

HIGHWAY RESEARCH RECORD

Number | Frost Action in Soils
393

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51st Annual Meeting

Subject Areas

62 Foundations (Soils)
64 Soil Science

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HIGHWAY RESEARCH RECORD 393

Page 3, the first line of the last paragraph should read:
If θ of a V-shaped pore is other than 44 deg or 0 deg, there will be a unique position

Page 4, in Figure 1 the right-hand column in the table should read:

P: P
i: w
-0.54:-1
0:-1
+0.54:-1

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FOREWORD

Frost action in soils has probably been in progress since the formation of this planet's soil systems and most certainly predates recorded history. Geological history bears record of these ancient actions, but man's annoyance with frost boils, mud holes, and loblollies has probably been vented more to the sky than toward obtaining economical solutions until rather recent times. This RECORD discloses part of the continuing struggle to overcome the frost problems that have been gaining scientific recognition since the middle 1800s and most recently have resulted in several substantial advancements in treatment.

A more complete knowledge of the phenomenon will undoubtedly bring about more intelligent and economically feasible solutions. This symposium was dedicated to that end. The papers by Miller; Anderson and Tice; and Williams deal mainly with the basic phenomena relating to secondary heaving, unfrozen water content in frozen soils, and the ice-water surface tension concept. All are but one step away from practical application and immediate utilization. Brandt; McGaw; and Penner offer practical solutions to parts of the total problem. Water repellency is an avoidance solution as is management of the water table, whereas the double testing proposed by Penner provides a more rational delineation of the problem.

The paper by Dunn and Hudec extends basic knowledge in the field of rock deterioration. The Laba and Aziz paper was not included in the symposium but relates to the same general topic, especially as the problem of long-time construction on frozen soils becomes more and more current and important.

FREEZING AND HEAVING OF SATURATED AND UNSATURATED SOILS

R. D. Miller, Cornell University

Analysis of the behavior of air-water, ice-water, and air-ice interfaces shows that the apparent contact angle, γ , between an air-ice interface and the wall of a soil pore ought to vary rapidly with changes in pore water pressure, P_w , and ice pressure, P_i . This paper gives the expected relationship and shows that cooling of fairly dry soil, with freezing nucleated at one spot, should cause certain pores to fill abruptly with ice, thus depleting the water content of the surrounding soil. This conclusion agrees with available data. It is suggested that the ice pressure is slightly greater than atmospheric pressure when pores fill. Limited data available in the literature indicate that a given moist soil may or may not heave; when it does, the ice lenses apparently form some distance behind the freezing front. To explain this, the concepts of primary and secondary heaving are proposed for saturated soil. Primary heaving occurs when the base of the growing ice lens coincides with the limit of freezing, and the rate of heaving is limited by the rate of heat extraction. Secondary heaving is believed to occur when the freezing extends below the nominal base of the (visible) ice lens. Ice in the frozen pores can move, relative to the particles, as an integral part of the growing lens. A solution model of secondary heaving is used to illustrate an increase of ice pressure from the freezing front to the base of the ice lens, which will occur at a point where P_i is equal to the overburden pressure. It is contended that secondary heaving produces larger heaving pressures than primary heaving and explains why previous theories underestimated the maximum heaving pressure of saturated soil. It is inferred that whenever heaving occurs as the freezing front is descending through the soil, the process must be secondary heaving. It is suggested that the development of significant heaving pressures by unsaturated soil will ordinarily involve secondary heaving and that the pressure developed is less than that developed by the same soil when it is saturated.

•RELATIVELY little attention has been given to freezing and heaving of moist soils. An earlier paper (3) reported the failure of a hypothesis to explain heaving in unsaturated soils, which leads to a new effort to understand the results along with similar results subsequently obtained by Hoekstra (8). Pursuit of this question has led to a conclusion, not as yet found elsewhere in the literature, that the apparent angle of contact between an air-ice interface and a soil particle is a variable that depends on temperature and pore water pressure. From this, it can be understood that freezing of unsaturated soils involves a process that causes some pores to abruptly fill with ice.

In relating these conclusions to limited experimental data, a concept was encountered that suggests that the current model of frost heaving in saturated soils is too simple as a general model. It is suggested that the simple model should be called primary heaving and that this is superseded by secondary heaving as the maximum heaving pressure is developed, or when the frost line is descending through the soil. It is suggested that the development of significant heaving pressures in unsaturated soils probably involves secondary heaving.

DEFINITIONS AND ASSUMPTIONS

The following are assumed to be sufficiently accurate for the purposes of this paper.

1. The freezing point of water, in the absence of free solutes, obeys a general form of the Clapeyron equation. When the deviation of the freezing point, ΔT , from 0 C is not too large,

$$\Delta T = [(P_w/\rho_w) - (P_i/\rho_i)]/(\Delta H_f/T) \quad (1)$$

where P_w is the (gauge) pore water pressure (negative in unsaturated soils), P_i is the (gauge) ice pressure, ρ is the density of the indicated phase, ΔH_f is the heat of phase transition, and T is the absolute temperature. The pore water pressure is as it would be as measured by a Tensiometer and does not represent the actual fluid pressure within an adsorbed water film.

2. The Kelvin equation for the pressure discontinuity at a curved phase boundary may be written for an air-water interface as

$$P_w - P_a = 2\sigma_{aw}/\bar{r}_{aw} \quad (2a)$$

where P_a is the pressure in the soil air, hereafter taken to be zero (gauge), σ_{aw} is the surface tension of the air-water interface, and \bar{r}_{aw} is the mean curvature of the interface, taken to be positive when the curvature is centered on the water side of the interface.

For an ice-water interface,

$$P_i - P_w = 2\sigma_{iw}/\bar{r}_{iw} \quad (2b)$$

where the mean radius of curvature is taken to be positive when centered on the ice side of the interface.

For an air-ice interface,

$$P_i - P_a = 2\sigma_{ai}/\bar{r}_{ai} \quad (2c)$$

where the mean radius of curvature is taken to be positive when centered on the ice side of the interface. It will be assumed that the curvature of the air-ice interface in soil pores changes rapidly through sublimation or condensation when there are exposed areas of adsorbed film water very close by.

3. When a soil particle has an adsorbed film of liquid water, it will be assumed that the air-water interface will always meet the film at an angle of 0 deg. It will be assumed that an air-water interface will always meet an air-ice interface at an angle of 0 deg. Finally, it will be assumed that an ice-water interface will always meet an adsorbed film at an angle of 180 deg. The first assumption is commonplace. The second seems plausible, if not inevitable. The third is consistent with experimental results if the others are true (9).

4. It will be assumed that $\sigma_{aw}:\sigma_{iw}:\sigma_{ai} = 2.20:1:3.20$. The relationship between σ_{aw} and σ_{iw} is based on the experimental results of Koopmans and Miller (9). The relationship with σ_{ai} follows from the assumed contact angle between the air-water and the ice-water interfaces. If the air-water interface had had its handbook value for 0 C in the experiments cited, the respective values would be related as 72.3:33.1:105.4. If the real value of σ_{aw} was some 5 or 10 percent lower, as it probably was in the absence of extraordinary precautions in the soil system involved, then the other values should be reduced by the same factor, thus keeping the ratios the same.

APPARENT CONTACT ANGLE OF THE AIR-ICE INTERFACES
WITH A SOIL PARTICLE

Given the assumptions in the preceding section, the expected configuration of the junction of the surface of mineral particle (having an adsorbed film) with an air-ice interface can be constructed as a function of P_i and P_w or of P_w and T as they are related to P_i in Eq. 1. Three examples are shown in Figure 1. The first is constructed

to give an apparent angle of contact, γ , of 90 deg, and it is seen that the ice pressure must be intermediate between the air pressure and the pore water pressure, i. e., less than atmospheric pressure. The second is constructed for equality of air and ice pressures, for which $\gamma = 68$ deg. The third is constructed for radii of curvature for air-water and air-ice interfaces that are of the same relative magnitudes as the first example, but where the sign of the air-ice curvature is reversed; i. e., the ice pressure is greater than atmospheric pressure, and the angle γ is 57 deg.

These constructions are for two-dimensional surfaces, for which the mean curvature is twice the radius of a circular cross section shown in the figure. From inspection of the figures it can be shown that

$$\gamma = \cos^{-1} \left[\frac{|\bar{r}_{aw}| - |\bar{r}_{iw}|}{|\bar{r}_{aw}| + |\bar{r}_{iw}|} \right] \quad (3a)$$

or, with substitutions from Eqs. 2a, 2b, and 2c,

$$\gamma = \cos^{-1} \left[\frac{(P_i/P_w) - (1 - \sigma_{iw}/\sigma_{aw})}{(P_i/P_w) - (1 + \sigma_{iw}/\sigma_{aw})} \right] \quad (3b)$$

In Eq. 3a, absolute values of \bar{r} are used to avoid confusion arising from sign conventions. Equation 3b is shown graphically in Figure 2.

AIR-WATER-ICE EQUILIBRIUMS IN PORES

Haines (7) described the accepted model of drying and wetting of soils in which the relationship between water content and P_w is controlled by pore geometry and surface tension of the air-water interface and by the history of wetting and drying. In principle, a similar exercise could be performed with the air-water-ice system for freezing and thawing of pore ice in unsaturated soil. This is a much more difficult task because of the additional phase and the variable contact angle for the air-ice interface and soil particles as described in the preceding section. At this time, only some general conclusions and relationships will be mentioned.

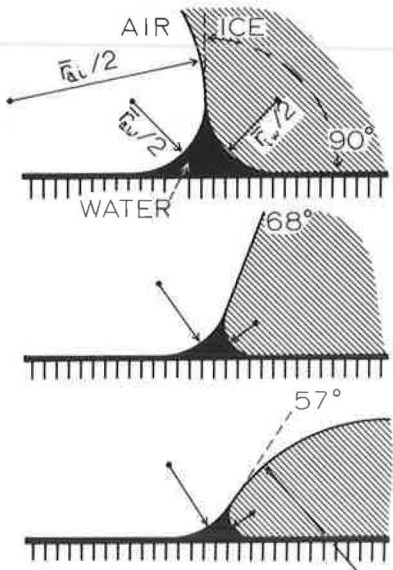
Inspection of Figure 1 shows that air and ice can coexist in a plane-parallel or cylindrical pore only if $\gamma = 90$ deg, $\bar{r}_{iw} = -\bar{r}_{aw} = d/2$ where d is the pore diameter, and the air-ice interface is reduced to a single point that the other two interfaces also have in common. [The interfaces (Fig. 1) in symmetric 2-dimensional pores can be visualized by laying the edge of a vertical mirror across the figure and rotating until the air-ice interface and its image make a smooth curve.] With the exception of this unique combination, the pore must be filled with ice, water, or air, depending on P_w and T .

Consider next a V-shaped pore (two dimensional) with an apex angle, θ , of $2 \times (90 \text{ deg} - 68 \text{ deg}) = 44$ deg (Fig. 3). For convenience, pores with apex angles larger than 44 deg will be referred to as wide pores, whereas those with apex angles smaller than 44 deg will be referred to as narrow pores.

The 44-deg pore is unique because, whenever P_w and T are such that $P_i = 0$ in Eq. 1, the air-ice interface will be flat and will intersect the respective pore walls at 68 deg without regard to how full the pore is. Thus, the width of the interface does not enter into the determination of permissible combinations of P_w and T at which air, ice, and water can coexist in the 44-deg pore.

If θ of a V-shaped pore is other than 68 deg or 0 deg, there will be a unique position of the interface in the pore determined by \bar{r}_{ai} and γ . In a narrow pore ($\theta < 44$ deg), the ice pressure can never equal or exceed atmospheric pressure when air is present in the pore. Conversely, in a wide pore, the ice pressure can never equal or be less than atmospheric pressure if air is present. Thus, in unsaturated soil, ice cannot form in a wide pore until all narrow pores have filled with ice, if freezing is nucleated in each.

Figure 1. Junction of (2-dimensional) air-ice, ice-water, and air-water interfaces at a surface having an adsorbed film of water.



	γ	$\bar{r}_{ai} : \bar{r}_{iw} : \bar{r}_{aw}$	$P_i : P_w$
Top	90 deg	-2.67 : +1 : -1	-0.45 : -1
Middle	68 deg	∞ : +0.45 : -1	0 : -1
Bottom	57 deg	+2.67 : +0.29 : -1	+0.45 : -1

Figure 2. The apparent angle of contact, γ , of an air-ice interface with a film of water adsorbed on a particle as function of ice pressure and pore water pressure.

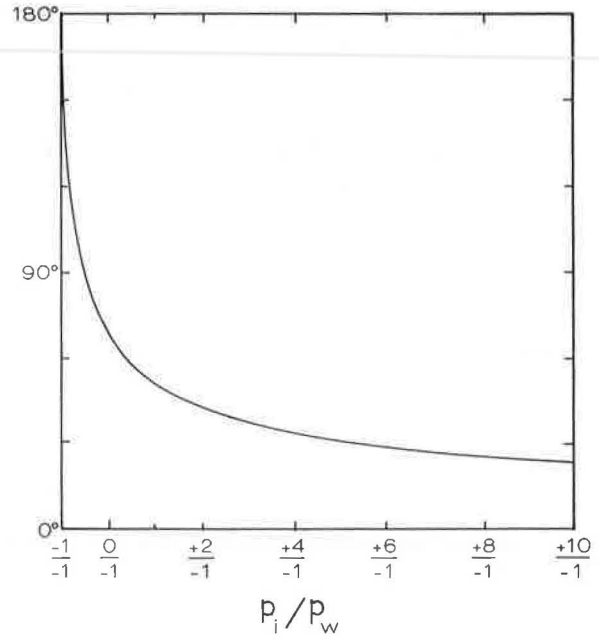
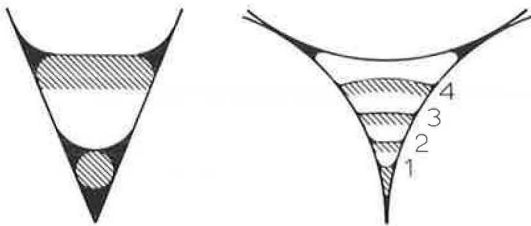


Figure 3. Air, ice, and water in pores. Left: Two of an infinite number of positions possible in a 44-deg pore when $P_i = 0$. Right: Stages of growth of an ice wedge as T decreases; P_w held constant.



Inspection of Figure 1 (with the aid of a mirror) shows that a contact angle of $\gamma = 57$ deg cannot be achieved in a V-shaped pore if $\theta < 2 \times (90 \text{ deg} - 57 \text{ deg}) = 66 \text{ deg}$, which is to say that the ice pressure in a 66-deg pore cannot exceed that which satisfies the ratio $P_i: P_w = +0.45: -1$. If we put this in more general terms, $\gamma \geq (90 \text{ deg} - \theta/2)$, and the limit of the ratio $P_i: P_w$ is as

$$\left[1 + \left(\frac{\sigma_{iw}}{\sigma_{aw}} \right) \frac{(\sin \theta/2) + 1}{(\sin \theta/2) - 1} \right]: [-1] \quad (4)$$

If the equilibrium conditions in the critical pore ($\theta = 44 \text{ deg}$) are perturbed, the results can be anticipated. If T is lowered (and the water content is held constant), P_i and γ must remain constant, and P_w must decrease. If T is lowered slightly (and P_w is held constant), the ice phase will diminish and disappear, leaving only water appropriate to the imposed P_w .

In a narrow V-shaped pore ($\theta < 44 \text{ deg}$), we can flatten the concave air-ice interface and decrease γ by lowering the temperature while holding P_w constant. Both effects require the pore to fill with ice until the width of the interface is sufficient to satisfy the new curvature and contact angle, at which point the interface will come to rest.

