Durability of Lime-Stabilized Soils

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The results of a laboratory investigation of the durability of 10 lime-stabilized clayey soils from various parts of Canada are discussed. A fixed quantity of lime was added to each soil, based on the quantities recommended for highway subgrade applications. Durability was assessed by means of cyclic freezing and thawing, and changes in durability were determined by measuring the changes in length of the laterally confined specimens. Standard compression tests were performed on all samples following the cyclic freezing, and, in addition, changes in permeability, soil suction, degree of saturation, and tensile strength were observed on selected samples.

The changes in durability of the lime-stabilized soils are discussed in relation to a tentative theory based on the behavior of portland-cement mortars under similar conditions. The overall agreement with the modified theory was satisfactory, but further detailed testing is needed to establish some of the specific limiting values.

•DURING the past decade, there has been a considerable amount of field and laboratory research into the behavior of lime-stabilized soils. This research has tended to concentrate on three general areas: (a) fundamental mineralogical research into the lime-clay mineral reaction; (b) laboratory research into the variation in mechanical properties due to variation in lime percentage; and (c) field and laboratory research into the optimum construction procedures for various applications of lime-stabilized soils. The results have indicated that lime stabilization, when used intelligently, may have very definite economic advantages in certain clay subgrades. The introduction of this construction process into Canada has been hindered by the lack of substantial research into, or of a general hypothesis related to, the possibly adverse effects of frost action, although some initial work has been completed in the United States (2).

In an attempt to develop a fuller understanding of the behavior of lime-stabilized soils under frost conditions, the Canadian Lime Institute has supported a modest laboratory program at Queen's University for three years. The purpose of this program was twofold: (a) to obtain data for representative Canadian soils under freezing conditions, and (b) to examine critically the results of the tests with regard to explaining and analyzing the durability phenomena.

The absence of any general hypothesis to explain the frost-durability relationship for stabilized soils meant that an "ad hoc" approach had to be used initially; i.e., a substantial amount of somewhat empirical data had to be established before a consistent pattern could be predicted. In turn, this approach necessitated that certain arbitrary decisions concerning testing procedures had to be taken to avoid unnecessary examination of minutae. As far as possible, the tests should be similar to those used elsewhere, but exact replication was not intended. As a result of this program, a general hypothesis was developed and is presented in this paper in conjunction with the test results. The hypothesis actually evolved in stages during the examination of preliminary

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testing, but for convenience in discussion, it is presented here in advance of a discussion of the test results.

In general, the results of the study show that when lime is added to clay soils, heaving due to frost action can be reduced below tolerable limits. Silty soils are improved with the addition of lime, but frost damage may still be severe (based on laboratory results). The criteria for frost susceptibility of stabilized soils appear to differ substantially from those for unstabilized soils.

GENERAL HYPOTHESIS

Recent work (3, 5) has shown that the lime-clay reaction is a calcium-silicate reaction, and this is also discussed by Ingles (9) who suggests that the same complex silicates are formed in the lime-soil mixture as those in portland cement mortar. If this is the case, lime-stabilized soil systems will exhibit the same trends in mechanical properties as those for cement mortars: a distinct gain in strength with time, a strength dependency on the compactness of the specimen and on the type of cementing agent used, and an increased durability with increased entrained-air content. These trends are well documented in the literature. It also suggests that theories of frost behavior, as developed for uncemented soils, will not be strictly applicable to the problem of frost behavior for stabilized soils.

Theories of cement-mortar durability suggest that due to the calcium-silicate reaction, a substantial increase develops in the percentage of total voids as fine pores, and that the water within these pores does not freeze within the normal range of highway pavement temperatures. In this connection, Powers (18) states for cement mortars:

> The granules, called gel particles, are exceedingly small, and interstitial spaces among them are correspondingly small. These spaces called gel pores are in fact so small that water cannot freeze in them at any temperature within the range of interest. . . . As hydration of cement proceeds, alkalies become more and more concentrated in the chemically free water. This fact, as well as dimensional factors already mentioned, influences the amount of ice that can exist in the paste at a given temperature. As freezing proceeds, pure ice separates from the solution. Hence freezing increases alkali concentration and lowers the melting point of the ice. . . .
>
> Hardened cement paste in concrete normally has insufficient escape boundaries. Introduction of additional air voids by using air-entraining agents is a means of providing escape boundaries sufficiently close together. . . .

Typically, the gel-pore diameter in hydrated cements is in the order of 20 to 40 Å for 60 percent of the pore volume, and preliminary tests on frost-susceptible clays suggest pore diameters in the order of 5,000 Å. At high water-cement ratios, the permeability of hardened cement paste is in the order of 1.2×10^{-10} cm/sec, whereas silty clays may have permeabilities approaching 10^{-8} cm/sec.

In a generalized form, a modified cement-mortar hypothesis which may be applicable to the durability of lime-stabilized soils under freezing and thawing conditions may be summarized as follows:

1. The addition of lime to the soil creates a flocculation (or aggregation) which alters the normal pore size distribution in two respects: a number of substantially larger pores are created adjacent to the flocs; and within the flocs the pores are exceptionally small. Manifestation of this effect is seen by comparing grain-size distribution curves before and after lime treatment (19).
2. Within the finer pores, as the calcium-silicate reaction progresses, there is an increase in the alkalinity of the pore water due to the presence of sodium and potassium hydroxides, which depresses the freezing point below that normally encountered in highway conditions. Clare and Cruchley (1) have shown that pH values in the order of 12.5 can be expected for lime-stabilized soils.

3. The larger pores between the flocs enable the fluid to flow more readily through the soil, and are manifest by an increased permeability. (There is a paucity of experimental data related to the permeability of stabilized soil in the literature, and one of the auxiliary purposes of this investigation was to obtain some preliminary values.)

4. The complex calcium-silicate reaction develops the usual cementing agents which bind the pore walls together. In this connection, Diamond and Kinter (3) state:

> The exact products formed vary somewhat with the kind of clay and the reaction conditions, especially temperature. There are commonly at least two phases produced, a calcium silicate hydrate and a calcium aluminate hydrate. The former is usually tobermorite gel, the latter is a well crystallined hexagonal compound, which is probably an impure tetra-calcium aluminate hydrate. . . .

