presumably the builder believed that these posts, extending to a depth of 5 feet, would never move.

Figure B.

Referring to Figure A, the principles which have been applied in Trow's efforts to seek a solution to this problem can be summarized as follows: (1) Reduction of the soil motion by the use of admixtures which will reduce or eliminate soil freezing and heaving or through the use of non-frost-susceptible soils. (2) Reduction of the area over which the moving soil is in contact with the pier or anything attached to the pier (adfreezing area). (3) Reduction of the force (frost grip) transferred from the moving soil to the pier by interposing a weak material between the two. (4) Op-

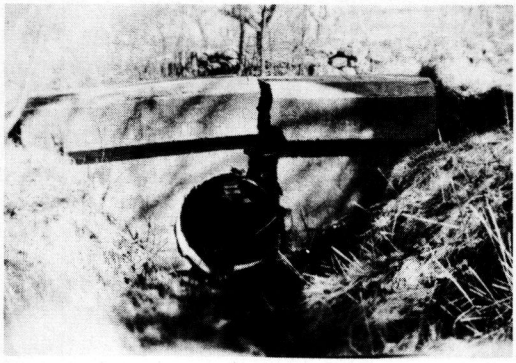


Figure C.

position to the lifting effect either by countering it with a downward load P of substantial magnitude or by providing a footing or enlarged base so that the uplift produced by frost heaving creates a downward reaction on the pier by the footing.

There are certain types of construction (such as culverts, monuments and bench marks for surveying purposes, and very-lightly loaded buildings) for which it is not possible to apply a sufficient downward load P to effectively prevent displacement. Furthermore, even though sufficient load can be applied to reduce the maximum displacement each winter to a small value, this does not ensure that the accumulation of net annual displacements will not eventually become serious.

It is frequently stated that if a wall, post, or pier is extended below the maximum depth of frost penetration it will not be moved by frost action. Just where this misconception originated is unknown. Possibly, it was derived from experience with buildings in which the heat loss through basement walls is adequate to prevent freezing of the outside soil to the surface of the wall. It is, of course, known that if freezing develops beneath the bottom of a footing or wall extremely large loads are required to counteract the lifting tendency of the ice. Quite possibly the more-favorable behavior which resulted from eliminating this particular kind of freezing was sufficient to establish a belief that satisfactory construction could be obtained by placing the bottoms of walls and footings below the depth of maximum frost penetration.

Figure D.

It is likewise widely claimed that the use of a substantial batter, often referred to as "frost batter", will suffice to counteract the tendency for heaving. This claim is based by some on a belief that the batter itself prevents the adhesion of frozen earth to the pier. Experience with battered piers suggests that this is an erroneous concept and certainly the results of the type of test illustrated by Figure 1 in