In a wide V-shaped pore, we can increase the convexity of the air-ice interface and decrease γ by lowering the temperature while holding P_w constant. Again, both effects will be satisfied by increasing the volume of the ice phase to some new rest position of the interface. In both, a small change in temperature may have a large effect on the volume of ice formed.

The generalized shape of a soil pore may begin with a very narrow angle at the point of contact between two particles, with the θ increasing rapidly away from the point of contact; however, the angle of the walls converges again on entering the open space between several particles. In a pore that diverges with distance away from a particle contact point, there is a transition from a narrow pore to the critical 44-deg angle to a wide pore.

Consider a pore with low water content, in which the air-water interfaces (before cooling) lie deep in the crevices (in the "narrow" region). Assume that the pore water is in equilibrium with water in a much larger body of soil such that the changes in P_w in one pore are buffered by the surrounding soil. Let the soil be cooled until stable ice can be formed in one water wedge, which is isolated from its neighbors and interfaces with them only via adsorbed films and the vapor phase, such that the nucleation of ice at the desired spot does not nucleate freezing elsewhere. As the temperature is gradually lowered, the ice wedge will grow (Fig. 3), and the ice-water interface rises in the crevice. The ice pressure will rise (assuming P_w stays constant) toward atmospheric pressure and beyond. However, at some point, any of three events may occur that will cause the entire pore to rapidly and irreversibly fill with ice: The periphery of the growing ice wedge may reach a neighboring water wedge, nucleating freezing there; the air-ice interface may encounter the surface of a particle on the far side of the pore; or the residual air space may become so narrow that it fills by a Haines jump. Both of the first two events are equivalent to "narrowing" the pore in terms of the divergence of the particle surfaces intersected by the air-ice interface, and, if the temperature is held constant, the pore must fill with ice. In the third, the water imbibed will promptly freeze. Because ice will reach atmospheric pressure in a two-dimensional wedge where the divergence of the walls is only 44 deg (and still deeper in a wedge between two spherical particles—where another component of curvature exists), it seems almost certain that the ice pressure will exceed atmospheric pressure before most pores can fill with ice. It appears, however, that most pores will fill at an ice pressure only slightly greater than atmospheric pressure.

The abrupt filling of a pore will cause an abrupt drawdown of P_w in the surrounding pores and a local rise of temperature. Both have the effect of reducing P_i until P_w and T return to ambient values, which delays the filling of neighboring pores with ice that is nucleated by the freezing of the first pore. As conditions return to normal, the

surrounding pores will follow the same sequence and will fill. The net result, then, is a growing body of ice-filled pores surrounded by ice-free pores that are being depleted of water. This expectation agrees with experimental observations (3, 8, 9).

If the soil is fairly wet at the outset such that water wedges are already in contact, it would seem that ice could propagate ahead of the ice-filled pores; however, as pores to the rear of the freezing front fill with ice, the drawdown will shift conditions toward those described for the freezing of a drier soil and the freezing front will be overtaken by the zone of filling. The freezing of fairly wet soils has not been visualized in detail.

It is to be expected that some pores will be bypassed without filling. The bypassed pores remain as sinks of water, buffering restoration of P_w . When a column is frozen from one end, however, the pores should eventually fill except that isolated air bubbles may be cut off.

A separate exercise, which must focus on the pore necks, is required to trace the thawing of an ice-filled pore in unsaturated soil. This will not be discussed here except to remark that again "jumps" involving abrupt melting and emptying are to be expected and that they occur at higher T (or lower P_w) than the filling jump so that there will be pronounced hysteresis effects.

The above analysis apparently differs from one offered by Globus (5). He evidently saw the phenomenon as one in which the appearance of ice grains in the voids shifted the pore size distribution downward (while lowering liquid water content), thereby greatly reducing the mean curvature of air-water menisci, which lowers P_w drastically.

HEAVING OF UNSATURATED SOILS

When an ice wedge exceeds atmospheric pressure, it will tend to force the adjoining particles apart. Beskow (1) probably observed this effect in his account of the expansion of (unloaded) moist sand on freezing. He attributed expansion to the formation of columnar ice at particle contacts. In his experiments, pore filling evidently did not occur, but it might not be expected in sand unless cooling is slow enough to permit translocation of water.

The qualitative account suggests that pore filling probably occurs when the ice pressure exceeds atmospheric pressure. If all pores are filled at the freezing front, heaving should ensue if the overburden pressure does not exceed the pressure for pore filling (primary heaving, to be described later). If very many of the pores are bypassed, their tendency to contact will be in conflict with the tendency of ice-filled pores to expand. Therefore, the net effect may not produce sufficient thrust to overcome the overburden pressure, even if it is only the weight of the frozen soil itself, until the fraction filled reaches a critical value.

Dirksen and Miller (3) found that, when columns of moist New Hampshire silt were frozen from one end with a relatively small overburden pressure (about 0.04 kg/cm^2), heaving began whenever conditions were such that the degree of ice saturation of the pores reached about 0.9 in the zone of active accumulation behind the freezing front.

In somewhat similar experiments, which unfortunately precluded heaving, Hoekstra (8) apparently observed ice lens formation at about the same degree of pore saturation. For lenses to form in his system, it was necessary for the soil to consolidate; therefore, the pressures that developed may have been fairly high.

Neither experimental technique could reveal precise water and ice content profiles close to the freezing front. Dirksen (13) and Hoekstra (8) both inferred, however, that identifiable ice lenses formed slightly behind the freezing front. A photograph contained in Dirksen's paper shows many ice lenses in the frozen soil but none in a zone about 3-mm thick located behind the freezing front as the column approaches a steady state. This evidence is inconclusive, however, because the sample was removed and photographed some hours after the rate of heaving had dwindled below the threshold of measurement.

PRIMARY AND SECONDARY HEAVING

The current model of frost heaving in saturated soils is shown in schematic form in Figure 4, which includes pressure profiles sketched on the right (neglecting hydro-

static components) as if heaving were in progress and as if the unfrozen soil offered very small impedance to water flow. Heaving in accord with this model will be called primary heaving. The identifying feature is that the base of the growing ice lens coincides with the limit of freezing. It is inferred (4) that lens growth ceases when the conditions diminish the mean radius of curvature to some critical value, $(\bar{r}_{1w})_{cr}$ at which the interface penetrates the pore neck, so that the pore water freezes in situ. Thus, the maximum heaving pressure expected (from primary heaving) is

$$(P_t)_{max} = P_w + 2\sigma_{1w}/(\bar{r}_{1w})_{cr} \quad (5)$$

Although tests of this equation have been described as "encouraging" in samples where the range of neck sizes is small (4), it is not clear what happens when there is a large range of neck sizes. Penner (12), using samples with fairly narrow particle size distributions, concluded that the critical pore neck (as computed from Eq. 5 for his data) was smaller than the size of most of the soil pores when "pore sizes" were computed from desorption curves. One suspects from his data that all pore necks were larger than the computed critical radii and that the residual water involved was "wedge water." Penner did not consider this possibility but instead visualized an ice lens with an irregular front that therefore existed at irregular temperatures (owing to the temperature gradient). Because of this, the ice lens always terminated at pores small enough to block further ice penetration at the local temperature. It will now be suggested that a different situation was more probable.

Suppose that the ice does penetrate the pore system for some distance beyond the base of the nominal ice lens as shown in Figure 5. It is suggested that in such circumstances the nominal ice lens may continue to grow and that the ice in the frozen fringe will move with it, while the particles remain stationary. This phenomenon will be called secondary heaving.

The author has shown (10) that ice that fills a large pore between two filter papers moves readily despite appearing to be stationary. When supercooled water is on the opposite sides of the respective filters, an increase of the water pressure on one side will cause supercooled water to emerge from the opposite side. Concurrent melting and freezing on opposite sides of the pore causes the ice to be in motion between its stationary boundaries, which are the unfrozen films adsorbed on the filter paper and the ice-water menisci in the filter pores. Thus, in principle, the ice in the frozen fringe could also move as an integral part of the ice body that includes the nominal ice lens. Water traverses the ice-filled pores in the ice phase and goes around the obstacles in the film phase.

The next troublesome question is the alleged failure of the particles to move with the moving ice. It is suggested that there is a pressure gradient in the ice whenever it is in contact with unfrozen films and whenever a temperature gradient exists (i. e., in the frozen fringe below the nominal ice lens) and the ice pressure increases as the temperature decreases. When the ice pressure reaches the overburden pressure, the intergranular stress reaches zero and particles will move with the ice; the base of a nominal ice lens will appear at this level. When the ice pressure is less than the overburden pressure, part of the load must be borne by the particle framework, and the particles will remain stationary. Before this contention can be accepted, it is necessary to explain how there can be a pressure gradient in the ice in the frozen fringe.

SOLUTION ANALOGS OF FROST HEAVING

Because the dynamics of adsorbed films are not easily understood or agreed on, the author has found it useful, on occasion, to think in terms of an equivalent system that involves familiar and well-understood concepts (2, 11). Heaving can be simulated by using a mental analog that employs an osmometer. The simple analog for primary heaving is shown in solid lines in Figure 6. An ice lens, in the form of a weighted piston, rests on a thin layer of aqueous solution. The ice pressure is fixed by its own weight and the load; the solution must be at the same pressure. The solution rests on a semipermeable membrane in contact with pore water at a lower pressure, P_w . The pressure difference (at equilibrium) will equal the osmotic pressure, π , of the solution

$(P_i - P_w = \pi)$. At equilibrium, the temperature of the ice-water interface, the ice pressure, and the pore water pressure below the membrane must satisfy Eq. 1. The vertical tube, S, simulates the unfrozen soil, its impedance to water flow, and the distance to the water table. If the temperature is lowered, water in the solution will be converted to ice and added to the base of the ice piston, the concentration of the residual solution will rise (unbalancing the osmometer), and pore water will be absorbed by the solution (osmosis), lifting the piston. If the temperature is held at the new level, ice growth will continue until the added weight of ice increases P_i to the point where Eq. 1 is again satisfied. Note that the pore water pressure, as measured by a Tensiometer, is not the solution pressure but is the water pressure beneath the membrane. The impedance to water movement represented by the membrane and the solution simulates the impedance offered by the unfrozen films shown in Figure 4.

To simulate secondary heaving requires that the analog be extended by adding a second stage (broken lines, Fig. 6). The lower stage now simulates conditions at the lower limit of freezing, and the upper stage simulates conditions at the upper (colder) extremity of continuous unfrozen films in the frozen fringe (simulated by the vertical tube, F), which terminate at the base of the nominal (visible) ice lens. At equilibrium, Eq. 1 must be satisfied at both levels, but the upper level is colder. At equilibrium, the maximum ice pressure at each level will have been reached, and this will be greater in the upper stage. At equilibrium (a steady state of heat flow upwards), there is no water movement; therefore, P_w should reach its hydrostatic value based on only the elevation above the water table (if second order effects such as thermo-osmosis are neglected). Consequently, we conclude that the ice pressure increases as the temperature decreases so long as the ice is in local equilibrium with the continuous film phase extending downward to the unfrozen soil. If the system is not in a steady state and heaving is in progress, there must be a hydraulic gradient across the frozen fringe, and P_w will decline more rapidly with elevation so that ice pressure will not rise as rapidly with decreasing temperature as in the static case.

The significant result is the conclusion that the maximum heaving pressure that can be developed during secondary heaving depends only on the temperature ΔT_o at the base of the nominal ice lens (the upper limit of the continuous films) and on the pore water pressure, $(P_w)_o$ (obtained from the hydrostatic equation for the elevation of the base of the nominal lens).

$$(P_i)_{max} = \rho_i [(P_w)_o / \rho_w + \Delta H_f \Delta T_o / T] \quad (6)$$

Confirmation of this conclusion may not be easy because it may be difficult to identify the position (and temperature) of the base of the lowest nominal ice lens, and attainment of equilibrium may be extremely slow and may require sensitive instruments and great patience.

"Tertiary" heaving could occur, in which the lowest lens gradually disappears, accompanied by the growth of an overlying lens at still lower temperature. Development of tertiary heaving pressure should not begin until the lower lens has disappeared. The ultimate maximum heaving pressure could turn out to be a constant that corresponds to the temperature at which unfrozen films disappear (or cease to conduct water) or the surface temperature if it is above this limit. It seems unlikely that the ultimate heaving pressure will be established experimentally in a reasonable length of time unless the rates of equilibration are faster than expected.

Because impedance to flow of water through the frozen fringe limits the rate of secondary heaving, increasing the temperature gradient will reduce the width of the fringe and its impedance. This suggests that the rate of secondary heaving would turn out to be inversely proportional to the thickness of the fringe, i. e., directly proportional to the temperature gradient in the fringe, an effect not to be confused with thermo-osmosis. It may be, however, that the heat conductivity of the frozen fringe may be the limiting factor in water transport in the frozen fringe (10), leading to the same result.

Figure 4. Primary heaving in saturated soil.

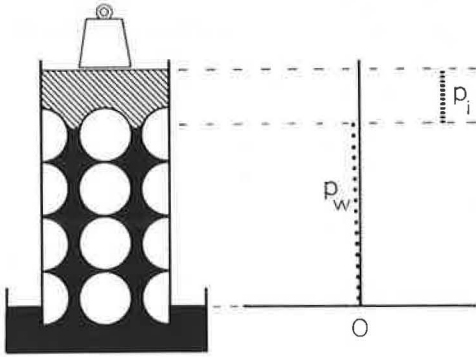


Figure 5. Secondary heaving in saturated soil.

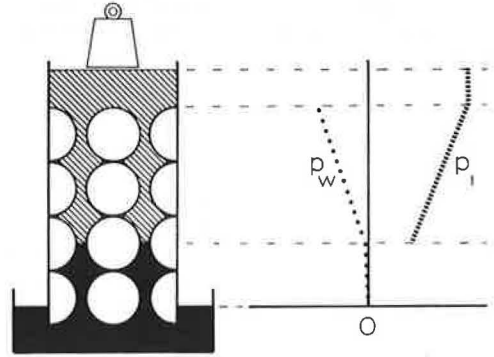


Figure 6. The solution (osmometer) analog of primary frost heaving (solid lines) with an added stage (dashed lines) to simulate conditions of secondary heaving.

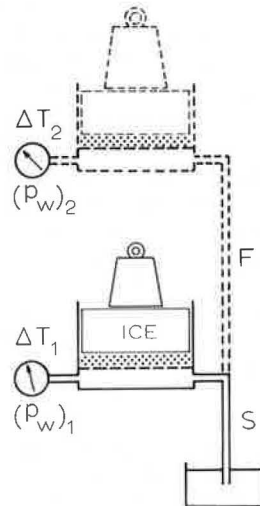
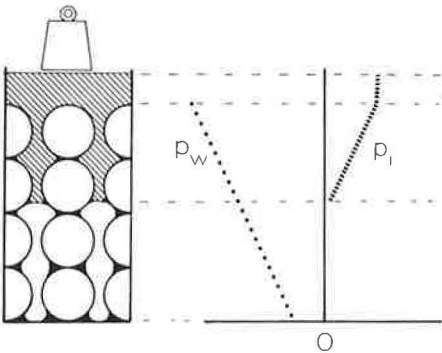


Figure 7. Secondary heaving in unsaturated soil.



HEAT OR WATER CONDUCTION LIMITATIONS ON RATE OF HEAVE

If, as proposed, primary heaving is identified with the absence of pore ice below a growing ice lens, the frost line will remain stationary because it is highly improbable that it will ever be cold enough below the lens base for spontaneous nucleation of a new ice lens at a lower depth. In other words, the rate of primary heaving should be directly proportional to the net rate of heat extraction at the frost line; i. e., heat extraction is the limiting process in determining the rate of primary heaving.