5. When the temperature is lowered below the depressed freezing point in the larger pores, ice formation begins with the usual volume expansion as the crystal is formed and suction develops. The volume expansion places the remaining fluid under an hydraulic gradient, which can be dissipated through the increased permeability of the soil. The soil pore tends to act as a rigid container due to the cementing action which is developed, and immediate expansion does not take place. Eventually, the available pore space is filled with ice, excess water is dispersed to other parts of the system, and heaving is stopped as the ice-water interface is destroyed.

6. If the cementing action is not sufficiently developed (i.e., in terms of adequate curing time or strength) the hydraulic pressures generated due to crystal growth may exceed the tensile strength of the pore structure. Due to a rearrangement in pore geometry and the resulting loss of confinement of the pore structure, the induced suction is increased. This leads to ice segregation, further volume expansion, and deterioration.

7. The interrelationship between permeability and strength is modified by the degree of saturation of the sample and the freezing temperatures encountered. If an appreciable number of unfilled pores exists near the freezing front and the amount of water which may be transformed into ice is small, it will be easier for the hydraulic pressures to be dissipated. As a rough first approximation, Powers and Brownyard (17) have suggested that for a temperature of -10 C, approximately 75 percent of the soil water will form ice, with the remainder staying in a liquid phase. Walker and Karabulut (22) have suggested similarities between the soil-lime system and that for cement mortars, but the details are not given and the system is presumed to be similar to a closed system.

Accepting this general pattern, the hypothesis suggests that two separate soil properties may be used in the assessment of the durability of the stabilized specimen:

1. The strength of the cemented soil skeleton must be sufficiently developed so that it can resist the expansive forces from the excess hydraulic pressures. The tensile strength will be the most direct measurement, but the compressive strength may also be used in view of the interrelationship between these two tests.

2. The permeability of the soil must be sufficiently improved so that excess hydraulic pressure may be dissipated during the freezing period. The improvement in permeability will also be manifested by a reduced suction potential for the same moisture content. This property will be interrelated to the initial degree of saturation and curing conditions.

As a combined result of adequate strength and permeability, the hypothesis suggests that satisfactory stabilized soils are dimensionally stable. If the permeability and suction, or the degree of cementation and strength gain is inadequate, expansion occurs in the soil pores and heaving results.

Penner (15) has suggested that the application of an external pressure of 19.7 psi would be adequate to prevent frost heaving for a freezing point depression of 0.01 C. In subsequent experimental work (16), he has shown that for potters flint, in a closed system, the application of approximately 1 kg/cm^2 (14.2 psi) of an equivalent overburden pressure was sufficient to reduce heaving. Hoekstra (8) measured the heaving pressure developed in frost-susceptible soils. For the marginal to non-frost-susceptible soils, a heaving pressure of 20 psi was developed, with greater values recorded for the more susceptible soils. Thompson (21) has shown that tensile strengths ranging from 20 to 200 psi may be anticipated for lime-stabilized soils.

Although the mechanism for heaving in a stabilized soil may be apparently different from that for unstabilized soils, the development of an internal tensile strength may not be too different from the application of an external pressure to the freezing process.

It was necessary to establish an overall survey of the problem before any detailed examination of an individual component began. For example, the permeability criterion would obviously be related to the laboratory freezing rate selected and the method of sample preparation, and a more rapid and severe freeze would require a higher permeability. The compressive strength parameters had been used in other investigations, and the length-change test has been used for studies of soil cement (Packard and Chapman, 14). For these reasons, the laboratory work concentrated on the strength-dimensional stability values with auxiliary tests performed for saturation, moisture content distribution, suction, and permeability.

TESTING PROGRAM

Although the number of durability investigations for lime-stabilized soils is limited, the work of Preus (19) in Minnesota, and of Davidson and his associates in Iowa has shown that the following trends may be anticipated: (a) increasing lime percentage, under comparable curing conditions, reduces lineal expansion; (b) a minimum curing time is necessary before the stabilized soil is capable of withstanding freezing and thawing conditions; and (c) the individual behavior of a stabilized soil could not conveniently be related to a single property such as plasticity.

To relate studies of Canadian soils with previous research, it was decided to perform tests essentially similar to those used in earlier studies. It was also decided to obtain a broad range of soil types for the study, and to use a consistent lime percentage, rather than using only one or two soil types with variations in the lime content and curing procedures. The apparent advantages of this procedure were (a) a representative range of mineralogical and particle size distribution for Canadian soils would be obtained, and

TABLE 1

PHYSICAL PROPERTIES OF 14 SOIL SAMPLES INITIALLY SELECTED FOR LIME STABILIZATION-FROST INVESTIGATION

Sample No.	Location	L.L.	P.L.	P.I.	Sand (%)	Silt (%)	Clay (%)	Unified Class. Symbol	Activity[a]	Predom. Clay Mineral[b]
A1[c]	90 mi NW of Edmonton	90	22	68	25	22.5	75	CH	0.91	M, I, Ch
A2	Drumheller	107	25	82	0	12	88	CH	0.93	
A3[d]	40 mi NW of Edmonton	34	12	22	45	28	27	CL	0.82	
S1[d]	15 mi N of Regina	32	12	20	39	40	21	CL	0.96	
S2[c]	40 mi S of Regina	39	13	25	47	31	22	CI	1.14	M, I-K
S3	Regina	81	22	59	5	28	67	CH	0.88	
M1	60 mi NW of Wpg.	62	17	45	15	30	55	CH	0.82	
M2[c]	20 mi N of Wpg.	79	23	56	5	24	71	CH	0.79	M, I, K-Ch
M3[d]	Winnipeg	106	26	80	0.5	7	92.5	CH	0.86	
O1[c]	London	38	17	21	10	46	44	CI	0.48	K-I, Ch
O2	Haileybury	52	20	32	17	41	42	CH	0.76	
O3[d]	Welland	70	24	46	5	23	72	CH	0.64	
Q1[c]	Quebec City	36	18	18	2	66	32	CL	0.56	
Q2[d]	Montreal	52	21	31	18	46	36	CH	0.85	

[a]Activity = P.I./clay fraction.
[b]M = montmorillonite, I = illite, Ch = chlorite, K = kaolinite.
[c]5 soils selected for preliminary investigation, 1962-1963 (stage I).
[d]5 soils selected for verification analysis, 1963-1964 (stage II).