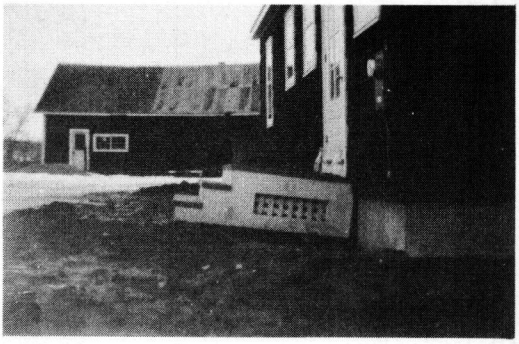
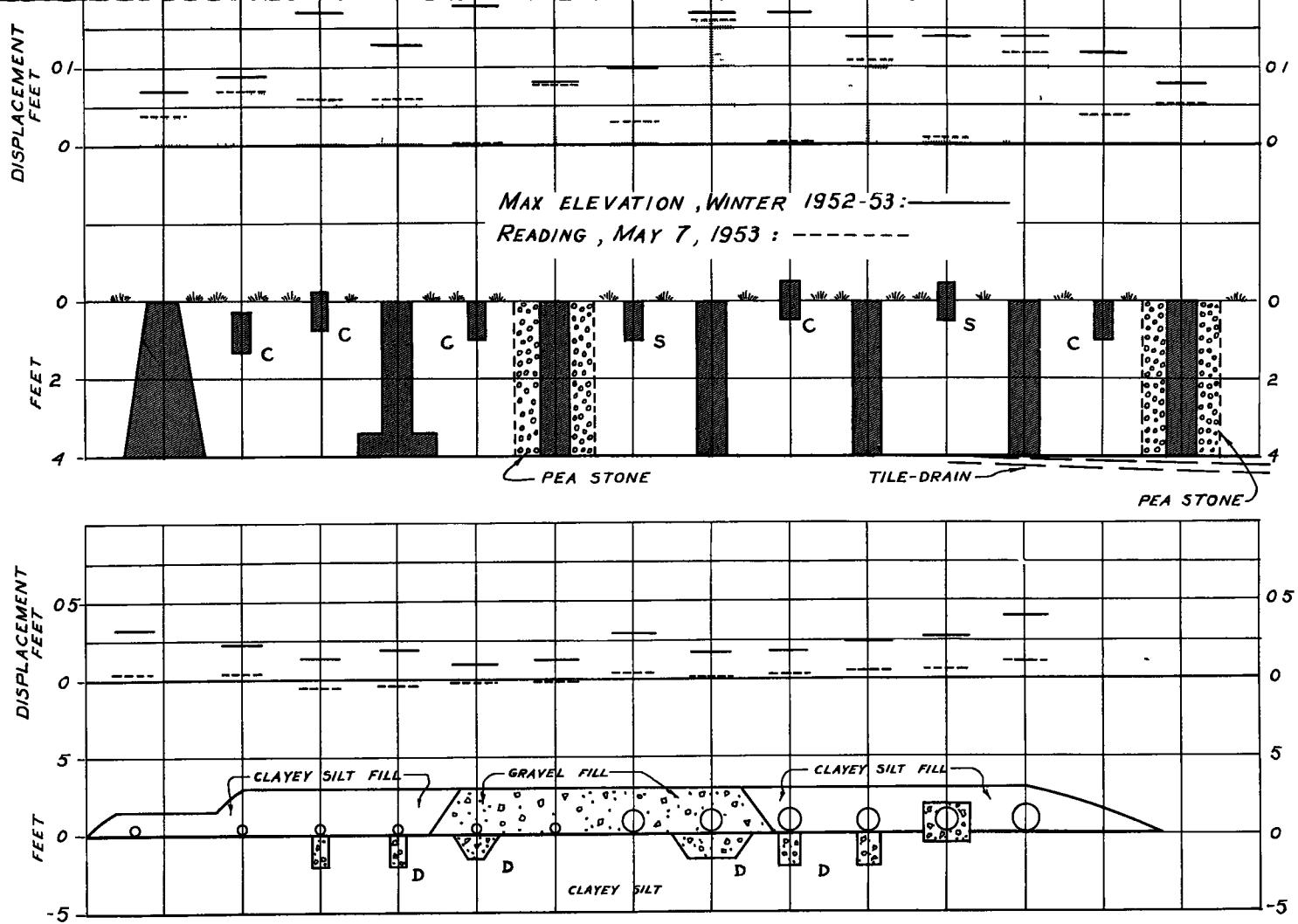


Figure E.

Trow's paper should suffice to emphasize the inaccuracy of this viewpoint. On the other hand, if such a battered pier is urged upward by soil expanding near its top, the continued batter below the freezing zone will induce soil reactions acting downward on the lower portion of the battered object.

Again experience indicates that this downward reaction is insufficient to be wholly effective. Specifically, the downward reaction will increase only as the pier moves upward against the soil and so creates passive earth pressure. Thus the earth is not rigid and must deform if it is to offer a reaction to the upward motion of the battered surface. The use of an enlarged lower portion, footing, or boot on a pier would appear to be more effective than the batter, but experience again indicates that even such a method is not reliable.


DISPLACEMENT FEET
0.1
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MAX ELEVATION, WINTER 1952-53:
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Figure F.

Observations of the behavior of light buildings without basements which are supported by short timber posts or concrete piers indicates that certain of these posts or piers periodically move to a much greater extent than others, even though the soil conditions appear to be uniform throughout the building area. Many concrete posts or piers with battered sides extending well below the maximum depth of frost penetration have been observed to move progressively until the supported structure is badly distorted.

In one instance a canning factory was supported on cedar posts approximately 20 percent of which moved upward during each winter and remained in an elevated position after each spring thaw. It was then only necessary to saw off a few inches from the top of each such post to relevel the sills and floors of the building. Eventually such posts worked almost entirely out of the ground and had to be replaced. It appears fortunate in this instance that the support was of timber rather than concrete and that the maintenance work could be done when canning activity was nil.