When transmission of water to the lens is the limiting process and the frost line descends with formation of successive ice lenses, the appearance of new lenses must signify at least some degree of ice penetration of the pores beneath the lowest visible lens. Hence we would infer that, whenever rate of heave is limited by water conduction and the frost line is progressing downward, secondary heaving is taking place. If this is true, secondary heaving is neither rare nor necessarily slow.

SECONDARY HEAVING IN UNSATURATED SOIL

Secondary heaving in unsaturated soil is shown in Figure 7 as if it were in progress. It differs from secondary heaving in saturated soil in that the frozen fringe terminates at an air-ice interface instead of an ice-water interface. During active freezing, the lowest water content will occur in the ice-free soil next to the freezing front. If a steady state is approached, with the temperature gradient becoming stationary, the water content gradient will ultimately reverse because of the effect of the temperature gradient in the unfrozen soil once active adsorption of water by the frozen soil has ceased. Distillation of water from the warm to the cold end of the unfrozen soil sets up a circulatory motion of the water, which returned toward the warm end in the liquid phase (6). Thus, P_w at the base of the frozen fringe depends on a more complex phenomenon than the position of the water table in the saturated soil. The maximum heaving pressure may be transient if P_w at the freezing front continues to decline as a steady state is approached. The ultimate steady-state heaving pressure developed by secondary heaving in unsaturated soil depends on the temperature at the base of the lowest nominal ice lens and on the P_w that can be sustained at the contact between the frozen fringe and the unfrozen soil in a steady state. It is anticipated that increasing the temperature gradient will increase this P_w , as will increasing the length of the unfrozen soil.

The maximum heaving pressure that can be developed by unsaturated soil will be less than that which can be developed in saturated soil but may, nevertheless, be sufficient to have important engineering and agricultural consequences. In fact, it is probably a far more extensive source of problems in agriculture than is heaving of saturated soils.

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PREDICTING UNFROZEN WATER CONTENTS IN FROZEN SOILS FROM SURFACE AREA MEASUREMENTS

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An empirical equation relating the unfrozen water content of partially frozen soils to temperature and specific surface area has been devised. In spite of the simplifying assumptions involved, this prediction equation is of sufficient accuracy for normal engineering requirements. Predicted unfrozen water contents (phase composition) computed from this equation are compared with experimentally obtained values for nine soils and two surface-active soil constituents and are found to be in good agreement, particularly at temperatures below -5 C (23 F). Possession of this prediction equation permits more refined calculations of frost and thaw penetration under natural or disturbed conditions and calculation of the time-dependent temperature distributions associated with foundations, footings, production oil wells, and pipelines.

•THE evidence establishing the existence of a continuous, unfrozen water phase separating ice from the mineral or organic matrix in frozen soils has come from many sources and is now a widely accepted fact (3, 4, 6, 16-19, 20, 23-25, 27-31). Questions regarding the mobility of the interfacial water, the nature of the ice phase, and the factors governing the quantity of water remaining unfrozen under given conditions, at least in qualitative terms, have been answered. The unfrozen, interfacial water behaves as a two-dimensional liquid, with water molecules and solutes freely mobile in this zone (13, 14, 15, 22). NMR spectra show decreasing, but significant, molecular mobility down to -40 C . Although it was once claimed that anomalous ice phases might exist in frozen soil, in all instances so far reported the ice formed is normal, hexagonal ice I (2). The principal factor determining the quantity of water remaining unfrozen in frozen soil is temperature (23). Pressure is a less important, but significant, factor; at a given temperature, an increase in pressure increases the unfrozen water content in a predictable way (18, 26, 29). Solute concentration in the interfacial zone seems to exert an effect roughly equal to that predicated from the freezing point depression (5, 8, 18). In addition to these factors, Bouyoucos (9) has pointed out that the geometrical arrangement of soil particles and the manner in which freezing was accomplished also are important.

During freezing or thawing, the associated latent heat of phase change does not appear instantaneously; instead, it is evolved or absorbed progressively as freezing or thawing proceeds. One of the major problems in performing calculations of frost penetration, or the thawing of permafrost due to heat transferred from foundations and footings, wells, and pipelines, has been a lack of understanding of how to treat the latent heat term that eventually appears. A particularly troublesome aspect of this problem has been the lack of a suitable function to adequately represent phase composition data. In this paper, we suggest a functional form and an equation that we believe will be found useful and adequate until further refinement becomes possible.

During the past decade, the various ways in which phase composition curves (unfrozen water contents as a function of temperature) could be obtained or derived from direct, experimental measurements as well as from other observations by indirect methods have been investigated (3, 7, 11, 20, 23, 28, 29, 30). In general, these methods yield comparable values for samples of a given soil, provided that sample histories are the same. Although several important questions remain to be investigated before it

can be said that our understanding is sufficient for present needs, an important and perhaps basic relationship has emerged: the dependence of unfrozen water contents on the specific surface area of the sample. The beginning of this idea can be traced back to Bouyoucos' (9) early work in which he related unfrozen water contents to the general classification of soil water, which in turn was known to be related to soil particle size. It seems first to have been considered explicitly by Nersosova and Tsytoich (23), who listed the following as the main factors determining the phase composition of frozen soil: (a) temperature, (b) pressure, (c) specific surface area of the solid phases, (d) chemical and mineralogical composition of the soil, (e) other physicochemical characteristics, especially the nature of exchangeable cations, and (f) solute content and composition. Of these, temperature was established as the dominant factor for a given soil, but the relative importance of those remaining was left undetermined. Dillon and Andersland (11) were next to focus on the importance of the interfacial area. They constructed a prediction equation that explicitly involved the specific surface area of the soil matrix, the plasticity index, and a defined "activity ratio." Although they did not emphasize it, the latter two parameters are closely related to the specific surface area of the soil mineral grains. Thus, it has become clear that specific surface area is a major determinant of the phase composition of frozen soils.

In an earlier paper (7), we reported unfrozen water content (phase composition) data that were determined by using the isothermal calorimeter method and unit surface area normalization. We have continued these measurements and now report complete phase composition data for ten representative soils and soil constituents (Figs. 1 and 2.) These data are well represented by the power curve

$$W_u = \alpha \theta^\beta \quad (1)$$

where W_u is given in percentage of dry sample weight, θ is the temperature below freezing in degrees C, and α and β are parameters characteristic of each soil. Values of α were obtained by using a least-squares regression of the experimentally measured values of W_u versus θ . These, together with the respective correlation coefficients and experimentally measured specific surface areas, are given in Table 1.

Figures 1 and 2 show the degree to which the normalization of unfrozen water content data to unit surface area unifies the results. In some respects, specific surface area is seen to be a more important factor than temperature in determining the water phase composition. This can be seen by examining Eq. 1 for silt and clay soils. The primary factors governing phase composition would be written in order of importance as

$$W_u = f(S, \theta, P, \Pi) \quad (2)$$

where S is the specific surface area, θ is temperature in degrees below 0 C (32 F), P is the confining or overburden pressure, and Π is the osmotic potential of the soil solution. In addition to these four primary factors, a complete consideration would involve such secondary factors as the nature of the fine pore geometry of the mineral grains, particle packing geometry, surface charge density and the suite of exchangeable, adsorbed ions. In this investigation, P and Π were held constant as the calorimetric, unfrozen water data shown in Figures 1 and 2 were obtained. Hence, the degree to which the curves in Figures 1 and 2 do not coincide indicate the combined effect of these secondary factors. A careful examination of the data for kaolinite and the two bentonites reveals that the data are better fit by segments of two different curves having the form of Eq. 1 (shown as dashed lines in Figs. 1 and 2). When this fact was discovered, the extensive data of Nersosova and Tsytoich were examined with the same result: segments of two power curves are required for a good fit of the data for kaolinite and montmorillonite. The basic significance of this observation is not readily apparent and is under continuing study. It is evident, in general, however, that the representation of phase composition data by a single power curve is quite satisfactory for most purposes (note the correlation coefficients given in Table 1); Eq. 1 should, however, be regarded as a first approximation, with the expectation that further refinements may become possible. There are no fundamental data on the influence of the secondary and tertiary

Figure 1. Phase composition plots (unfrozen water contents W_u versus temperature below freezing θ) for five representative soils.

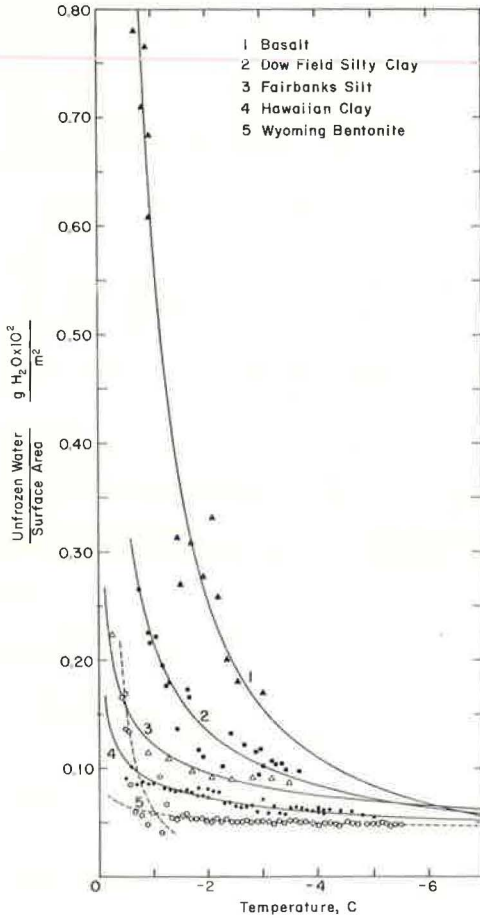


Figure 2. Phase composition plots (unfrozen water contents W_u versus temperature below freezing θ) for five representative soils.

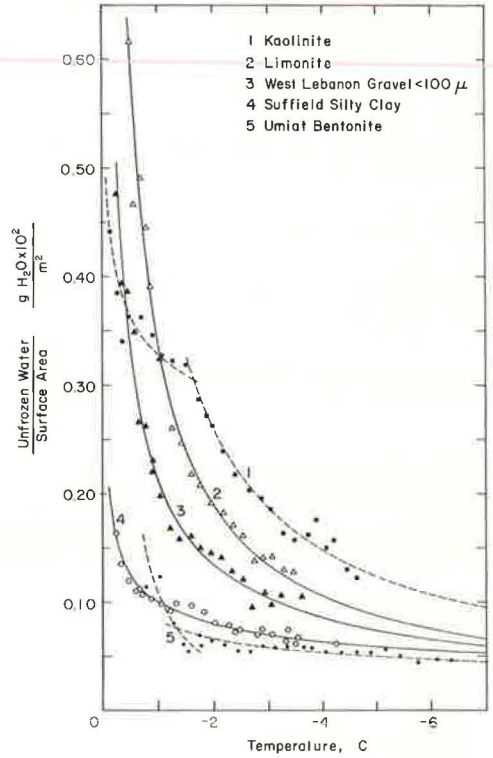


Table 1. Values of α and β , experimentally measured specific surface areas, and correlation coefficients.

Soil	Surface Area (m ² /g)	α	β	r
Basalt	6	3.45	-1.13	0.96
Rust	10	11.05	-0.80	0.95
West Lebanon gravel, <100 μ	18	3.82	-0.64	0.94
Limonite	26	8.82	-0.83	0.99
Fairbanks silt	40	4.81	-0.33	0.96
Dow Field silty clay	50	10.35	-0.61	0.94
Kaolinite	84	23.80	-0.36	0.90
Suffield silty clay	140	13.92	-0.31	0.98
Hawaiian clay	382	32.42	-0.24	0.93
Wyoming bentonite	800	55.99	-0.29	0.72
Umiat bentonite	800	67.55	-0.34	0.83

factors that influence phase compositions, and it is clear that their acquisition will be a laborious and lengthy process. In the absence of the basic relationships needed for thoughtful refinement of Eq. 1, let us explore some additional empirical correlations. Two interesting relationships appear when the values of α and β (Table 1) are regressed against the specific surface areas, S . It is found that

$$\ln \alpha = 0.5519 \ln S + 0.2618 \quad (3)$$

with a correlation coefficient of 0.90 and

$$\ln(-\beta) = 0.2640 \ln S + 0.3711 \quad (4)$$

with a correlation coefficient of 0.86.

Additional phase composition data are not likely to alter significantly the two relationships expressed in Eqs. 3 and 4. Therefore, one can use Eqs. 1, 3, and 4 to obtain

$$\ln W_u = 0.2618 + 0.5519 \ln S - 1.4495 \ln \theta^{-0.264} \quad (5)$$

a convenient computation form of a general phase composition equation.

An indication of the accuracy of unfrozen water contents predicted by Eq. 5 is given in Table 2, which contains unfrozen water contents calculated for $\theta = 1$ and $\theta = 10$. The differences in the observed values (obtained from Eq. 1 and the appropriate values of α and β from Table 1) and the calculated values (obtained from Eq. 5) are seen to be relatively large in only a few cases, notably, Umiat bentonite, kaolinite, and rust, at $\theta = 1$. At lower temperatures, $\theta \geq 10$, the differences diminish rapidly and eventually become practically insignificant.

A family of water phase composition curves calculated for the range of specific surface areas encompassing all naturally occurring soils is shown in Figure 3. The curves provide a quick estimate of W_u for any soil when its specific surface area is known or can be estimated. The expansion of certain clay minerals when wetted, which exposes an "internal" surface area, must be accounted for in S . Methods are available for measuring the total (internal and external) surface, S (10, 12, 21); when measured values of S are not available, it is possible to derive a "geometric" specific surface area from grain size distribution curves to obtain good estimates of W_u (11). (Specific surface areas derived from nitrogen adsorption isotherms are unsatisfactory because they are a measure of "external" surface only.)

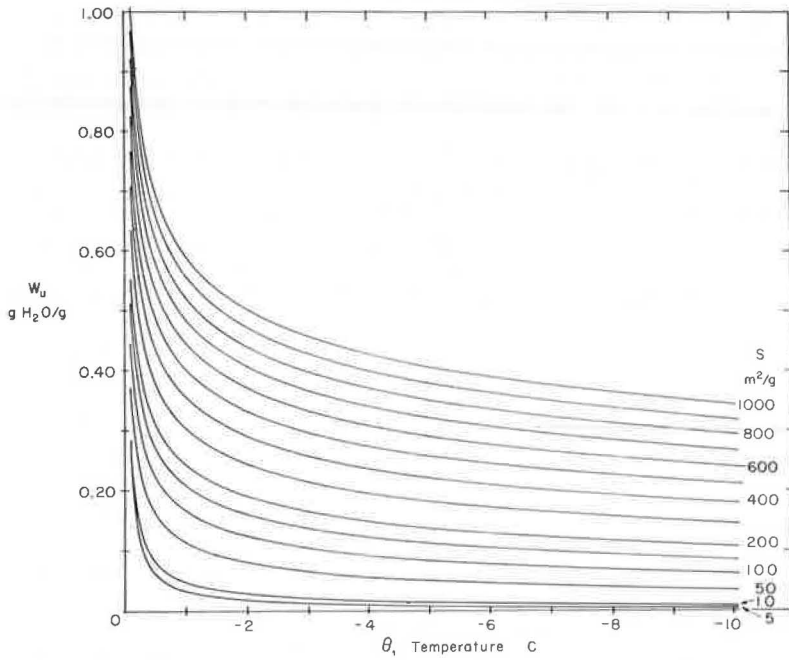
Equation 5 is similar to one proposed by Dillon and Andersland. However, Eq. 5 is considerably simpler and more convenient in that it requires that only one soil parameter, S , be known. In terms of accuracy of predicted values of W_u , Eq. 5 is probably equal to the Dillon and Andersland equation, but this cannot be verified at present because the data required for a complete test are not yet available. Because Eq. 5 rests on correlations obtained from a very extensive body of water phase composition data, we believe that it can be used effectively in frost penetration calculations to predict thaw and degradation of permafrost under natural and disturbed conditions and to calculate the time-dependent temperature distributions associated with foundations, footings, production oil and water wells, and buried or overhead pipelines. In such calculations it is necessary to consider the term

$$\int_{\theta_1}^{\theta_2} L (\partial W_u / \partial \theta) d\theta$$

in a heat capacity Eq. 3. This term, in which L is the latent heat of phase change, accounts for the heat involved as the relative amounts of ice and unfrozen water change in the temperature interval $(\theta_2 - \theta_1)$. Usually, L is taken as a constant equal to 79.7 cal/g, which is satisfactory in all but exceptional cases. However, it is known that the latent heat of phase change is not constant but varies with $W_u(1)$. The manner in which this complication may be dealt with is treated in a subsequent paper.