(b) secondary mineralogical effects which may develop between a particular soil mineral and lime type might not introduce a distorted test pattern when overall results were compared to the hypothesis.

Fourteen clayey soil samples were collected from widely distributed points after examination of geological and pedological maps, and after consultation with various members of provincial highway departments. The results of the standard identification tests and predominant clay minerals are given in Table 1. Subsequently ten samples were selected for detailed testing.

The lime percentage for each soil was determined by the method suggested by Hilt and Davidson (7) by which the "lime-fixation capacity" is obtained for each soil; i.e., the lime percentage above which changes in the plastic limit are minimal (Fig. 1). A reasonable correlation could be established between the fixation capacity and the percentage of clay sizes (Fig. 2). The design percentage was then obtained by adding an additional amount (+4%) above the fixation value for each of the soils.

Current construction practice allows the lime to be mixed and blended with the soil for a period of at least 24 hr before compaction, and this practice was followed for each sample at the design lime percentage. In addition, compaction is usually specified at 100 percent standard density at optimum moisture content conditions. Although the working hypothesis suggests that initial degree of saturation and dry unit weight (or strength) are important variables, it was decided to test the samples at the usual specified design conditions. Subsequent studies could be used to establish what field limits can be tolerated. The results of the standard compaction tests for the untreated and treated soils, along with the design percentages of lime, are given in Table 2.

The specific freeze-thaw method selected consisted of several modifications to the British freeze-thaw method (Fig. 3). The test simulates the "open-ended" freezing

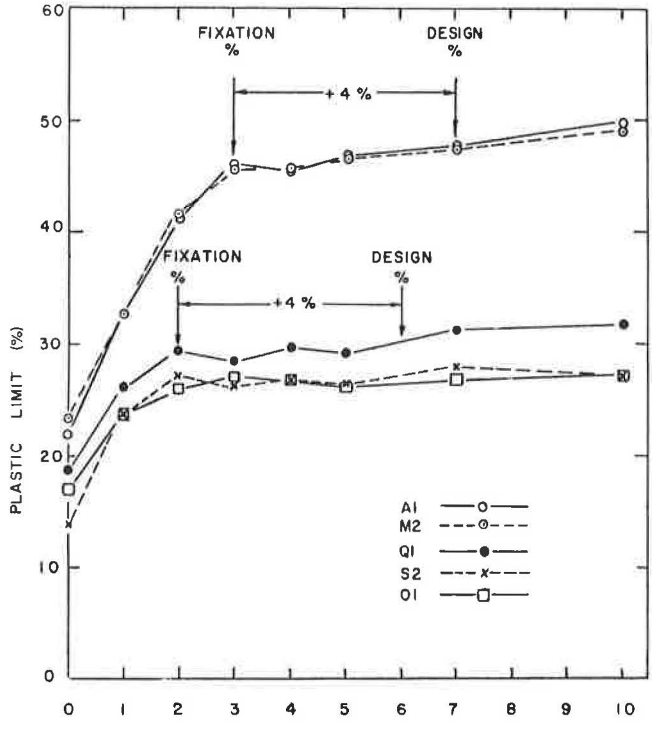

Figure 1. Relationship between plastic limit and percent lime illustrating method of selecting design percentage for stage 1.

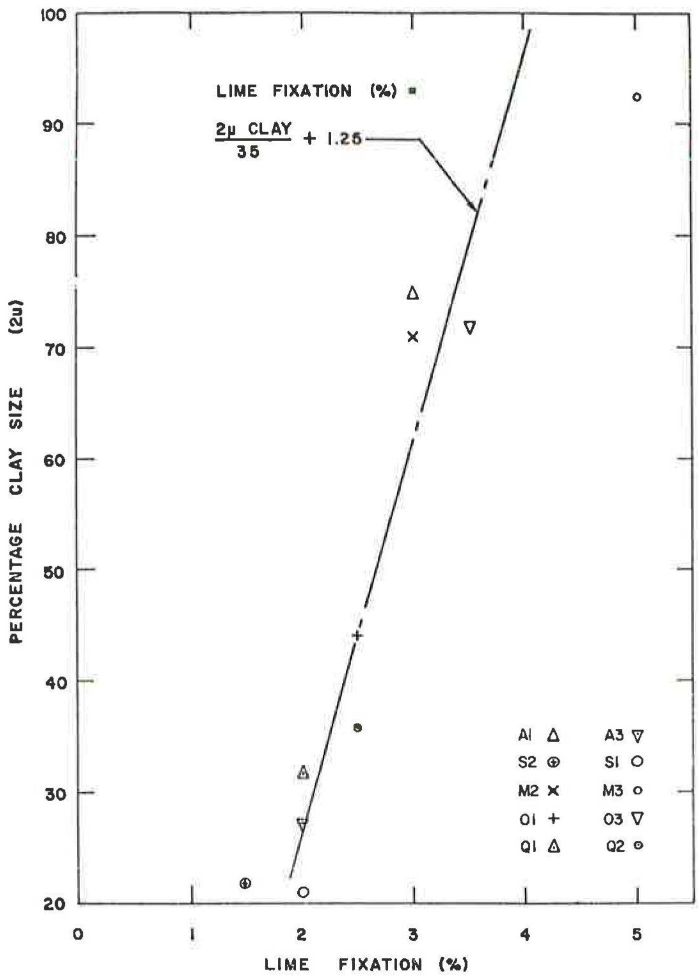

Figure 2. Relationship between fixation lime percentage and clay-size content.