In planning an additional building for this particular site, which was underlain by a glacial till consisting primarily of clay containing many stones, it was recommended that if posts or pedestals were to be used they be surrounded by granular material which, in turn, should be drained by means of clay tile. Owing to the very heavy floor loads, approximating 600 lb. per sq. ft., it was found that such a type of construction would be approximately 50 percent more expensive than the utilization of a compacted gravel fill upon which a concrete floor slab could be directly laid. The cost of supporting the structure on piers not surrounded by drained gravel would have been approximately equal to the cost of a slab placed upon gravel fill but would not have offered the certainty of satisfactory performance inherent in the use of gravel fill.

Similar observations also indicate that when such structures are supported on posts bearing upon pads which rest on the surface of the ground, less trouble results than when the pads are buried. In the former case, loss of contact between post and pad is unusual, whereas in the latter they soon part company. Hence, although the building may become distorted during heaving weather, the original grades are likely to be regained automatically when the ground thaws, and there is no net annual increase in elevation. Certainly the annual maintenance of this type of construction is less than where releveling must be accomplished each spring.

One form of protection not mentioned in the paper consists of supporting loads on steel rods or pipes which bear upon a pad or plate located below the frost line. A pipe of larger diameter, used as a loose sleeve extending from above the ground surface to the pad or plate, will move with the heaving ground without transferring any reaction to the smaller rod or pipe. This method is commonly used in northern New England for supporting light loads and is effective until the sleeve has been elevated so that its upper end is in contact with the supported structure. Thereafter the sleeve itself may tend to force the structure upward, although if the buried pad or plate is securely anchored to the smaller rod this will help to resist upward motion. If wet earth or water collects between rod and sleeve and freezes, the sleeve becomes useless, and in certain soils, the sleeves may corrode fairly rapidly. However, the great majority of such installations appear to function in a thoroughly satisfactory manner.

Figure F illustrates typical installations, some of them rather similar to Trow's piers, which have been observed for the past 2 years. The lower portion of this figure shows experimental culverts under which various types of bedding have been utilized in order to determine the one most effective in reducing frost displacement. Where granular bedding has been used, certain beds (D) have free-draining outlets, while the other beds are simply laid in trenches without outlets.

Culverts which are not deeply buried tend to move upward progressively from winter to winter until they eventually become exposed at the roadway surface. Prior to that time upward displacement of the roadway surface over the culvert necessitates frequent maintenance, and during winter this results in a bump in the pavement surface. The upper portion of the figure illustrates various small piers and cylinders, some placed without any antiheave treatment and others installed in various ways intended to counteract movement by frost. All piers extend to a depth of 4 feet below ground surface. The smaller objects are either standard con-

crete cylinders (C) or hollow steel cylinders with capped ends (S) of the same dimensions as the concrete cylinders.

It should be realized that only the accumulated behavior over a period of several years will furnish a reasonable indication of the effectiveness of various types of design. The results shown on the figure, which apply to a single year's observations only, indicate the maximum upward displacement occurring some time in the late winter or early spring. The resultant displacement representing the sum of the annual net increases in elevation appears to be more important from the economic standpoint than the maximum displacement attained in any single winter, although in certain types of installations the magnitude of displacement during the winter alone will be sufficient to cause considerable damage.

The immediate conclusion to be drawn from the data shown in Figure F is that while some designs appear more effective than others, none of those adopted have proven wholly effective. Whether in the course of 5 or 10 years there will be marked differences in the observed accumulated displacements remains to be seen. Owing to the differences in behavior which are commonly observed within a small area, such as may be covered by a single building, it is probable that behavior of various designs should be placed upon a statistical basis. In other words, the behavior of a substantial number of piers of a given design is likely to be somewhat erratic and the relative frequency of unsatisfactory behavior as compared with similar frequency with respect to a different design would provide a criterion for recommending one in preference to the other. It may prove impracticable to establish a design which is 100 percent effective at all times. This is particularly true in view of possible deficiencies in workmanship.

Consequently, it is the writer's belief that Trow's work should be encouraged and extended in order that erratic behavior may not play an important part in the conclusions which may be drawn from such investigations.

In addition to the culvert behavior illustrated in Figure F, observations have been made on similar installations under actual roadways. In some cases behavior is contrary to what would logically be expected, and the difficulty has been traced to movement of benchmarks constructed of concrete in accordance with some of the pier designs also shown in Figure F. We are thoroughly convinced that the only completely trustworthy benchmarks that should be used in making wintertime observations are those which are established on bedrock or upon substantially constructed heated buildings. Any monuments, irrespective of their design, tend to greater or less displacements whenever they are surrounded by freezing soil.