Table 2. Unfrozen water contents.

Soil	Observed W_u (percent)		Calculated ΔW_u (percent)	
	$\theta - 1$	$\theta - 10$	$\theta - 1$	$\theta - 10$
Basalt	3.45	0.25	0.04	0.19
Rust	11.05	1.75	-6.29	-0.96
West Lebanon gravel, <100 μ	3.82	0.87	2.58	0.47
Limonite	8.82	1.29	-0.97	0.62
Fairbanks silt	4.81	2.27	5.14	0.55
Dow Field silty clay	10.35	2.52	0.91	0.91
Kaolinite	23.80	10.30	-8.81	-4.98
Suffield silty clay	13.92	6.75	5.95	1.28
Hawaiian clay	32.42	18.53	2.15	-1.27
Wyoming bentonite	55.99	28.66	-4.00	0.70
Umat bentonite	67.55	30.66	-15.56	-1.30

Figure 3. Calculated phase composition curves for various values of specific surface area.

ACKNOWLEDGMENTS

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USE OF THE ICE-WATER SURFACE TENSION CONCEPT IN ENGINEERING PRACTICE

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Many authors use surface tension phenomena to explain relationships among soil type, pressures developed during freezing, and freezing temperature. These relationships are ascribed to effects arising from the situation of the ice and water interfaces within soil pores. Two particular applications are (a) prediction of pressures (or "suctions") developed at the frost line and, thus, the susceptibility of a soil to frost heave, and (b) prediction of the proportions of ice and water as a function of soil type and temperature and, thus, of the thermal properties of the soil. Recently developed soil-testing techniques for these purposes are described. Theoretical and experimental studies indicate these tests to be the basis of improved practical procedures, and the manner of their utilization is discussed. The role of surface-tension concepts in providing a relatively simple picture of many aspects of frozen soil behavior can be compared to the importance of capillarity and surface tension in understanding the behavior of unfrozen soils.

• THERE are two aspects of soil freezing that have been the subject of theoretical and experimental studies of practical significance: (a) prediction of the susceptibility of soils to frost heave, and (b) prediction of the phase composition (proportions of ice and water) and hence thermal properties of frozen soils. Both aspects involve consideration of surface tension phenomena; however, the practical applications do not require detailed understanding of the phenomena. The prediction procedures have only recently been developed, however, and before they are likely to be widely accepted, both engineer and scientist must be satisfied as to the validity of their theoretical background. This will be briefly described in order to show how the procedures advocated are arrived at. Also, the extent to which the procedures have been, or could be, successfully employed in field situations is reviewed.

SUSCEPTIBILITY OF SOILS TO FROST HEAVE

There are several procedures for predicting the susceptibility of soils to heave, which have had considerable economic importance (1). Grain size criteria are the most widely used. However, most highway engineers are aware that failures (when the soil does not act as predicted) are not uncommon; there is also a general suspicion that materials are frequently rejected as fill although they would in fact not be susceptible to heaving if utilized under appropriate conditions.

Let us review some fundamental points:

1. Frost heave involves the increase in moisture content at freezing, which results from a movement of water to the freezing layer.
2. The movement of water results (for all practical purposes) from a hydraulic pressure gradient in the water.
3. Such a hydraulic gradient must involve the development of a lower water pressure at the frost line than otherwise exists in the water in the underlying soil.
4. If the occurrence and magnitude of such pressure drops at a frost line are predictable, it is but a short step to predicting whether frost heave will occur.

Laboratory Measurement of Suction Developed at Freezing

Laboratory measurement of the pore water pressure at a penetrating frost line is, in principle, simple. A soil sample is placed on a saturated porous filter that is connected to a pore water pressure measuring device; the sample is frozen, starting at the top. One such apparatus (2) also included provision for raising the confining pressure (σ_3) and for initially applying an equal "back" pressure to the pore water. When the frost line is immediately above the porous filter, the observed water pressure must be exactly the same as it is at the frost line. In practice, the apparatus was somewhat complicated, particularly the arrangements for freezing and for the precise measurement of the water pressures and for avoiding stresses due to lateral containment of the sample. Setting up the apparatus for each test was time-consuming, and repetitive freezing of the same sample was difficult. Recently, by using controlled thermoelectric cooling (3) and a transducer to measure the pore water pressure, a new version has been used that allows a test to be completed in about 10 minutes with a reproducibility in successive freezing of 5 to 10 cm of waterhead (0.005 to 0.01 kg/cm²). This new apparatus (the component parts are now available commercially) enables us to make detailed investigation of factors that may influence the suction developed.

The investigations have shown that the suction (pore pressure drop) for a given soil is always relative to the confining pressure. If a soil under a confining pressure of 1 kg/cm² (one atmosphere) shows a suction at the frost line of -0.5 kg/cm², at a confining pressure (and initial pore pressure) of 5 kg/cm² the pore water pressure (P_w) at the frost line will be 4.5 kg/cm². The quantity generally known as the heaving pressure is a confining pressure, and it is also equal to the pressure in the ice in lenses (P_i). In talking therefore of the suction developed at the frost line, we should take care to specify that the relevant quantity is

$$P_i - P_w$$

If we establish the value $P_i - P_w$ for a soil sample, we can immediately define the pore water pressure, P_w , that can occur at the frost line at a specified depth in the ground by giving P_i the value of the overburden pressure at that depth. Frost heaving (i.e., ice accumulation) will occur if this value of P_w is lower than the preexisting pore water pressures (or suctions, if above the water table). In practice, frost susceptibility is best defined in terms of the thickness X_n of the layer (if any) in which ice accumulation can occur. Because overburden pressure ultimately prevents ice accumulation, there is a maximum depth for every soil below which accumulation will not occur.

The criterion for ice accumulation is

$$P_i - P_w > \gamma_s X - (X - Z) \quad (1)$$

where

$$\begin{aligned} \gamma_s X &= \text{overburden pressure (grams/cm}^{-2}\text{), and} \\ Z &= \text{depth to water table from ground surface (cm).} \end{aligned}$$

At a depth X_n (cm),

$$P_i - P_w = \gamma_s X_n - (X_n - Z) \quad (2)$$

and there will be no ice accumulation at depths greater than this. From Eq. 2

$$X_n = [(P_i - P_w) - Z]/(\gamma_s - 1) \quad (3)$$

If X_n has a negative value, there will be no frost heave. Otherwise, heave will occur if the material in question lies within the depth X_n from the surface. Figure 1 shows the relation of X_n to $P_i - P_w$, γ_s , and Z .

Alternative Test Procedure

Although its use represents the most direct and precise method of investigating the pore pressure effects, the apparatus described is not entirely suitable for practical purposes because of its relatively high expense. However, an examination of theoretical aspects suggests an alternative test procedure based on analogy. According to several studies (4, 5, 6, 7), the pore pressure decrease, or more precisely $P_1 - P_w$, follows from the limitation on the size of ice crystals imposed by the soil pores. It has been well known to physical chemists for many years that, for small crystals, there is a difference in pressure between the solid and surrounding melt, which is a function of the crystal size. With some simplification, this can be expressed for ice and water as

$$P_1 - P_w = 2\sigma_{iw}/r \quad (4)$$

where

r = effective radius of crystal, \approx radius of pore, and
 σ_{iw} = surface tension for ice to water (8), ≈ 30 erg/cm⁻².

This equation is analogous to the well-known capillary equation governing the rise of water in capillary tubes:

$$P_a - P_w = 2\sigma_{aw}/r \quad (5)$$

where

P_a = pressure of air,
 P_w = pressure of water, and
 σ_{aw} = surface tension for water to air, ≈ 72 erg/cm⁻².

It is well known that this equation, at least for relatively coarse-pored soils, describes the pressure state of water in wet but unsaturated soils with the air-water interfaces in pores of size r . If a saturated soil is placed on a filter attached to a drainage tube containing water at atmospheric pressure and air pressure is applied progressively to the soil sample, the water is eventually displaced from the soil pores. If appropriate attention is given to preventing the leakage of air directly to the drainage system and to the rate of application of the air pressure, the pressure at which air first spreads through the soil is revealed by observation of the drainage (Fig. 2). A series of experiments (9), using translucent soils and microphotography, showed that at this given air pressure a front of air passed through the sample (Fig. 3), which could be likened to penetration of a frost line. Because the pore water pressure is known to be atmospheric (i.e., zero), the air pressure applied is also equal to $P_a - P_w$. It appears reasonable to assume that the quantity r is the same in Eqs. 4 and 5. Accordingly, if the measured value of $P_a - P_w$, which is known as the air intrusion value, is multiplied by the ratio $\sigma_{iw}/\sigma_{aw} = 0.42$, we derive the value of $P_1 - P_w$ for that soil. A series of experiments (2) established the correlation between the two quantities (Fig. 4, 2). A form of the apparatus for measuring air intrusion value has been developed (by Geonor in Norway) commercially (Fig. 5).

Field Testing and Application in Engineering Practice

The foregoing investigations suggest a potentially precise method of assessment of frost susceptibility based on determination of the value $P_1 - P_w$. Whether the determination is made directly by using a freezing test or indirectly by using an air intrusion test, the direct relevance of this quantity, the explicit consideration of overburden pressures and groundwater conditions it permits, and its relatively substantial theoretical background are in contrast to the arbitrary nature of conventional tests and criteria.

Tests have been carried out in cooperation with the Ontario Department of Transportation and Communications, which involve the sampling of highway subgrade material. For old roads with widespread frost damage, the calculation of $P_1 - P_w$ following air

Figure 1. Thickness of soil layer affected by ice accumulation as a function of depth to water table and bulk density.

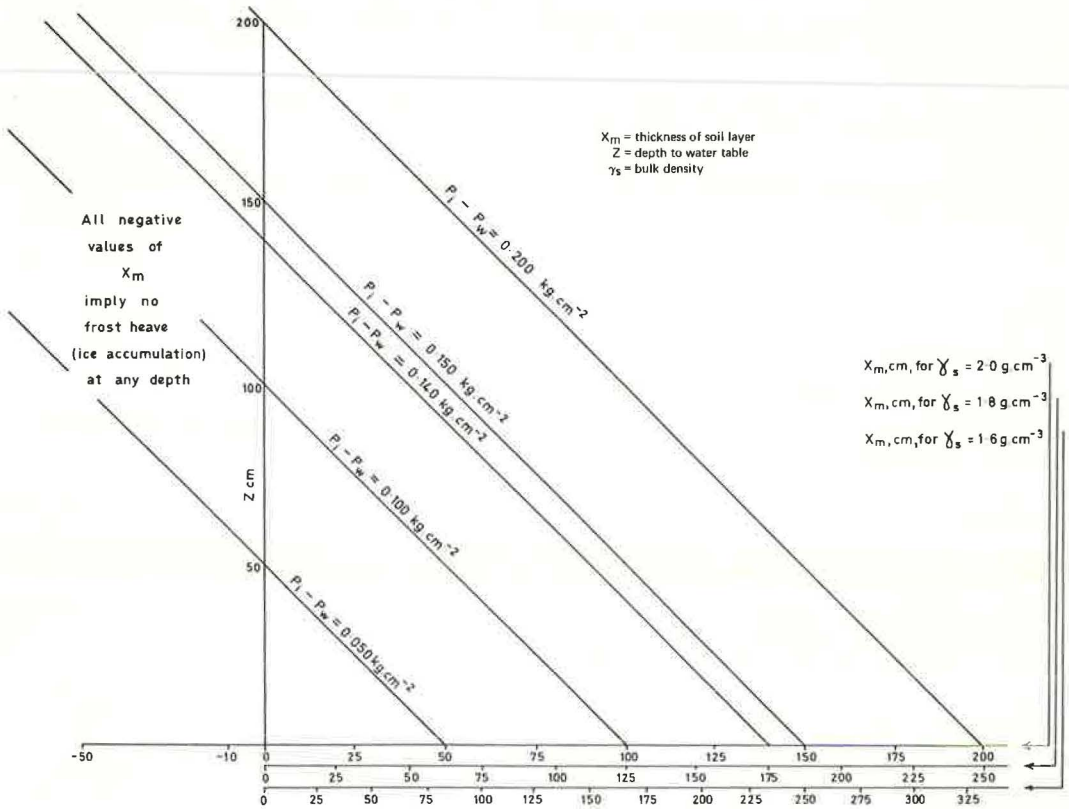


Figure 2. Observations from air intrusion test on highway subgrade material.

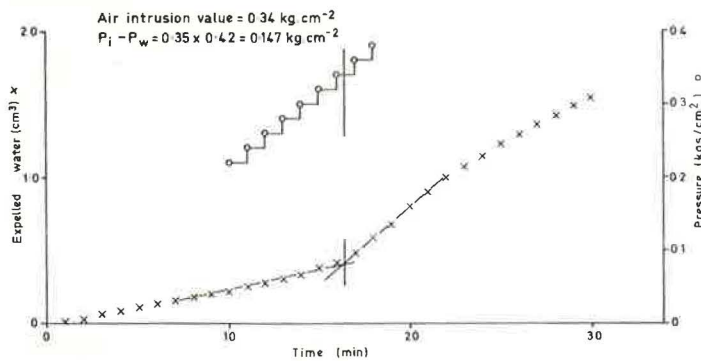


Figure 3. Translucent soil showing advance of air front through pores (light regions) during air intrusion.

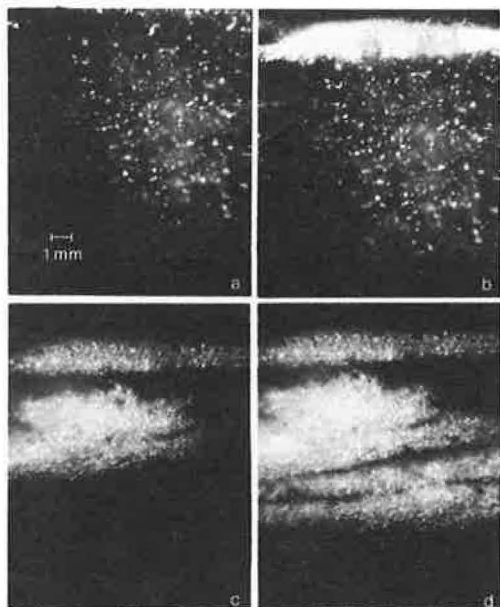


Figure 4. Relation between air intrusion value and pore pressure at a penetrating frost line.