TABLE 2

STANDARD COMPACTION TEST RESULTS AND DESIGN PERCENTAGES OF LIME

Sample No.	Percent Lime		Untreated Soil		Treated Soil	
	Fixation	Design	Std. Density (pcf)	Opt. Moist. (%)	Std.	Opt.
A1	3	7	91.5	23.0	83.0	30.0
A3	3	6	114.8	13.5	107.8	16.0
S1	2	6	124.0	12.0	110.0	15.0
S2	1.5	6	103.0	18.0	99.5	17.0
M2	3	7	89.5	25.0	84.0	35.0
M3	5	7	88.0	28.0	80.4	28.0
O1	2.5	6	109.5	17.0	101.0	22.5
O3	3.5	7	94.6	18.0	87.3	26.0
Q1	2	6	108.4	17.0	98.7	24.5
Q2	2.5	7	101.5	23.0	91.8	20.0

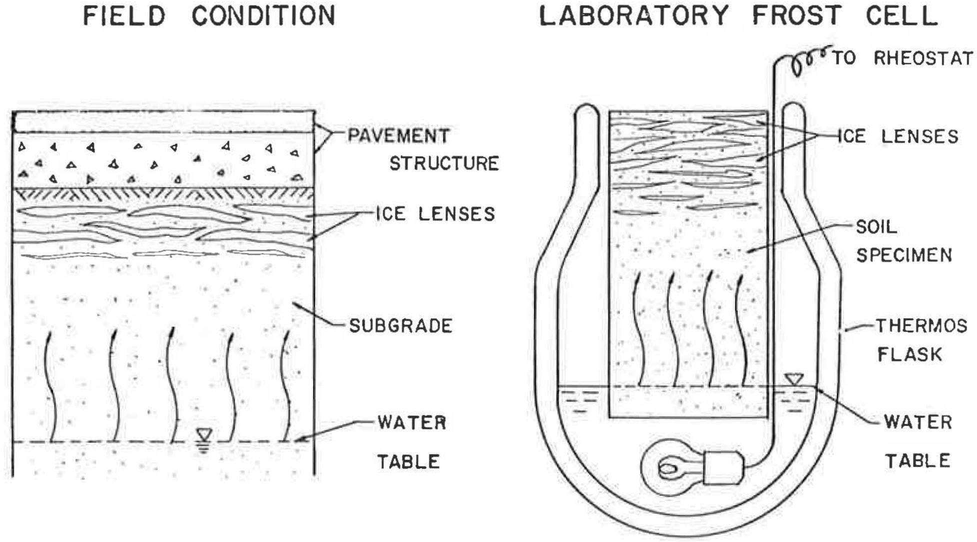

Figure 3. Design of laboratory frost cell to simulate field frost condition.

conditions that occur in the field, and has been correlated by Kalankamary and Davidson (11) to freezing conditions in Iowa. A cursory study of the general climatic differences between Iowa and central Canada showed that Canada has a slightly lower mean annual temperature so that some minor modifications were adopted. The main features of the freeze-thaw tests were

1. Freezing temperature was maintained at 12 F;
2. The number of freeze-thaw cycles was kept at 7;
3. The sample was subjected to 16 hr of freezing, followed by 8 hr of thawing at 72 F; and
4. An open-ended system was used, allowing additional water to reach the 3- × 2-in. diameter samples at all times.

To insure that the sample was subjected to substantial frost penetration, thermocouple measurements were taken during the initial series, and the foregoing cycle was selected so that the frost penetration was at least 2 in.

After the soil specimens had been compacted to 100 percent standard density and moisture by dynamic and static means, they were sealed in Saran Wrap, and cured at room temperature for varying periods of time. Selected samples were then subjected to freeze-thaw testing, or used as control samples. Moisture content distribution was measured in ½-in. thick layers for treated and untreated soil specimens after various periods of curing before or after the freeze-thaw cycling. In addition, length changes were measured during the tests by means of dial gages mounted on top of the specimens. Following the freezing cycles, selected samples were subjected to unconfined compression tests, tension tests, permeability tests, suction tests, and analyzed for moisture content.

EXPERIMENTAL RESULTS

Inasmuch as the samples were to be subjected to a variety of tests after the freeze-thaw cycling, an inordinate number of samples would be necessary if a statistical evaluation was required for each lime percentage and curing time for each soil. It was felt that the variety of soil types tested would present an overall pattern for this initial stage, and subsequent statistical details could be assessed by later testing.

The assessment of durability for a stabilized soil is purely subjective, especially when results are restricted to laboratory work, and a variety of criteria may be selected.

For the purposes of this research, a non-frost-susceptible stabilized soil was considered to be a sample which exhibited less than 0.02-in. heave (1.0% heave in the freezing zone) after 7 cycles of freezing and thawing under the prescribed conditions. This value was selected for the following reasons:

1. Dimensional stability of the soil is of more importance than residual strength, because in most cases in Canada lime stabilization is utilized for subgrade construction where stresses are a minimum.

2. Lineal change of freeze-thaw specimens of soil cement correlates with other assessments of durability. For a different method of freeze-thaw testing, Packard and Chapman (14) used 0.1 percent lineal change as a durability measure.

Strength Relationships

The results of the unconfined compression test determination for the ten samples are given in Table 3, and typical experimental results are shown in Figures 4, 5 and 6 for three of the soils tested. (The samples were numbered as follows: the first letter represented the particular province, this number being the sample number within that province, and the second number represented the percentage of lime added. Thus an A1-7 soil represents the first sample from Alberta with 7 percent lime added). Examination of the test results suggests that heavy and silty clays (CH) should develop a minimum strength of 200 psi before being subjected to frost action, and that clayey silts (CL) should develop a minimum strength of 300 psi.

Theoretically, the measurement of the developed tensile strength appears to be a more logical method of assessing the behavior of the cemented soil when subjected to frost action because the expansive pressures must overcome the tensile strength of the cell walls if heaving is to take place. Metcalf and Frydman (13) have shown that the tensile strength, as measured by the Brazilian or indirect tension test, is between one-twelfth and one-tenth of the unconfined compression strength for stabilized soils; Thompson has suggested a value approximately one-eighth the unconfined strength.

These previous investigations imply that the unconfined compression strength could be an adequate measure of the tensile strength; but for comparison purposes, indirect tensile tests were performed on samples of the first five soils tested. These results are also given in Table 3, and may be summarized as follows: for heavy and silty clays (CH), the minimum tensile strength should be between 20 and 30 psi; for silty clays (CL), the minimum tensile strength must exceed 45 psi.