The installations and observations illustrated by Figure F have been made at the University of Maine. They represent part of a project which has been largely financed by a grant from the University's Coe Research Fund. The corrugated metal culvert pipes were generously provided by Bancroft and Martin Rolling Mills of Portland. For an associated similar project, corrugated metal pipe was furnished by the New England Metal Culvert Co. of Boston and concrete pipe by the Hume Pipe Co. of New England.

PHILIP KEENE, Engineer of Soils and Foundations, Connecticut Highway Department—The author is to be commended on presenting an interesting paper containing valuable data on a subject seldom investigated experimentally in the past. It is to be hoped that he will continue to work on this project and that Nature will provide him with colder winters than those of recent years so that his remedial devices will be severely tested.

The writer was asked to give in this discussion an account of differential frost heaving of a highway pavement that was related to the presence of pipe culverts of various sizes under the pavement. The highway is a part of Route 15 in Hartford, Connecticut, known as the Wilbur Cross Highway, where it crosses a flat swampy area called the South Meadows, located in the southeast corner of the city. The highway is a four-lane expressway with a 10-foot raised median strip of grass. The section studied is about a mile long. The pavement is on a low fill 4 to 7 feet high. The pavement is 9 inches of concrete, but not reinforced, laid in 1942. Subbase is 12 inches of bankrun gravel throughout. The embankment is chiefly organic clayey silt taken from the ditch excavations on both sides and from nearby borrow areas. Ground-water table under the pavement is 5 to 6 feet below pavement.

The bankrun-gravel subbase is a red gravel of Triassic origin, having approximately the following gradation:

100%	passing	5-inch	sieve
60%	"	¼ "	"
20%	"	No. 40	"
10%	"	No. 100	"
8%	"	No. 200	"
2%	so-called clay sizes		

The portion passing the No. 40 sieve has not sufficient plasticity to permit making a plastic limit test but that passing the No. 100 sieve was slightly plastic.

The silt is a gray organic clayey silt apparently deposited during past floods of the Connecticut River, whose normal stream bed is 1/2 mile east of this swamp. The top of swamp is about 3 to 7 feet above normal river level. The clayey silt contains about 15 percent of fine sand 55 percent of the so-called silt sizes (greater than 0.005 mm.), and 30 percent of the clay sizes. Its liquid limit ranges from 37 to 26 and its plastic limit is about 24. Its natural water content varied between 45 percent and 26 percent of dry weight, but in the embankment its water content is normally about 21 to 32 percent. Standard Proctor density is about 101 pcf. at 22 percent optimum water content.

My attention was first called to cracks in the pavement in October 1944. As the pavement surface at the cracks was flat then, differential frost heaving due to the so-called chimney action was suspected. All cracks are transverse to the parkway. There are 30 cracks, of which half extend a full pavement width (two 12-foot lanes) and half are only half width (one 12-foot lane). Of these, all but three are above or close to cross culverts. There are one ore more cracks in the pavement above or close to each of the four large (36-inch and 48-inch) cross culverts and similar cracks as 15 of the 29 small (12-inch) cross culverts. The 12-inch culverts extend only under two lanes and the cracks appear only in the lanes under which the culvert is located. The 600 feet of longitudinal culvert caused no cracks.

In January 1945, 100 points were painted on the centerline of northbound and southbound lanes. Elevations were read on them four times during that winter and the following spring. Most of the points were located at a crack or at transverse joints near the crack, to furnish a profile of differential heaving extending for 25 to 50 feet on each side of a culvert. These readings verified the belief that excessive differential heaving at the culverts created a condition that resulted in the cracks. Furthermore, the worst differential heaves become obvious to the motorist during the latter half of severe winters.

At this site the three requirements for frost heaves were present: cold temperature, silty or clayey soil, and high water table. The pavement heaved rather uniformly (½ inch to 1½ inches) between culverts, causing only three cracks in the mile of pavement. At the cross culverts, differential heaving (in excess of the uniform heaves) varied from ½ inch or 1 inch of rise in 10 feet to 1 inch or 1½ inch of rise in 15 feet or 20 feet. At the culverts, cold air inside the pipe apparently created an additional frost zone adjacent to the pipe circumference. The larger-diameter culverts caused more differential heaving and worse cracking of the pavement than the small culverts. The large culverts were less clogged with snow, had a larger zone of frost adjacent to their outer surface, had a larger cross-section area for cold air per square foot of outer surface, and had a horizontal opening at each end, whereas the 12-inch culverts connect at one end to two inlets which rise vertically to horizontal gratings at the gutters.