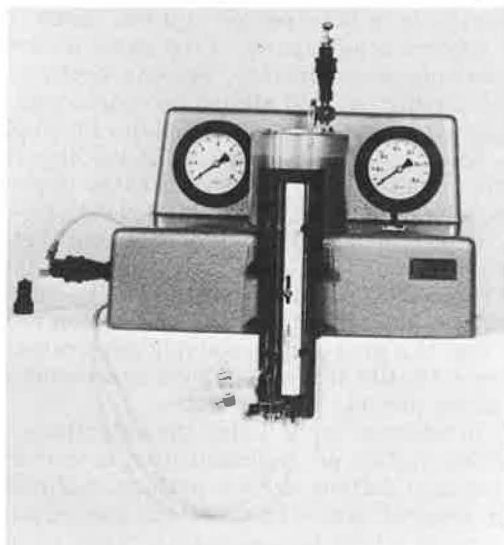
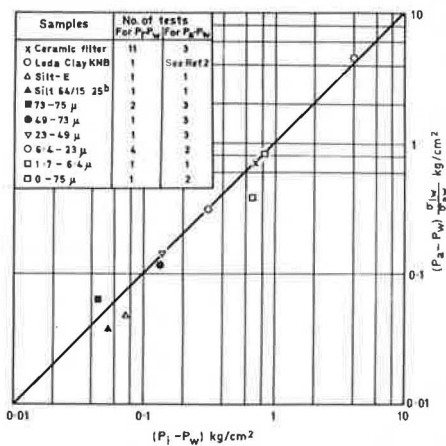


Figure 5. Air intrusion apparatus.



intrusion tests merely confirmed the frost-susceptible nature of the material. A series of tests made on materials that were classified as satisfactory by conventional methods (from a new highway built to the department's current standards but which nevertheless apparently experienced some heaving) is summarized in Table 1. The tests show that heaving is to be expected in three cases (in one case only in very near surface layers). In several other cases, if the depth to water table Z (actually assessed in spring) had been somewhat smaller, heaving would have also been expected. Perhaps, "depth to water table" should always be considered as being closer to the surface than it actually is to allow for wet conditions with incomplete drainage during a freezing period.

Finally, it is of interest that the department's higher classification of ASC (Acceptable Sand Cushion) has been applied to precisely those soils that the present tests also show as having the best (i.e., lowest) $P_1 - P_w$ values.

It is recognized by the department that their procedures do not rule out occasional heaving. Such occurrences are often related to particularly wet conditions, and the experienced engineer is usually able to take special measures to prevent, or at least to rectify, the situation. The conclusion to be drawn from the tests summarized, however, is that the procedure involving determination of $P_1 - P_w$ leads to predictions of frost susceptibility that are in good agreement with previous experience and that often allow a more precise assessment.

In addition to the latter investigations, air intrusion tests have been applied in the interpretation of ice distribution in various permafrost situations (10, 11, 12). These field applications are not perhaps sufficient to demonstrate the value of the procedures for general use. The questions that remain to be answered are related to technical innovation rather than scientific research and require fairly large-scale tests in association with highway construction. For example, in many cases in engineering practice a limited amount of heave may be tolerated, and the problem of limits for acceptability arises (the procedures only give a yes or no answer). Natural soil materials vary substantially over small distances, and decisions as to frequency of sampling are required. Because the groundwater conditions are explicitly considered in the analysis (common experience has also shown they must be), methods for assessing these are necessary. Groundwater conditions during the winter are a subject that requires much investigation.

In many highway projects, the question arises as to the frost susceptibility of a relatively fine uniform silty-sandy material that may be available in substantial quantity. To regard such soils as unacceptable, if in fact they could be utilized as construction materials under appropriate overburden and groundwater conditions, is a mistake of considerable economic importance. To utilize them and experience subsequent failure are equally serious. It is precisely with marginal soils of this type that the air intrusion test is most easily performed. The time involved per sample compares favorably with that for grain size analysis. The determination of $P_1 - P_w$ for "dirty" gravels and other mixed soils can be better determined by using the freezing tests. Perhaps the air intrusion procedure (which is inexpensive and "portable") should be regarded as an on-site test, with occasional samples being tested using the freezing procedure at a central laboratory. Experience to date indicates that, for simplicity and speed of operation and for reproducibility of results, the freezing test is to be favored over the more indirect air intrusion approach.

The answer to these questions will only be obtained from fairly comprehensive trials. At the very least, such trials would give further insight into the problem of frost susceptibility.

PHASE COMPOSITION AND THERMAL PROPERTIES

It has been firmly established in the last 20 years that frozen soils contain both ice and unfrozen water (13, 14, 15). The amount of unfrozen water is a function of temperature and of soil material, and in fine-grained soils it is present in very significant quantities at those temperatures that are of the greatest practical interest (Fig. 6, 15).

Our knowledge of the interrelationship of soil composition, temperature, pressure, and proportions of ice and water present is currently of the greatest importance in making possible the prediction of thermal properties of freezing soils. The latent heat involved in changes in the proportions of ice and water present in the soil in most cases

totally dominates the values now known as the apparent specific heat (15, 16) and the apparent heat capacity of freezing soils (Fig. 7, 16). It is relatively simple to calculate, with a reasonable degree of accuracy, heat capacity and thermal conductivity once the proportions of ice and water, as a function of temperature, are known.

These values may be predicted by using relatively simple and standard laboratory tests, which are carried out at room temperature, that determine the suction-moisture content relationship of the soil. As water is removed from a (unfrozen) soil, the remaining water is under a progressively greater negative pressure or suction (17). This phenomenon, well known to agronomists for many years, is in large measure ascribable (at least for fairly wet soils) to the capillary relationship given above: $P_a - P_w = 2\sigma_{aw}/r$. With the reduction of water content, the interfaces between air and water (menisci) lie in smaller and smaller pores, and accordingly the water shows a greater suction (i.e., $P_a - P_w$). The relationship between suction and moisture content is determined by using "suction plates" or "pressure membrane" apparatus (18). It is important to note that we are not concerned in this case with a specific critical value of $P_a - P_w$ (at which air first enters the soil) but instead with a series of values (Fig. 8, 15) greater than the air intrusion value.

Analogous to this drying process is the freezing of the soil after the initial formation of ice; freezing continues progressively as the temperature falls below 0 C with extension of the already present ice in the soil into smaller and smaller pores. A progressively lower temperature is required for freezing to continue. The temperature (11) is given by

$$\Delta T = 2\sigma_{1w} TV/rL \quad (6)$$

where

- ΔT = temperature below 0 C,
- T = temperature, deg K,
- V = specific volume of water, and
- L = latent heat of fusion.

By combining Eqs. 4, 5, and 6, it follows that the water content found with the pressure membrane apparatus at a suction of $P_a - P_w$ is the water content that will occur in the frozen soil at temperature ΔT as given by

$$\Delta T = [(P_a - P_w) (\sigma_{1w}/\sigma_{aw}) TV]/L \quad (7)$$

[The term σ_{1w}/σ_{aw} is modified in certain cases (11, 19).]

The relation has been verified (18) with a high degree of precision over a narrow temperature range (0 to -0.2 C) and with somewhat lower precision to approximately -1.0 C (15) (Fig. 9). Determining the proportions of ice and water by calorimetric or other means is often cumbersome, as also is the direct determination of the thermal properties as a function of temperature. The suction moisture content test that is carried out widely in many agricultural laboratories and that can with equal ease be utilized in geotechnical laboratories appears to be a convenient means of arriving at thermal properties.

Application in Engineering Practice

Sophisticated methods of highway design, especially those involving thermal insulation, and other procedures designed to limit the penetration of frost require precise values for the thermal properties of heat capacity, thermal conductivity, and related functions. General engineering practice where these quantities are required, however, is still to use a somewhat arbitrary assessment based on a comparison of the material in question with materials that have been the subject of experimental measurements of thermal properties. The experimental studies of unfrozen water contents merely point out the inaccuracy of such a procedure, and in time the procedures now advocated may become more widely used.

Figure 6. Amounts of water remaining unfrozen at various negative temperatures.

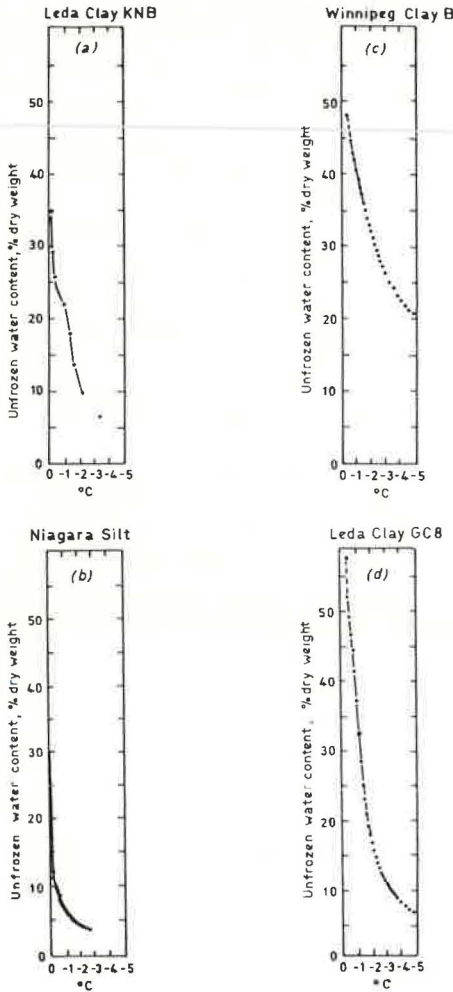


Figure 7. Apparent volumetric heat capacity of frozen soils.

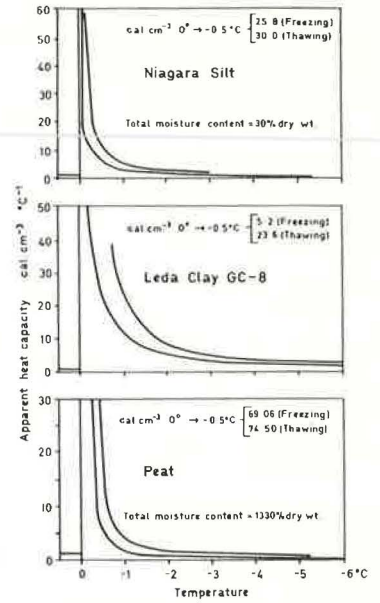


Table 1. Summary of frost damage test results.

Highway	Sample Number	Station	Pavement Condition	Observed Depth to Water Table, Z (cm)	Classification ^a	P ₁ - P _n (grams/cm ⁻²)	Depth to Which Heaving Could Occur (X _a , cm)
Location 1a							
60	LE-78	300 + 00LT	Center-line cracking	86	A	220	134 ^b
	LE-79	302 + 73RT	Uneven pavement	56	ASC	25	0(-31)
	LE-80	305 + 44RT	Center-line cracking	69	ASC	29	0(-40)
	LE-81	305 + 44RT	Center-line cracking	69	A	63	0(-6)
	LE-82	282 + 70LT	Transverse cracking	71	A	55	0(-16)
	LE-83	281 + 05LT	Center-line cracking	76	B	42	0(-34)
	LE-84	278 + 55RT	Center-line cracking	69	A	84	15
	LE-85	283 + 00RT	Uneven pavement?	76	ASC	16	0(-60)
Location 1b							
	LE-75	404 + 40LT	Uneven pavement?	102	NASC	38	0(-64)
	LE-76	402 + 50LT	Uneven pavement?	96	A	80	0(-16)
	LE-77	404 + 32RT	Uneven pavement?	66	A	147	81 ^b
Location 1c							
	LE-74	421 + 17LT	Uneven pavement?	104	ASC	21	0(-83)

Note: γ_s assumed to be 2 grams/cm⁻³.

^aThe department classifies soils for suitability in highway construction on a descending scale: ASC, NASC, and A to E. The classification is not exclusively concerned with frost susceptibility but with overall suitability under normal conditions.

^bSample actually taken within this depth.

Figure 8. Suction-moisture content relations of four soils (determined at room temperature).

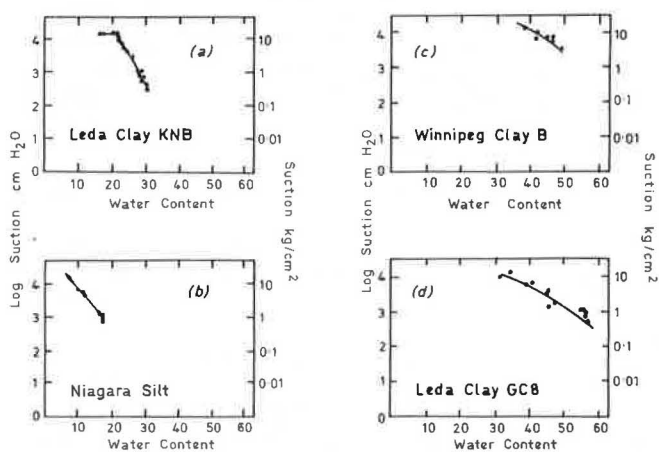
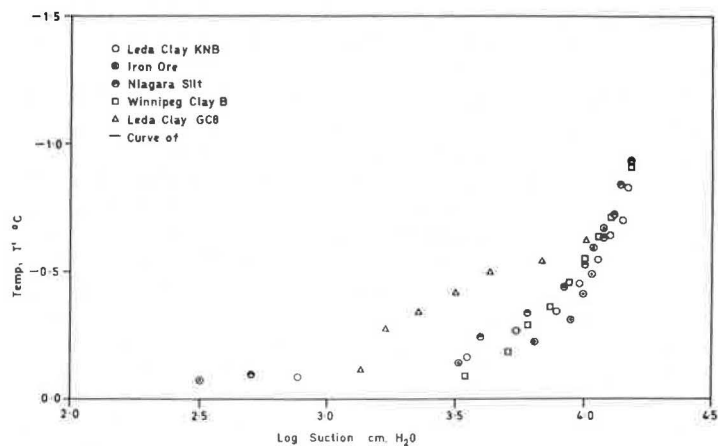


Figure 9. Relation between suction measured in suction-moisture content tests and temperature at which an equal quantity of unfrozen water occurs in the frozen soil.



The use of pressure membrane and suction plate measurements appears justified for predicting thermal properties to -1.5 or -2 C. This covers the range of temperature where unfrozen water content and thermal properties vary most critically. Although capillary theory is scarcely applicable to lower temperatures (there being insufficient water to provide menisci), well-established correlations with other tests involving soil-water free energy (suction) relations (18) give additional significance to these procedures.

BRIDGING THE GAP: THEORY TO PRACTICE

The increased understanding of soil properties deriving from surface tension concepts is also important because of the role such understanding has in the more or less intuitive approach used in much engineering design. It provides a conceptual framework for considering the effects that applied stresses have on the frozen soil system and for understanding the deformation properties of frozen ground as they are known from both laboratory and field experience.

The role of exposure to freezing and thawing in changing the properties of clays is now understood—the suctions developed during freezing are sufficiently great to produce a substantial preconsolidation effect, evident everywhere in the ground surface layers exposed to annual freezing, while the formation of ice lenses leads to numerous discontinuities and a friable soil structure (11). The effects of temperature change, within the frozen state, in producing moisture migrations and other long-term changes in the distribution of ice and water become clearer. The latter are also involved in the phenomenon of long-term creep, and the deformations to be expected in association with load-bearing foundations exposed to temperature change in cold regions over long periods of time.

Very significant contributions have been made, especially in the study of phase composition of frozen soils, by authors who do not support surface tension concepts. In a notable example, Low et al. (20) use relative humidity and thermodynamic functions as the basis for analysis; such an approach, however, demands a specialized understanding that is not required for the more easily grasped physical concepts. The various approaches of different authors are often not contradictory but complementary and reflect the different aims and backgrounds of the agronomist, engineer, and research scientist.