Recently, Wissa et al. (23) have shown that the internal friction of a lime-stabilized silt remains essentially constant as the curing time is increased. A second trend showed that the cohesion intercept increases on a logarithmic scale as curing time increases, in a manner similar to that reported by Dumbleton (4) for the unconfined compression strength. Assuming that the Griffith crack theory is applicable to these brittle lime-stabilized clays, this second trend suggests that the tensile strength will also increase logarithmically with curing time. The increase in strength with time for the limited tensile and compressive strengths is shown in Figure 7 and confirms these trends. The implications of the trend suggest that lime-stabilized soils will become increasingly durable to freezing and thawing conditions as the curing proceeds, and this is also confirmed by the numerical values of lineal expansion recorded in this series. The Griffith crack theory (c.f. Jaeger, 10) also suggests that for a range of values of ϕ' from 30 to 37 deg, the tensile strength will vary from one-seventh to one-eighth of the unconfined compressive strength. A constant value is not to be expected because the ratio varies with the angle of internal friction, but this range of the ratio is generally consistent with these test results.

Permeability and Suction

The results of the unsaturated permeability tests on the first five samples are shown in Figure 8, and the method of test is given in the Appendix. In general, there was a substantial increase in the permeability for the non-frost-susceptible heavy clay soils, presumably due to the flocculation developed. The silty clay soils showed erratic or

TABLE 3

UNCONFINED COMPRESSION STRENGTH, TENSILE STRENGTH AND LENGTH CHANGE FOR LIME-STABILIZED SOIL SAMPLES[a]

Sample No.	q_u (psi)			s_t (psi)			Heave (in.)		
	14 Days Curing	28 Days Curing	4 Mo. Curing	14 Days Curing	28 Days Curing	4 Mo. Curing	14 Days Curing	28 Days Curing	4 Mo. Curing
A1-7	190	232	301	19	27	24	0.015	0.007	0.002
A3-6	130	180	456	-	-	-	0.088	0.005	-
S1-6	292	384	550	-	-	-	0.004	0.004	-
S2-6	167	228	431	17	24	49	0.258	0.005	0.005
M2-7	250	291	358	28	31	39	0.003	0.005	0.005
M3-7	130	156	112	-	-	-	0.020	0.048	-
O1-6	112	146	360	13	23	46	0.463	0.191	0.008
O3-7	191	206	307	-	-	-	0.474	0.259	-
Q1-6	110	134	289	16	20	47	0.522	0.114	0.080
Q2-7	229	268	408	-	-	-	0.004	0.004	-

[a]q_u = average unconfined compression strength from test samples, corrected by a factor of 0.96 to convert to standard length vs diameter ratio;
s_t = tensile strength as determined by the indirect tensile test.

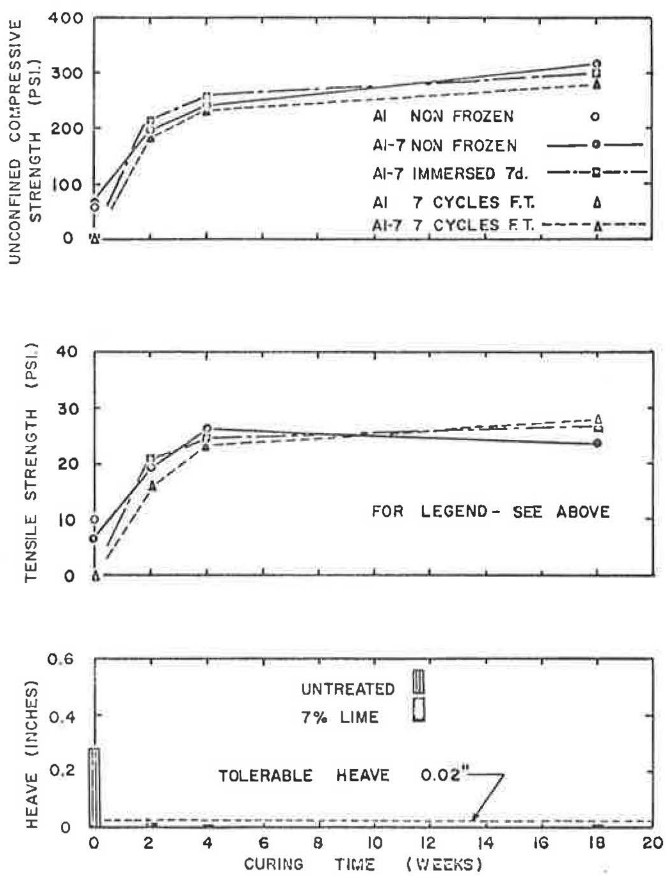

Figure 4. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil A1.

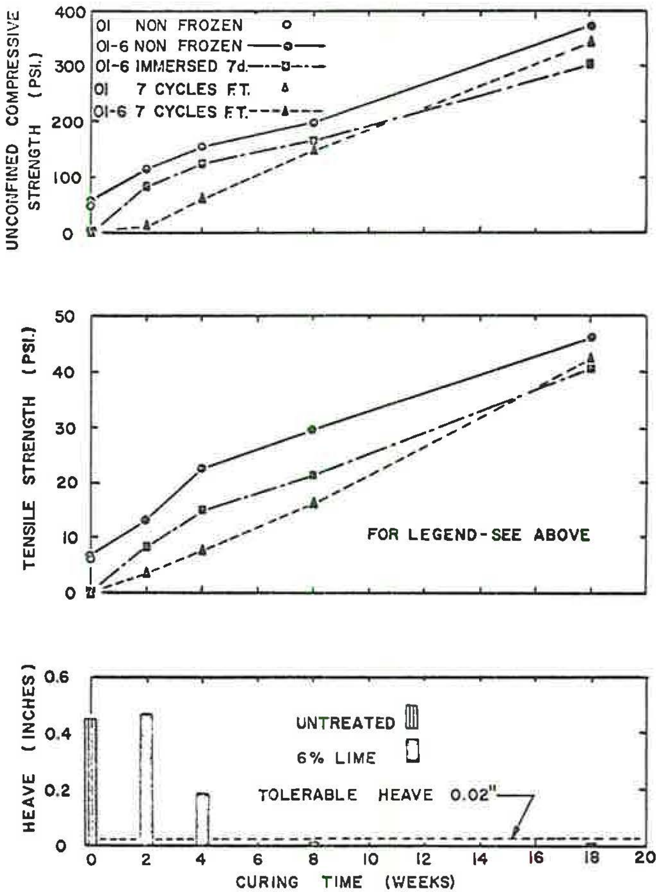

Figure 5. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil O1.