Pavement cracking at the 12-inch culverts was absent when the distance from top of pipe to bottom of subbase was more than about 24 feet. Also, it was absent when top of pipe was more than about 5 feet above ground-water table. Therefore, for conditions like those on this project, a rule might be established for cross culverts that when (1) average distances from top of pipe to top of pavement is less than 4 feet for pipes under 24 inch diameter or less than 5 feet for pipes 24 inches and larger and (2) pipe invert is less than 4 feet above wet ground (or ground-water table, in the case of cuts and shallow fills), the pipe shall be surrounded by a 12-inch ring of gravel fill. This assumes the fill or cut material is a badly frost-heaving soil-silt or clay. If it contains a substantial amount of coarser material, the gravel fill can be omitted. The gravel fill should not be extended up to the pavement or subbase, as that would result in a depression if the highway heaves uniformly.

This treatment is not needed for longitudinal culverts, as they apparently have much weaker chimney action. It is not needed for bridges and box culverts having an earth fill above, as their roofs are too high above the ground-water table and they are designed to avoid an accumulation of surface infiltration.

Some HRB Publications Relating to Frost

Bibliography 3: FROST ACTION IN SOILS (1948) 57 pp. (mimeo.) $.45.

A chronologically arranged, annotated and indexed bibliography, containing important references from the year 1890 to 1948.

Bulletin 40: LOAD CARRYING CAPACITY OF ROADS AS AFFECTED BY FROST ACTION (1951) 38 pp. $.75.

Contains a committee report involving tests conducted in six states, and the paper "The Effect of Temperature on the Bearing Value of Frozen Soils," by Miles S. Kersten and Allen E. Cox.

Bulletin 54: LOAD CAPACITY OF ROADS AFFECTED BY FROST (1952) 17 pp. $.30.

The fourth committee report of this project with the objective to determine the percentage loss of strength that may occur in highways subjected to freezing and thawing.

Bulletin 71: SOIL TEMPERATURE AND GROUND FREEZING (1953) 124 pp. $1.80.

Contains the following studies: Cold-Room Studies of Frost Action in Soils, A Progress Report, James F. Haley; Frost Design Criteria for Pavements, Kenneth A. Linell; Soil-Temperature Comparisons Under Varying Covers, George A. Crabb, Jr., and James L. Smith; Calculation of Depth of Freezing and Thawing Under Pavements, Harry Carlson and Miles S. Kersten; Frost Action Research Needs, by A. W. Johnson and C. W. Lovell, Jr.

Bulletin 96: LOAD-CARRYING CAPACITY OF FROST-AFFECTED ROADS (1955) 23 pp. $.45.

Reports of Nebraska, Indiana and Oregon on road strength loss due to freezing and thawing action.

Bulletin 100: SOIL FREEZING (1955) 35 pp. $.60.

Special Report 1: FROST ACTION IN ROADS AND AIRFIELDS, A REVIEW OF THE LITERATURE, 1765—1951 (1952) 288 pp. $3.00.

Presents a detailed study and review of important works on the subject. Includes 242 illustrations, a bibliography and a subject index.

Special Report 2: FROST ACTION IN SOILS, A SYMPOSIUM (1952) 385 pp. $3.75.

Because of the wide diversity of subjects in this symposium, the subject matter was divided into seven sub-units. These are: (1) Climate and Distribution of Soil, (two papers); (2) Soil Temperature and Thermal Properties of Soils, (four papers and discussion); (3) Soil Moisture and Moisture Movements, (seven papers); (4) Basic Data Pertaining to Frost Action, (eight papers); (5) Frost Action and Spring Breakup, (seven papers); (6) Remedies and Treatments, (seven papers); (7) Needed Research Pertaining to Frost Action, (two papers).

Research Report 10-D: COMMITTEE REPORT AND MANUAL OF RECOMMENDED TESTING PROCEDURES ON LOAD CARRYING CAPACITY OF ROADS AS AFFECTED BY FROST ACTION (1950) 18 pp. $.45.

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