There is such a wide measure of agreement over many of the experimental observations (carried out by different scientists) that, no matter what conceptual pictures we may use to explain the observations, we must conclude that we are dealing with a substantial body of knowledge. The situation is similar to that of the role of capillarity in unfrozen soils; whether we refer to "apparent cohesion," negative pore pressures and effective stress, or relative free energy, there is no doubt that the engineering procedures involved in working with unsaturated soils have been greatly aided by the application of quite elementary concepts of capillarity and surface tension. This progress has occurred in spite of the fact that some soil physicists and other scientists may still be discussing at length the exact nature of and behavior associated with surface tension forces. The potential benefits in the field of frozen-ground engineering from the studies described in this paper may be similar.

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CHEMICAL ADDITIVES TO REDUCE FROST HEAVE AND WATER ACCUMULATION IN SOILS

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Soil waterproofing chemicals are known to reduce frost heave in soils, particularly when soils are dried; the present study demonstrates that these chemicals work in moist soils as well. The mechanism and limitations are partly defined in this paper. A model soil waterproofing compound, 4-tert-butylcatechol (TBC), was studied as representative of the general group. Water absorption to growing ice lenses was measured as an indication of frost-heave rates. Capillary conductivity was also measured and correlated to heave and treatability. As little as 0.01 percent TBC produced minimum ice lens growth and capillary flow in some soils. A-4 soils responded well to treatment, whereas soils containing more than 50 percent clay did not. A simple hydraulic conductivity test should be considered as a major soil frost-susceptibility criterion.

• FROST HEAVE in soils has been reduced through the use of chemicals from the broad classes of cementing agents, plugging agents, dispersants, waterproofing chemicals, and freezing-point depressants. These types of chemicals treat one or more of three conditions (freezing temperatures, a frost-sensitive soil, and a water source) necessary to produce frost heave. Cementing agents decrease frost sensitivity by bridging soil grains and by plugging pores. Freezing-point depressants, primarily sodium and calcium chloride, decrease the freezing temperature of matrix water. Waterproofing chemicals limit water absorption by soils and consequently interfere with the water supply. Dispersants destroy soil structure and allow clay grains to plug capillary pores.

Lambe (25) tested about 40 additives and estimated their potential as frost-heave modifiers. He rated dispersants and inorganic cationic aggregates as most promising because they are easy to apply, low-cost, and effective at moderately low concentrations. Lambe rated waterproofing agents somewhat less promising even though they were excellent for modifying frost heave. His judgment was based on the need for extensive drying once soils have been treated and because more than 0.5 percent of these additives was needed. These disadvantages no longer exist because more recently discovered waterproofing chemicals will limit water absorption in moist soils when added at a rate of 0.1 percent or less (4, 8-20, 22).

Specific chemical groups containing various hydrophobic substituents limit liquid water absorption by soil. These have been identified by extensive laboratory investigation, as reported elsewhere (4, 8-20). Among the many compounds tested, 4-tert-butylcatechol (TBC) is one of the most effective for silty and silty clay soils. TBC was being commercially developed for several years under the trademark Terbec soil stabilizer and during that time was placed in more than 25 test roads to treat highly frost-sensitive soils.

Some roads were constructed entirely from the native soil with a seal-coat overlay, whereas others were constructed with a gravel base and asphalt wearing surface overlying a water-repellent soil layer. When they were constructed with adequate thicknesses of gravel base, roads endured spring thaws with little or no damage. Exceptions normally could be traced to poor drainage during critical freezing periods. These field data prompted investigations of ice accumulation in TBC-treated soils to help define limiting conditions for adequate performance.

TBC is known to effectively limit liquid water adsorption by many soils, but it does not interfere with vapor phase transmission (21). In dry soils, this phenomenon is easily understood because chemicals like TBC change soil mineral grain surfaces from water wettable to water repellent. TBC changes the wettability of soils because the ortho-hydroxyl groups on the phenyl ring coordinate with aluminum ions on mineral grain surfaces in such a way that the hydrophobic tert-butyl group extends from clay surfaces (7). The apparent contact angle between such a treated mineral grain and water is sufficiently high such that positive pressures must be applied before water penetrates dry water-repellent soil matrices (1, 5, 6).

Moist soils also are changed significantly by TBC, but the mechanism is more difficult to isolate than for dry soils. One explanation could be that water films in capillaries are disrupted by hydrophobic chemicals, thus reducing the number of pores available for conducting water (21). Another explanation could be that absorbed hydrophobes introduce disorder into water layers around clay particles and into water in soil pores. The net effect is decreased affinity for water, which can be expressed as a decrease in osmotic repulsion among soil grains (2, 21). In the latter case, water migration would be slowed by thin moisture films rather than by disrupted capillaries. The first explanation seems to be an extreme case of the second because both require extensive alteration of water structure and of forces between water and soil grains.

METHODS

Numerous freezing tests can be used to characterize the frost sensitivity of soils and the influence of chemical additives. Most tests seek to rank soils in a relative order of frost susceptibility or to rank chemical treatments in a relative order of effectiveness. The needs in the present study were somewhat different, however, because limiting conditions for adequate chemical performance in the field needed to be defined. We needed to quantify frost heave as influenced by drainage, temperature gradient, chemical concentration, and soil type. We chose to follow ice lens growth at stationary freezing fronts rather than program temperatures for advancing fronts, and we monitored water absorption under fixed overburden pressures rather than using heaving rate or pressure.

Many separate runs were required to study the significant variables; each run required about 3 weeks. Consequently, an automated test was devised to minimize long, tedious manual labor and to greatly simplify data handling. Ice lens formation was followed by continuously monitoring water absorption and temperature profiles using an automated data system with punched-tape output.

A chamber was built to hold eight soil cores; each core was contained in a separate cell (Fig. 1). The selected arrangement corresponds roughly to an open soil system as described by Jumikis (23). Ceramic frits, with bubble pressures below a moisture tension of 150 cm, were mounted in an aluminum plate and were connected to water reservoirs to control moisture tension. Filter paper underneath the soil cores assisted water conduction across the core-frit discontinuity. Frit permeabilities were sufficiently high to supply water at unrestricted rates. Cores were jacketed by rubber membranes, surrounded by polystyrene foam and Styrofoam brand plastic foam, and capped with Saran Wrap brand plastic film to prevent moisture loss.

Cores were frozen from the top by coolant that was circulated through surcharge weights equivalent to 1 psi or approximately 1 ft depth of gravel base. As soon as the cores reached freezing temperatures, ice was nucleated with liquid nitrogen. Bottom temperatures were held above freezing by coolant that was circulated through a large aluminum plate. Zero isotherms normally were adjusted to the center of the cores, forcing ice lenses to grow 2 in. from the frits. Typically, top temperatures were adjusted to -3 C and bottom temperatures to +3 C, producing a temperature gradient of 0.6 C/cm between the bottom plate and the top surcharge weight. However, typical temperature profiles measured by thermocouples within the cores registered 0.4 to 0.5 C/cm. Temperatures were controlled within about ± 0.15 C, corresponding roughly to the accuracy limits of iron-constantan thermocouples and the Pace zero-degree reference unit used in the temperature measuring system.

Water absorption was measured by using specially constructed syringe pumps capable of detecting a 0.01-cc flow (these have since been replaced by Statham Universal load cells). Voltage proportional to volume of flow was logged for 8 m using a data acquisition system. Thermocouples and flow meters were sampled hourly by the unit, although sampling intervals could be selected between 5 min and 8 hr. When a run was completed, an IBM 1130 computing system read the data from punched tapes, processed the data, and plotted temperature profiles and cumulative water absorption for each soil core. With this system, changes in temperature profiles can easily be compared and correlated with changes in water-absorption rate. Furthermore, the amount of water absorbed during capillary equilibration prior to freezing is known, and the absorption rate during early stages of ice lens growth can be determined from the cumulative plot. At the end of a run, moisture contents were determined for core segments. Average ending moisture contents were calculated for each core and could be compared to moisture absorption plotted by the computing system.

In the ice lens test, cores 5 by 10 cm (2 by 4 in.) were compacted at optimum using double-end loading and 500-psi pressure. TBC was added to these soils in the mixing water. About 4,000 ppm of this chemical are soluble in water at room temperature, but concentrations of 7,000 to 8,000 ppm are needed to treat most soils at 0.1 percent additive. Consequently, such emulsions must be shaken vigorously before being atomized onto soil. All cores used in the ice lens test were wrapped in plastic film and stored moist until used. Only a few cores were partially dried to observe the influence of drying on frost susceptibility of TBC-treated soil. Cores completely air-dried were never placed in the test because they re-absorb only small quantities of water and do not grow ice lenses. Furthermore, air-drying is never achieved in the field for sub-base soil layers in humid climates.

Other tests were run in this study to aid the interpretation of the ice lens test data. Capillary flow and conductivity were determined by using apparatus similar to that used by Sharma and Uehara (29), which was initially devised by Nielsen and Bigger (27). Porous ceramic frits were placed at both ends of cores 2 in. in diameter and 1 in. high with a 1-psi surcharge weight on top. Water reservoirs were connected to the frits and adjusted to moisture tensions of 25 and 75 cm. The tension gradient forces capillary water to flow through the soil cores. Water loss from the reservoir set at a tension of 25 cm was followed by the use of Statham Universal load cells. Eight channels of capillary flow were logged on punched tape simultaneously with the ice lens test data and later processed by computer. A typical run would take 3 to 4 days.

Unconfined compressive strength is also given for selected soils as related to TBC concentration and curing conditions. Most soil cores were immersed at optimum moisture after a moist cure, but some were immersed after air-drying. Strengths were determined on cores 1.13 in. in diameter and 2.25 in. high, which had been compacted at optimum moisture and 500 psi.

Soils used in this paper are described in Table 1 and include five from Michigan, two from Iowa, and four from West Virginia. Sample numbers identify sample sources, while letter designations (e.g., 11-B and 11-C) indicate different samples from nearly the same source. Five soils are classed A-4, two A-6, three A-7-6, and one A-2-4. Several additional soils were run through the tests described in this paper, but they only tend to confirm trends established by the selected data. Several soil-type names are omitted because the sample was not taken within the normal soil profile but rather from the underlying parent material.

RESULTS

Ending Moisture Contents

Typical moisture profiles for TBC-treated and untreated samples are shown in Figure 2 for Iowa silt cores (soil 23-B). The two top bar graphs are based on cores exposed to a moisture tension of 50 cm and 0.5 C/cm temperature gradient for 25 days. Starting water contents are indicated by solid lines across bars, and average ending water contents are indicated by arrows above each moisture profile. Shaded bars designate increases in moisture content for core segments.

Ice lenses grew at similar depths for treated and untreated cores in the top pair. The unfrozen lower half of the untreated core contained about 20 percent water, essentially the equilibrium moisture content present at a tension of 50 cm. In contrast, the moisture in TBC-treated soil was depleted in the unfrozen soil, whereas it accumulated in the frozen portion. The average ending moisture contents for the untreated core was 11 percent higher than at the start, whereas it was only 1 percent higher for the TBC-treated core. Much more water accumulated in both treated and untreated cores when they contacted free water (Fig. 2). Average ending moisture contents were about 32 percent and 23 percent for untreated and TBC-treated samples respectively. Ice lenses were very prominent in both. Under these saturated conditions, water moved more easily through unfrozen TBC-treated layers to freezing fronts. Also, the fact that the ice lens was growing less than 1 in. above free water probably enhanced ice lens development in the treated sample because there was very little unfrozen water repellent soil to limit water migration.

Average ending moisture contents after ice lens growth are plotted in Figure 3 for many cores of Iowa silt, soil 23-B, exposed to moisture tensions ranging from 0 to 150 cm. For untreated and treated cores alike, more water accumulated while the cores contacted free water than at tensions of 10 cm and more. However, there appeared to be very little difference in total water accumulation between tensions of 10 and 150 cm. TBC-treated samples consistently contained 7 percent less water than untreated samples at all tensions.

Soil moisture data from a field test road in Iowa compare roughly to the data shown in Figure 3. Many soil cores were taken immediately after spring thaw from a 3-year-old road. Moisture content of TBC-treated soil layers ranged between 14 and 19 percent, whereas comparable untreated layers ranged between 20 and 25 percent, a 6 percent average difference. Other field installations have been sampled after several winters of freeze-thaw, and moisture contents always have been slightly above optimum moisture for TBC-treated soils.

Most experiments began with cores at optimum moisture. Drier water-repellent soils should be expected to limit ice lens formation even more than moist soils. Consequently, Iowa silt cores (23-D) were dried back slowly in a constant humidity cabinet before being placed in the freezing chamber. Untreated cores began at 8.8 and 9.5 percent water before being exposed to tensions of 0 and 50 cm respectively. After exposure to a temperature gradient of 0.6 C/cm for 21 days, they contained 30.8 and 24.4 percent water respectively, but about 26 and 21 percent of that was absorbed by capillarity before freezing began, as indicated by the bottom moisture content bars for unfrozen soil shown in Figure 4.

Both TBC-treated cores began the experiment at 6.2 percent water (bottom, Fig. 4). Average ending water contents were 9.8 percent and 8.7 percent respectively for tensions of 0 and 50 cm, thereby increasing 3.6 percent and 2.5 percent. Ice lenses were barely distinguishable. Surprisingly, the moisture increase at a tension of 50 cm for these partially dried cores was nearly the same as for cores beginning at optimum in Figure 3. So, water moved to freezing fronts in the drier cores as easily as in more moist cores, but final water contents were substantially lower.

Water Absorption Plots

Moisture profiles and average ending moisture contents developed during ice lens growth were used as criteria for TBC performance in the preceding section. These represent ending values after several weeks of freezing and provide no information about what is happening during the run. This deficiency was overcome by continuously monitoring water absorption for each soil core using an off-line data acquisition system. Water absorption plots, similar to those shown in Figure 5, were machine generated for each run.

In the ice lens test, soil cores were first equilibrated with capillary moisture for about 1 week, while the bottom temperature was held near +3 C. Freezing was then initiated by cooling cores from the top and nucleating ice with liquid nitrogen. Ice nucleation was followed by abrupt exotherms 1 to 2 C high that lasted about 20 hours, during which moisture absorption increased markedly. The maximum flow rates

Table 1. Soil description and analysis.

State	Identifying Number	USDA Type or Texture	Mechanical Analyses						AASHO Classification
			Sand	AASHO Silt	Clay	LL	PL	PI	
Michigan	11-B	Loam	49	31	20	31	22	9	A-4
	11-C	Clay loam	25	24	51	37	19	18	A-6
	13-C	Selkirk silty clay	9	34	57	57	24	33	A-7-6
	14-D	Selkirk silt loam	6	65	29	24	21	3	A-4
	14-E	Selkirk silt loam	5	55	40	28	18	10	A-4
Iowa	23-B	Silt loam	30	53	17	29	25	4	A-4
	23-D	Silt loam	8	81	11	—	—	—	A-4
West Virginia ^a	2	Upshur silty clay	2	27	71	62	22	40	A-7-6
	3	DeKalb sandy loam	44	25	21	17	17	N. P.	A-2-4
	5	Gilpin silt loam	14	32	54	47	27	20	A-7-6
	10	Berks silt loam	25	39	36	31	19	12	A-6

^aSupplied by Lyle K. Moulton and Ray Martin, Civil Engineering Department, West Virginia University.

Figure 1. Cell for growing ice lenses in soil cores.