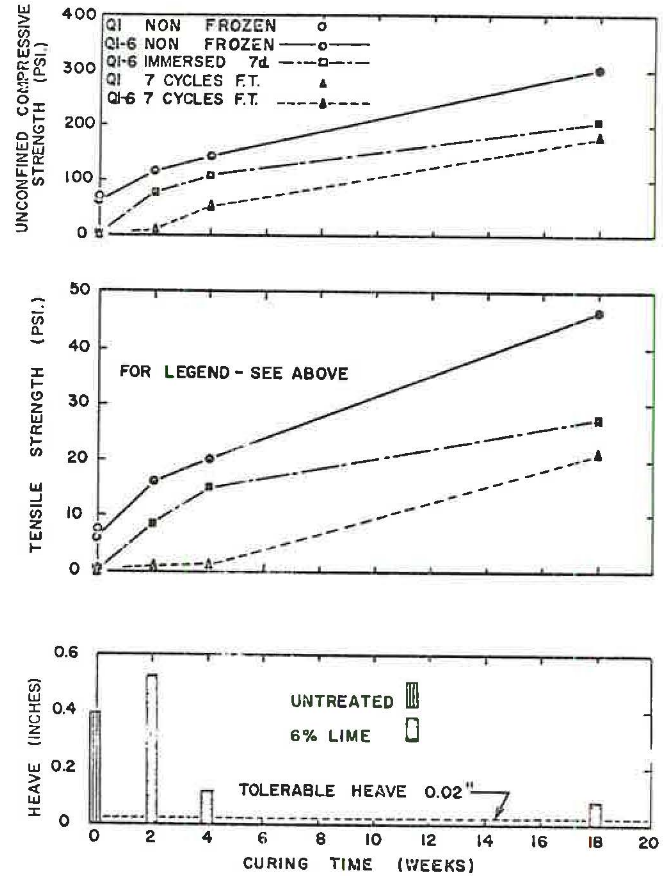

Figure 6. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil Q1.

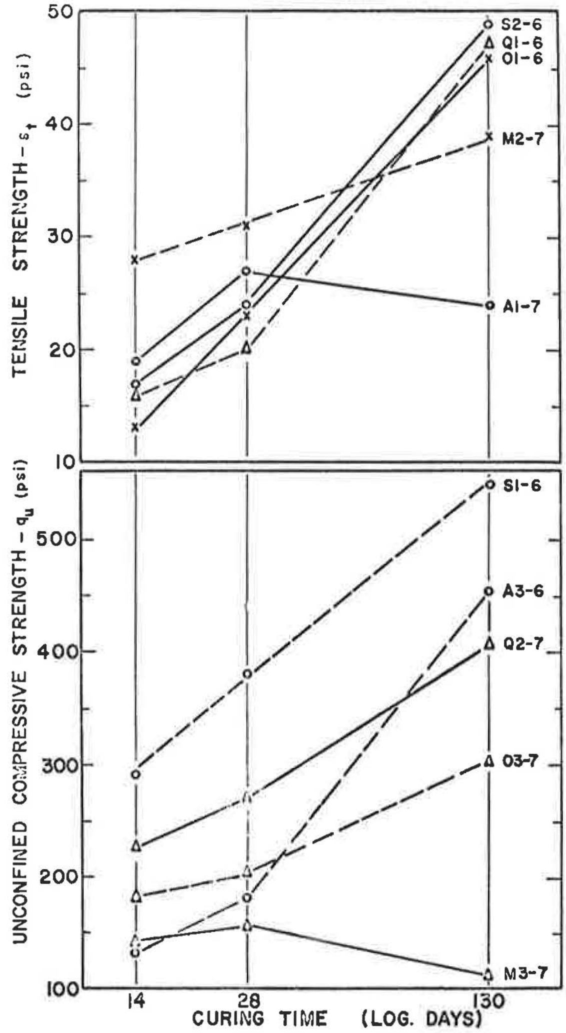

Figure 7. Relationships between unconfined compressive strength, tensile strength, and curing time.

no changes in permeability and were frost-susceptible under these conditions. The difference in void ratios between untreated and treated soils (0.10) may account for a tenfold change in relative permeabilities. The important point is that permeability changes with time for the treated samples such as S2-6 and A1-7 while at a constant void ratio. These samples developed an adequate resistance to the test freezing and thawing.

Powers (18) claimed that deterioration by frost in hardened cement pastes in concrete occurs mainly due to low permeability. It is suggested that the use of air-entraining agents has proven to be successful in providing adequate escape channels to dissipate the hydraulic pressures associated with the growing ice crystals. The pattern in the lime-stabilized soils appears to be similar.

Recently, Fossberg (6) has suggested that the application of lime to clay soils develops a lowering of the permeability; typically, a reduction from 10^{-6} to 10^{-8} in./min after 28 days curing and at a void ratio of 1.5. However, these samples were molded immediately after the addition of lime so that there was little opportunity for flocculation to develop.

Laguros (12) has reported that the permeability of a soil increased from 8.6×10^{-6} cm/sec to 6.8×10^{-5} cm/sec, although exact details of the "rotting" time are not given.

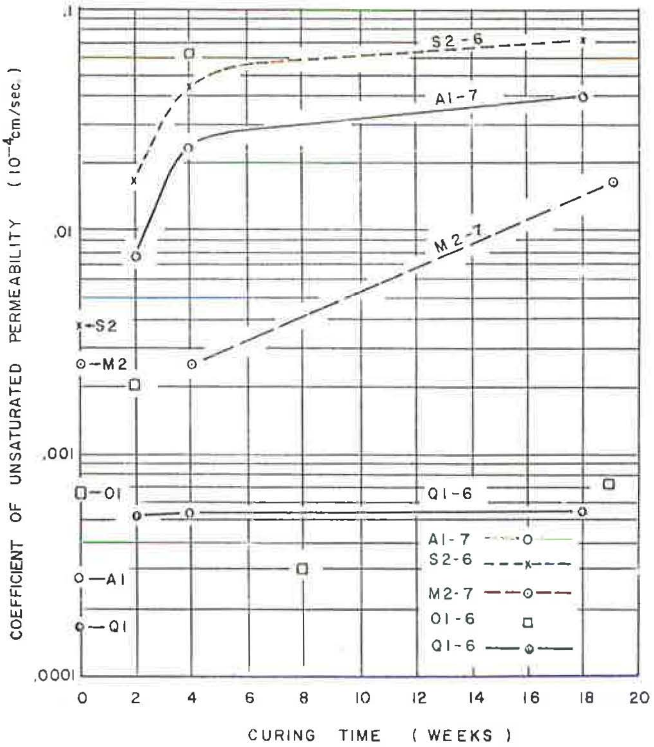

Figure 8. Relationship between coefficient of unsaturated permeability for untreated and lime-treated soils at various curing periods.