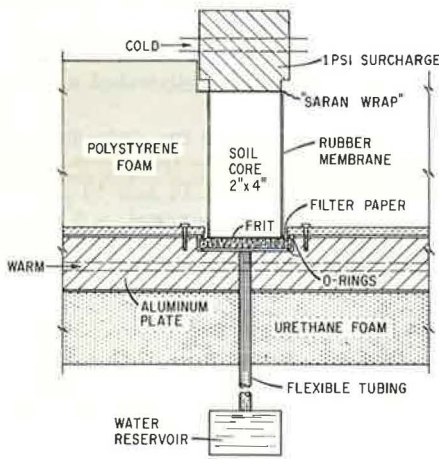


Figure 2. Moisture profiles for Iowa silt (23-B).

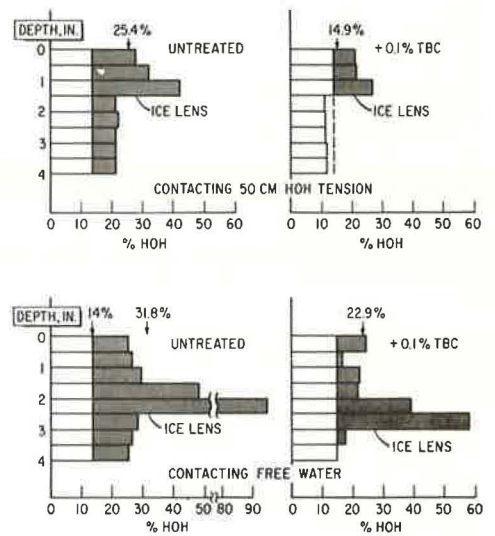


Figure 3. Average ending moisture contents for Iowa silt (23-B).

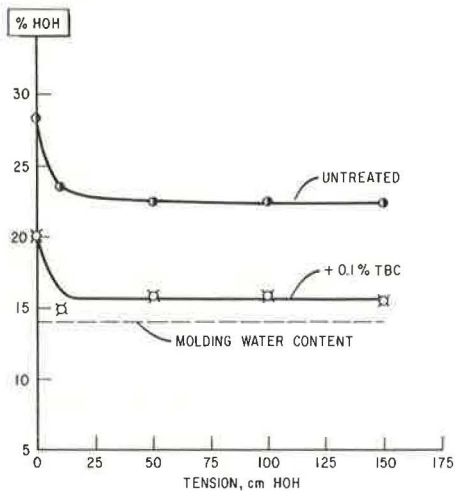
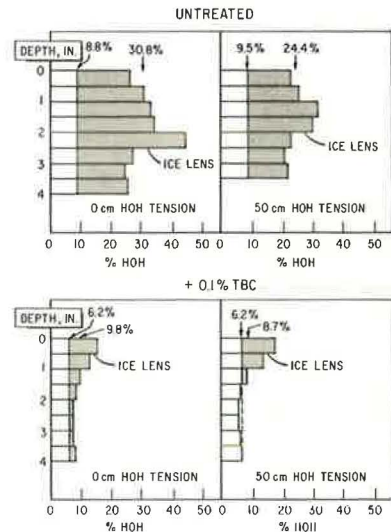


Figure 4. Moisture profiles for Iowa silt (23-D).



lasted from about 20 to 50 hours and then decreased with time, occasionally going to zero. The water absorption rates diminished with time because moisture was becoming depleted below the ice lens and because the ice lens was reaching equilibrium as described by Penner (28). In Figure 5, cumulative water flow at a tension of 100 cm is plotted for soil cores treated with TBC levels ranging from 0 to 0.1 percent. Cumulative total flow can be converted to percentage of water content increase as indicated by the right-hand axis in Figure 5. Roughly 400 grams of dry soil were contained in each core; therefore 4 cc of water absorption represent approximately 1 percent increase in moisture content.

Ice Lens Test Compared to Capillary Flow Test

Maximum water absorption rates during initial stages of ice lens growth were calculated from cumulative water plots similar to those shown in Figure 5. The values for soil 14-D are shown in Figure 6A. Water absorption ranged from 0.38 cc/hr in untreated samples to 0.12 cc/hr, one-third less, for samples containing 0.1 percent TBC. A core containing 0.01 percent TBC flowed at 0.18 cc/hr, roughly half the untreated rate. Average ending moisture contents for the same set of cores are shown in Figure 6B, showing essentially the same trend as the maximum flow rates.

The capillary flow test was run on soil 14-E because soil 14-D had been depleted. Flow rates through these 1-in. thick cores were determined with a 50-cc head differential; the higher reservoir was set at a tension of 25 cm and the lower at a tension of 75 cm. Flow rates decreased from 0.48 cc/hr for untreated samples to 0.04 cc/hr, a tenfold decrease, for all TBC concentrations between 0.01 and 0.1 percent.

The plots shown in Figure 6 strongly indicate that capillary water flow in this particular soil can be markedly decreased by using TBC concentrations as low as 0.01 percent, whereas field installations always have been constructed using about 0.1 percent TBC. It is interesting to note in Figure 6 that flow rates for untreated soil are nearly the same in both the ice lens and capillary flow tests, but that TBC decreased capillary flow much more than flow to growing ice lenses. The flows to ice lenses were roughly four times the capillary flow rates for TBC-treated samples. This relationship appears in all the soils tested, but is less pronounced, capillary flow normally being roughly half the flow to ice lenses for TBC-treated samples. The difference in Figure 6 can be attributed partly to the clay content of soil 14-E being higher than that of soil 14-D.

In general, water flowed faster through untreated soil 1 in. thick under a moisture tension gradient of 50 cm than through 2-in. thick soil to a growing ice lens; however, the reverse was true for TBC-treated samples. That is, water flowed roughly half as fast through 1 in. of soil under a tension gradient of 50 cm as through 2 in. of soil to a growing ice lens. This means that the ice lens test generated a driving force equivalent to a moisture tension gradient of about 200 cm in TBC-treated samples. In untreated samples, the driving force equivalent was below a moisture tension gradient of 25 cm. This further suggests desaturation was prominent in TBC-treated samples but not in untreated porous soils.

Similar data for the ice lens and capillary conductivity tests are shown for soils 23-D and 11-B in Figures 7 and 8. Both these soils respond well to TBC treatment. For soil 23-D, flows to ice lenses decreased from 0.9 cc/hr for untreated to 0.1 cc/hr, about tenfold, for all TBC levels between 0.01 and 0.15 percent. Capillary flow rates decreased from 1.7 cc/hr for untreated to about 0.05 cc/hr, a thirtyfold decrease, for all TBC levels over 0.025 percent. The more sandy soil 11-B responded similarly, but required about 0.05 percent TBC for optimum effectiveness.

Soils containing more than 50 percent clay characteristically do not respond well to TBC treatment. Figure 9 shows flow rates to ice lenses for soils 11-C and 13-C, which contained 51 percent and 57 percent clay respectively. Maximum water absorption rates during initial stages of ice lens growth were essentially unaffected by adding TBC because the flows for untreated soils were already quite low and limited ice lens formation. The curve for soil 14-D is included from Figure 6A as a reference to show how responsive soils appear on the same plot.

Figure 5. Water absorption rates for Michigan silt (14-D).

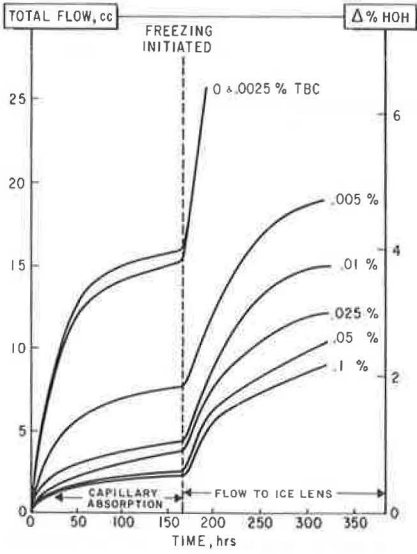


Figure 6. Moisture characteristics of Michigan silt (14-D).

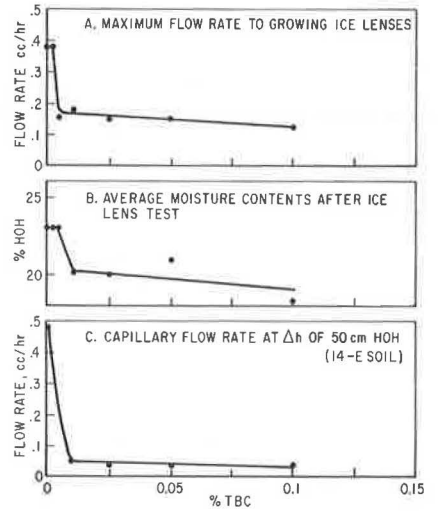


Figure 7. Moisture characteristics of Iowa silt (23-D).

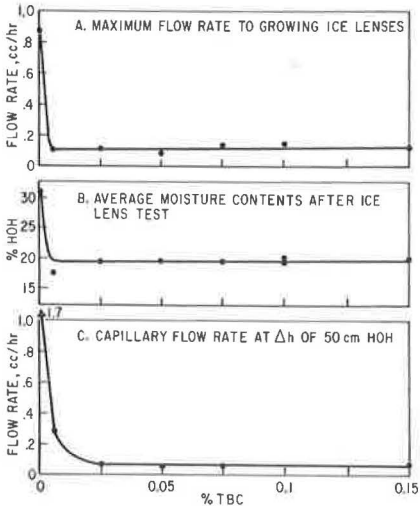
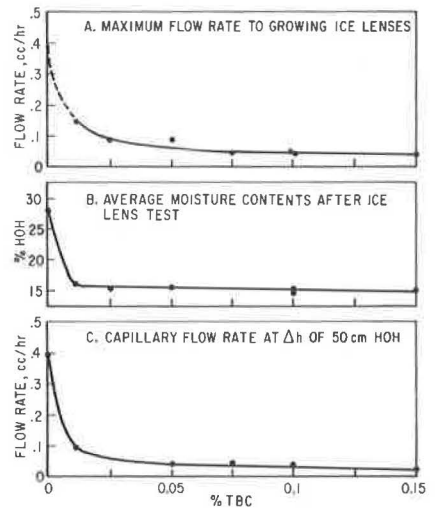


Figure 8. Moisture characteristics of Michigan loam (11-B).



Critical TBC Concentrations

Critical TBC concentrations determined by the ice lens test and the capillary flow test are significantly lower than those indicated by immersed, unconfined compressive strengths (Fig. 10). Immersed strengths have been used exclusively in the past to characterize critical chemical concentrations for field installations. Cores for three different soils, 14-D, 11-B, and 23-D, were immersed while at optimum moisture, and their strengths were measured (Fig. 10).

Two soils, 14-E and 11-B, require 0.1 percent TBC to achieve optimum immersed strength, whereas soil 23-D when immersed moist is near optimum with 0.05 percent TBC even though the strengths were relatively low. When air-dried prior to immersion, soil 23-D produces a strength curve represented by the dashed line in Figure 10, indicating 0.1 to 0.15 percent TBC is required for optimum performance. The discrepancy between critical concentrations indicated by the strength test, particularly after drying, compared to those from the flow tests suggests the strength test may overestimate chemical concentrations needed in field placement for adequate waterproofing. However, there are other questions regarding long-term permanence that must be considered, as well as the relevance of the flow tests to enduring field performance.

West Virginia Soils

Similar tests were run on ten soils from West Virginia. The data from four representative soils in that series are presented in Figure 11 and Table 2. In this case, the capillary flow test was used to predict which soils would respond to TBC treatment in the ice lens test. All four soils produced essentially no immersed strength when treated with TBC and would normally be considered untreatable. However, soils 3 and 10 responded reasonably well to TBC in the capillary flow test. Soils 2 and 5 essentially did not respond to TBC treatment because their clay contents are 71 percent and 54 percent respectively, which produces low untreated flow rates. Generally, more TBC was required to reduce capillary flow in West Virginia soils than in the Michigan and Iowa soils tested.

Average ending moisture contents for the West Virginia soils are given in Table 2. TBC at 0.1 percent decreased moisture contents by 9.2 percent and 19.3 percent for soils 3 and 10 respectively but only reduced moisture accumulation in soil 5 by 3.6 percent. It did not alter moisture in soil 2. These results fall in essentially the order predicted from the capillary flow rate plots in Figure 11. This suggests that a strong relationship exists between the two parameters and that the simple capillary flow test can be used as a major frost-susceptibility criterion for soils similar to those studied in this report.

DISCUSSION OF RESULTS

Chemicals classed broadly as soil waterproofing agents are currently believed to modify soil physical properties by roughly similar mechanisms. The major differences among waterproofing chemicals arise from degree of effect rather than from difference of effect. This can be inferred because chemical structures active as soil waterproofing agents all contain hydrophilic groups that adsorb on soil grains and hydrophobic groups that extend outward from grain surfaces (4). Consequently, TBC can be used as a model compound for studying the mechanism involved in limiting frost heave with soil waterproofing compounds.

TBC appears to limit ice lens formation in frost-susceptible soils primarily because capillary flow through unfrozen soil is reduced and because the tendency for unfrozen soils to absorb water is decreased. Treated soil cores absorb very little capillary water when exposed to controlled moisture tensions, but untreated specimens absorb water freely. Thus, untreated cores begin the freezing cycle at considerably higher water contents than do treated samples. As growing ice lenses consume water from unfrozen soil, additional water is freely supplied through highly frost-susceptible silty soils. But, TBC reduces flow to ice lenses enough to deplete water in the unfrozen

Figure 9. Comparison of maximum water absorption rates of moderately frost-susceptible soils (11-C and 13-D) and highly frost-susceptible soil (14-D).

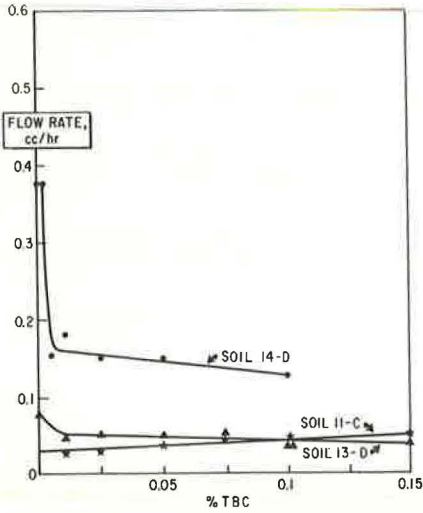


Figure 10. Immersed unconfined compressive strengths of three soils containing increasing levels of TBC.

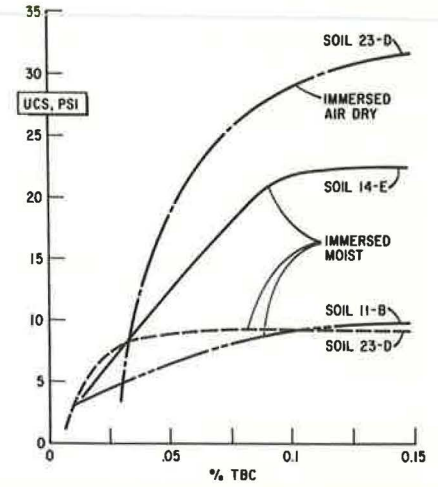


Figure 11. Capillary flow rate (at Δh of 50-cm HOH tension) for West Virginia soils.

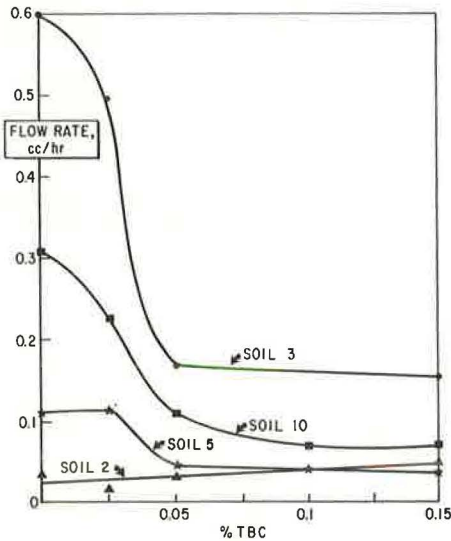


Table 2. Average moisture contents after ice lens test run at moisture tension of 50 cm.