Ranganatham (20) computed the value of permeability for lime-stabilized soils from the results of consolidation tests. These samples were allowed to rot for 1 wk (and hence develop flocculation) before testing. In these cases, the permeability increased by at least 10 times, and this trend is consistent with those reported here. It is suggested that when permeability tests are conducted for lime-stabilized soils the samples should be allowed to cure in an uncompacted state similar to the conditions which develop in the field.

The results of the drying curve suction tests are shown in Figure 9, and indicate that the suction potential of the soil is decreased under the foregoing methods of sample preparation. This trend is consistent with the permeability results, as an increase in the size and number of the larger pores is indicated.

The results of Clare and Cruchley (1) and Fossberg (6) show that the suction is increased due to the addition of lime to the soil. However, in both cases, the samples were not allowed to develop any substantial flocculation so that the results are not directly comparable.

Degree of Saturation

Both the suction and permeability characteristics are affected by the initial degree of saturation of the sample. For samples with low initial saturation after molding and curing, the air voids will provide reservoirs into which the excess hydraulic pressures may dissipate, thus reducing the heaving pressures created during ice formation. The relationship between frost-susceptible and non-frost-susceptible soils and the degree of saturation before and after freezing is shown in Figure 10, and it is suggested that

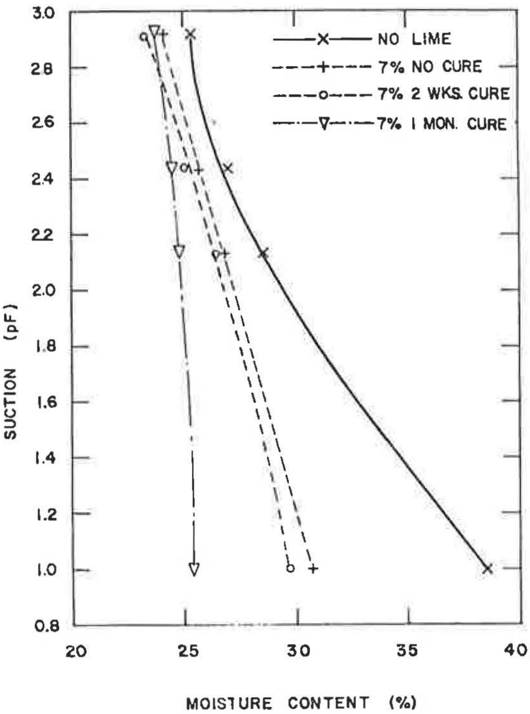

Figure 9. Suction moisture content relationships for specimens of soil Q2 with and without lime addition.

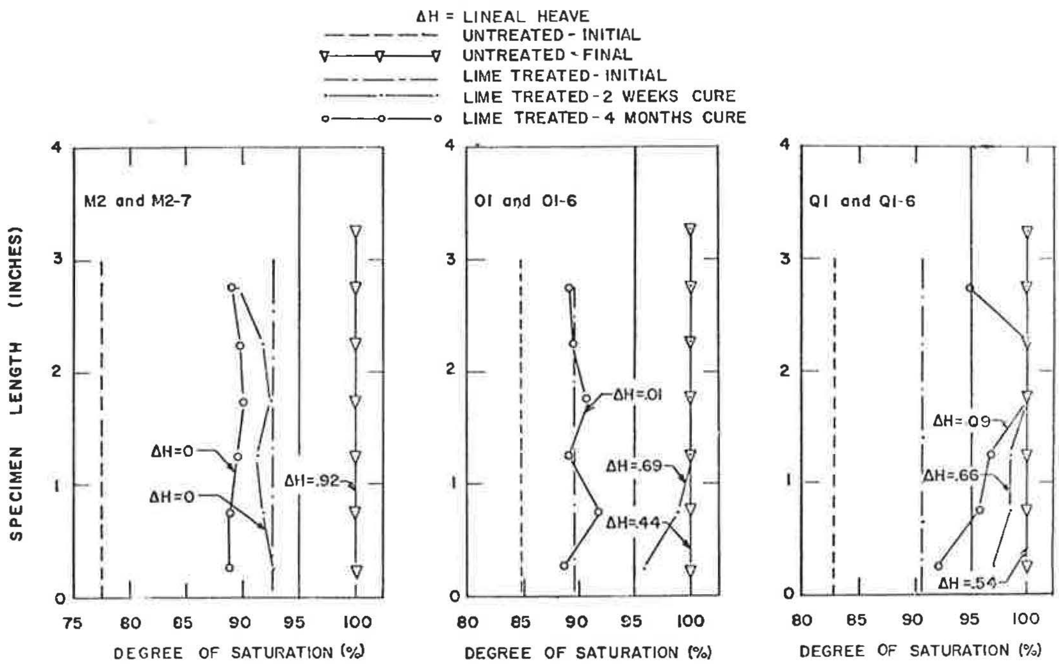

Figure 10. Degree of saturation profiles for untreated and lime-treated specimens before and after frost action.

when lime-stabilized soils possess an initial degree of saturation less than 95 percent, providing that adequate strength is developed, these soils will be non-frost-susceptible.

These results were obtained by cutting the freeze-thaw specimens into 0.5-in. layers and measuring the moisture content before and after the full cycle was completed. All samples were frost-susceptible in the untreated state. Samples M2-7 and O1-6 were non-frost-susceptible after 2 wk and 4 mo cure, respectively, whereas sample O1-6 still heaved after 4 mo. The absence of an altered moisture profile in the durable soil-lime specimens suggests that no further water is being drawn into the system.

The saturation data should be considered as tentative at this stage, because the volumetric expansion and moisture distribution will also depend on the freezing temperatures selected and the type of sample preparation.

DISCUSSION

There are several unanswered questions of substance related to the development of this hypothesis. For example, what is the exact nature of the pore-size redistribution? What influence has the pore-water alkalinity and cyclic-freezing temperature on the freezing point depression and suction?

The behavior of the soil-lime system appears to parallel that of the cement-mortar system insofar as the development of a durable soil under freezing conditions. An alteration of the soil void system is developed which indicates an increase in the number of large voids due to flocculation, and a corresponding increase in the number of small voids within the cement-gel (and within which presumably the water does not freeze). This void size distribution, when coupled with the developed strength, reduces the frost susceptibility of the natural soils.

In the absence of field data, the results of this preliminary laboratory study do not permit the establishment of limiting design criteria for use in routine highway construction. However, several important concepts have been indicated which should be useful for further laboratory research, and several design criteria are indicated which can be used for the preliminary field tests.

The experimental criterion for dimensional stability, a 1 percent length change within the frozen zone of the sample, is presumed to be adequate for subgrade conditions. This dimensional stability is developed for heavy clay samples when a compressive strength on the order of 200 psi is developed, or a tensile strength of 30 psi. These values are applicable for samples compacted to 100 percent standard density and moisture. The durable samples had unsaturated permeabilities larger than 5×10^{-8} cm/sec, but the interrelationship between strength, permeability, and freezing conditions needs to be further investigated before these values may be used for all soils.

The determination of the lime-fixation point is one of the primary steps in arriving at a suitable lime percentage. Generally, if the amount of lime used is below the percentage needed for fixation, the soil will merely be modified. This means that its plastic behavior will be altered, but that no substantial strength development will take place. If the design lime percentage exceeds the fixation point by 4 percent, the treated soils would be resistant to frost deterioration, providing that the soil activity exceeds 0.75, and the silt content does not exceed 50 percent. The modifying effect of lime on the durability of underdesigned stabilized soils, and the possibility of utilizing fly ash to promote the development of the complex calcium-silicate cements need further investigation.

The primary purpose of the research was to establish a general hypothesis for the durability of typical lime-stabilized Canadian soils. The arguments developed are consistent with trends developed elsewhere and the results obtained. It is suggested that there is a rational explanation for the arbitrary requirements for non-frost-susceptible stabilized soils, such as requiring a minimum unconfined compressive strength after cyclic freezing. The results also suggest that the reason for the differing frost behavior of various soil types lies in the amount of complex silicates which may be developed, and which in turn will influence the tensile strength and amount of flocculation.

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Appendix

TEST METHOD FOR UNSATURATED COEFFICIENT OF PERMEABILITY

The coefficient of unsaturated permeability was determined for both untreated and lime-treated soils at varying curing periods before and after frost action. The apparatus operated on the falling head principle and the essential components and assembly methods are shown in Figure 11.

The test specimen, coated with a silicone lubricant to prevent lime-grease reaction, was placed on the screen inside the lucite tube (Fig. 11). The bottom of the tube containing the specimen on the screen was then immersed in about ¼ in. water so that the water level was even with the bottom of the specimen. Preparatory investigations using only wax around the entire specimen proved unsuccessful because wax shrinkage allowed water to escape along the sides of the specimen. A combination of a ¾-in. wax seal at the bottom of the specimen and a grease seal around the remaining portion of the specimen proved effective. Melted wax was then poured through a thin-stemmed funnel until it reached the level of the grease fittings located in the lower portion of the apparatus. When the wax was thoroughly set, automotive grease was forced through the four grease fittings uniformly until the specimen was completely encased.

The top of the apparatus was then assembled and de-aired water was poured into the burette until the apparatus was completely filled. Air bubbles were removed through

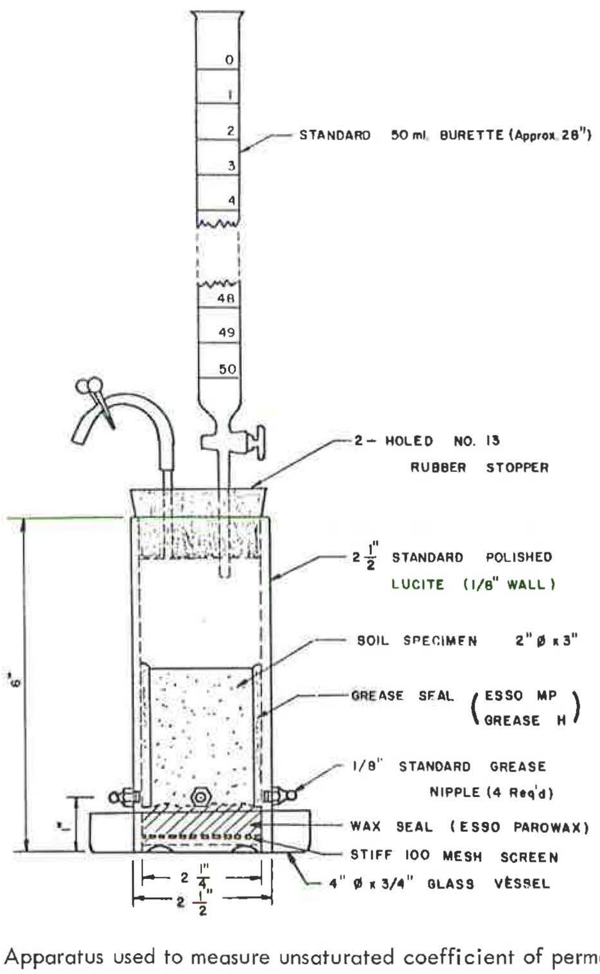

Figure 11. Apparatus used to measure unsaturated coefficient of permeability.

the vent at the top of the rubber stopper. No attempt was made to remove air from the specimens because under field conditions the degree of saturation would be less than 100 percent. De-aired water was used as a standard in all tests. Before testing commenced, the apparatus was kept in operation until steady flow conditions prevailed, generally requiring the apparatus to operate overnight. Typical test values measured ranged from 3.8×10^{-4} cm/sec to 1.7×10^{-8} cm/sec.