Soil	Percentage of HOH After Freezing Test		
	Untreated	+0.1 Percent TBC	Decrease (percentage of HOH)
West Virginia 3	31.0	21.8	9.2
West Virginia 10	38.0	18.7	19.3
West Virginia 5	25.2	21.6	3.6
West Virginia 2	23.0	22.8	0.2

layer. In this study, flow to ice lenses was reduced threefold in one soil and tenfold in another. As a net effect, Iowa silt loam cores frozen 21 or more days contained about 7 to 10 percent more water in untreated samples than in treated samples. Final moisture contents for other responsive soils were 5, 13, and 19 percent higher in untreated samples. Reduced affinity for water is accomplished by using as little as 0.01 percent TBC in some soils.

Ice lens formation was increased significantly when cores contacted free water as contrasted to higher moisture tensions. However, tensions between 10 and 150 cm did not seem to alter extent of ice lens formation in Iowa silt loam. However, unreported data indicate moisture accumulation in other soils was more dependent on moisture tension, although differences were small. Apparently, the range of tensions considered in this study influenced the net driving force only slightly compared to the driving force induced by water moving from liquid to solid phase during ice lens formation. More careful experiments would be necessary to confirm this result and to test quantitative models previously proposed for calculating maximum frost-heave rates (30).

Temperature gradient also contributed in an inconsistent and apparently minor way to the net driving force during ice lens growth. This observation refutes data reported previously (3) in which temperature gradient was found to significantly alter water flow rates to growing ice lenses. More recent data suggest ice nucleation difficulties generated the earlier trend. Low temperature gradients produced water absorption rates equivalent to those from much higher gradients in earlier studies. As long as the temperature gradient supports nonlimiting heat flow, the contribution to water absorption should be small compared to the driving force generated by freezing liquid water supplied from unfrozen soil.

As discussed previously, hydrophobic chemicals change soil properties when soils are moist as well as when dry. The mechanism in dry soil is at least qualitatively understood (21), whereas that for moist soils is poorly understood (2). Lambe (25) demonstrated that waterproofing chemicals are excellent for decreasing heave rates after soils are dried, whereas the present paper shows newer chemicals limit ice lens growth in moist soils. Data presented in this paper also show that TBC reduces unsaturated moisture flow tenfold to thirtyfold in some porous soils and that all reductions in frost heave observed in this study can be attributed to reduced flow through unfrozen soil. The reduced capillary flow can in turn be attributed to disrupted water structure or perhaps to blocked pores in moist soils.

Even though the reduced capillary flow model appears to be an adequate explanation for how waterproofing chemicals reduce frost heave, other contributing factors may be discovered to be important by using different tests. Conceivably, heaving pressure could be altered by reducing the ice water interfacial energy. Also, the thickness of unfrozen water layers around mineral grains in frozen soil may be changed. Neither of these contributions could be assessed from the present studies.

Soils of varying frost susceptibility and type were included in this study to see how TBC influenced heave in a range of soils. Furthermore, the results from the ice lens, capillary flow, and immersed strength tests could be compared. TBC effectively treated all soils from the A-4 group, but the different tests identified different critical TBC levels. For soils from the A-4 classification, the flow test and the ice lens test indicated that 0.01 to 0.025 percent TBC minimized moisture flow. But, the immersed UCS test indicated that 0.05 to 0.1 percent TBC maximized strength in moist soils. For samples immersed after drying, 0.1 percent TBC is normally needed.

One A-6 soil from West Virginia containing 36 percent clay responded to TBC treatment as indicated by the capillary flow and ice lens tests (requiring 0.1 percent TBC), but air-dry cores disintegrated when immersed. Another A-6 soil from Michigan failed all tests regardless of TBC level because it contained more than 50 percent clay. An A-2-4 soil from West Virginia responded to treatment in the flow and ice lens tests but produced very low immersed strengths. Three soils in the A-7-6 classification did not respond to TBC treatment in any of the tests. Each contained more than 50 percent clay and produced low capillary flow rates when untreated.

Soils responding to TBC treatment as indicated by the hydraulic flow test are classed as SS or SSLs using a scheme proposed by Miller (26). In SS or SSLs soils, solid-solid

contact between sand and silt grains dominates the soil matrix, and moisture flows primarily through capillary pores. Conversely, TBC increased hydraulic conductivity slightly in soils classed SLS, which contained more than 50 percent clay. In SLS soils, clay grains are separated by adsorbed moisture films, and moisture migrates primarily through the adsorbed films. This observation means that TBC and probably other soil waterproofing chemicals do not reduce flow through nonfrozen adsorbed water films at the freezing front. Rather, TBC reduces frost heave by disrupting flow through larger capillary pores.

The correlation between the hydraulic conductivity test and ice lens test strongly suggests that the flow test can be used to characterize the frost susceptibility of soils. The test is simple, fast, and dependable in comparison to most freezing tests. Torrence and Miller (30) developed a relationship between water flux and heave rate for conditions of nonlimiting heat flow that further suggests hydraulic conductivity should be used more extensively as a measure of frost susceptibility. It would be most interesting to compare hydraulic conductivity with the accelerated heaving-rate test recently proposed by Kaplar (24).

CONCLUSIONS

A model soil waterproofing compound, TBC, reduces frost heave in soils primarily by interfering with moisture movement through unfrozen soil. Moisture flow rates to growing ice lenses and capillary flow rates are both minimized in many soils by using 0.01 to 0.05 percent TBC. In contrast, immersed strengths are maximized by using 0.05 to 0.1 percent TBC (or higher optimum chemical concentration than indicated by moisture flow tests). Reduced frost heave in dry TBC-treated soils is easily understood because small hydrophobic pores with high apparent contact angles remain dry when exposed to water, thus reducing flow. In moist TBC-treated soils, reduced capillary flow may be caused by disrupted moisture films in capillaries or by reduced moisture film thickness, both of which reflect alteration of water structure.

Close correlation between frost heave and hydraulic conductivity has been reported by previous investigators and is strongly supported in this study. Therefore, hydraulic conductivity should be seriously considered as a major frost-susceptibility criterion. Soils in the A-4 classification produced high untreated hydraulic conductivities that were reduced threefold to thirtyfold by TBC treatment. Ice lens formation was similarly reduced but not to the same extent. Soils containing more than 50 percent clay (A-7-6 and one A-6 classification) produced low untreated hydraulic conductivity, had little tendency to grow ice lenses, and did not respond to TBC treatment.

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DISCUSSION

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Dr. Brandt is to be commended for his fine paper dealing with the use of soil waterproofing chemicals to reduce water accumulation and frost heaving in soils. The rather innovative and sophisticated laboratory testing techniques and data acquisition systems described by the author are particularly laudable.

In general, the writers agree with the author's objectives and some of his conclusions. There is a very real need "to quantify frost heave as influenced by drainage, temperature gradient, chemical concentration, and soil type." More important, however, from the writers' viewpoint, is the author's implication that the needs of his study were dictated by the fact that "limiting conditions for adequate chemical performance in the field needed to be defined." The writers are not convinced that the results of tests reported by the author, or his suggestion that hydraulic conductivity be "considered as a major frost-susceptibility criterion," entirely satisfy the latter objective.

The purpose of TBC, or for that matter any waterproofing chemical, is to reduce moisture absorption in the field, to retain as much as possible the as-compacted density and strength of the subgrade, and to limit the detrimental effects of frost action, particularly with respect to frost heave and the associated loss of subgrade support during spring thaw. Although the hydraulic conductivity of the subgrade soil and the resultant changes in subgrade moisture content are a very real part of the varied mechanisms associated with the performance of pavements, it appears to the writers that some measure of soil strength and its influence on pavement design might be a more realistic way to evaluate the effectiveness of a waterproofing chemical such as TBC than some of the indirect tests employed by the author. Accordingly, in a series of tests conducted recently at West Virginia University, the writers attempted to evaluate the effectiveness of TBC in terms of its ability to economically reduce the thickness of the required pavement structure for both frost and no-frost conditions (31).

Ten typical West Virginia subgrade soils were selected for study, utilizing the pavement design procedure prescribed by the West Virginia Department of Highways (32). This method is an adaptation of the California procedure originally developed by Hveem (33) and modified by the results of the AASHO Road Test (34). Briefly, this pavement design procedure evaluates the adequacy of the subgrade by using the following steps: (a) preparation of test specimens by kneading compaction, (b) determination of the consolidation or drainage characteristics of these specimens by an exudation pressure test, (c) determination of the swell characteristics of the compacted samples by an expansion pressure test, and (d) determination of the undrained strength of the soil in terms of resistance value as defined by the Stabilometer test. Soils treated with 0.05 percent and 0.10 percent TBC as well as untreated soils were evaluated by this procedure.

For the TBC-treated soils, it was found that, in general, both the exudation pressures and swell pressures were lower than for the untreated soils. For most of the soils, the TBC treatment resulted in higher Stabilometer resistance values. The overall result was that, for no-frost conditions, the TBC treatment resulted in a net reduction in the thickness of the required pavement structure for eight of the ten soils tested. At the 0.05 percent treatment level, the reductions in the non-frost-susceptible subbase ranged from 0 to 2 in., whereas at the 0.10 percent treatment level, the reductions ranged from 1 to 6 in. For the eight soils, where TBC treatment of the subgrade would have led to reductions in the design pavement thickness, only one soil displayed optimization of the TBC treatment at the 0.05 percent treatment level. In fact, there were distinct indications that treatment levels higher than 0.1 percent might have been re-

quired to produce optimum reductions in required pavement thickness. These results differ significantly from the results that would have been obtained by using hydraulic conductivity as a basis for optimization of the TBC treatment level.

Although the methods used in the West Virginia University study did not permit direct laboratory evaluation of the ability of TBC to limit ice lens development, frost heave, and loss of subgrade support during spring thaw, these factors were considered in terms of the procedure developed by Moulton and Schaub (35). This method of evaluating the effect of TBC on the design of pavements for frost conditions is considered to be conservative, and a more direct laboratory procedure has been proposed by Martin (31). Nevertheless, the results obtained were similar to those for the no-frost condition; i.e., very definite reductions in the required thicknesses of the pavement structures were observed for most of the soils studied. However, these reductions were generally less than observed for the no-frost condition at the same treatment level.

Based on these results, the writers concluded that TBC showed considerable promise as a waterproofing soil stabilizer for use with West Virginia soils. However, a cost comparison showed that, at current prices, the use of TBC for the waterproofing of subgrade soils in West Virginia could not be justified from an economic standpoint. This is not meant to imply that TBC treatment would not be suitable or economical for other soils in other areas. In fact, it is recommended rather strongly that each individual soil be evaluated to determine the actual concentration of TBC required for effective treatment. However, the writers do not feel that hydraulic conductivity by itself is entirely adequate to define the TBC treatment levels necessary to ensure optimum pavement design and performance.

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AUTHOR'S CLOSURE

The largest single deterrent to advancements in soil stabilization is the lack of criteria, based on laboratory tests, that can be used to predict field performance. Consequently, I agree with Moulton and Martin that the ice lens test and the hydraulic flow test are inadequate to define all the "limiting conditions for adequate chemical performance in the field." Rather, the need to define limiting conditions led to measuring ice lens growth and hydraulic conductivity in an attempt to understand how and under what conditions waterproofing chemicals limit water migration to freezing fronts in soils. Certainly, the tests cannot be used as direct criteria for a design procedure based on strength-retention parameters. However, they can be used indirectly to help define the conditions under which strength loss during thaw will be minimized by additives.

The design procedure referred to by Moulton and Martin offers considerable promise as a tool for evaluating soil stabilizing chemicals. Design cross sections are calculated from direct laboratory tests, and costs for constructing various cross sections can be compared to determine whether a proposed soil stabilization treatment is competitive

with conventional construction practice. Numerous cross-section alternatives can be evaluated by the model; then cross sections that are both technically and economically optimum can be placed into long-term field tests to verify the design model as applied to chemically stabilized soil layers. However, before this can be accomplished, improvements are needed in predicting strength loss during spring thaw after frost has penetrated subgrade soils, as indicated by the fact that Martin (31) has proposed a direct procedure to evaluate this very important parameter.

Hydraulic flow rates are shown in Figure 11 for only four of the ten soils from West Virginia that have been characterized by Moulton and Martin and by the author. Actually, six of the ten soils produced lower hydraulic flow rates when treated with 0.1 percent TBC than when treated with 0.05 percent chemical, two produced minimal flow at the 0.05 percent treatment level, and the remaining two soils did not respond to treatment. This agrees closely with the minimal treatment levels described by Moulton and Martin. However, data for soils from other states in the original paper demonstrate that different minimum concentrations are indicated by each measurable parameter: flow rate to growing ice lenses, moisture contents after ice lens formation, capillary flow rate, immersed strength of moist cores, and immersed strength of air-dried cores. We have yet to determine which test produces the minimum concentration that can be most reliably translated to expected field performance. Perhaps the procedures advocated by Moulton and Martin can help resolve this problem.

FROST HEAVING VERSUS DEPTH TO WATER TABLE

Richard McGaw, U. S. Army Cold Regions Research and Engineering Laboratory

A laboratory investigation of the influence of water table depth on the freezing characteristics of four soil types is described. The soils ranged from gravelly sand to sandy clay. Specimens were 42 in. long, with external water tables maintained at depths of 6, 18, 30, and 42 in. Specimens were frozen to a depth of 6 in. at rates of penetration between 0.10 and 0.50 in./day. The following relationships were obtained and are shown in the paper: rate of heave versus rate of penetration, rate of heave versus water table depth, and heave ratio versus water table depth. Portions of the data are extended graphically to give estimates of the influence of water table depths in excess of 3.5 ft. Rate of heave and heave ratio (ratio of heave rate to penetration rate) were observed to be functions both of water table depth and rate of penetration. With water table depth held constant, rate of heave increased with faster penetration rates. With freezing rate constant, rate of heave decreased with deeper water tables. With a single exception, heave ratio was reduced by increases in either penetration rate or water table depth. A reduction in heave ratio is shown to indicate a reduction in the water content of the frozen soil. A simple method is described by which heave ratio data may be used to obtain an estimate of the initial stability of a soil upon thawing.

•ONE of the major factors that controls the amount of heave exhibited by a freezing soil is the availability of free water, which in turn is a function of the proximity of a water table. This report describes a laboratory investigation of the frost heaving characteristics of four typical soil types under various conditions of water table depth.

A laboratory water table is typically developed by placing a specimen on a porous stone to which is applied a pressure head determined by the height of an external constant-level reservoir. The elevation at which the pressure of the soil water is equal to atmospheric pressure is taken to be the elevation of the water table. Under static conditions, the presence of the porous stone is of no consequence, and the water table in the specimen coincides with the water surface in the external reservoir. Under conditions of upward flow, as during freezing from above, the porous stone represents a constant-pressure aquifer of unlimited capacity and hence of essentially infinite permeability. In a dynamic situation, the proximity to the freezing front of the region of constant fluid pressure may be an important factor in producing heave.

Figure 1 shows typical moisture tension profiles at three distinct times during freezing for specimens of similar soil that differ only in length. In each case, the static water table is 6 in. below the top of the specimen. Moisture tensions at the freezing front are taken to be similar, and freezing is at the same rate. Nevertheless, heaving will be greater for the shorter specimen because hydraulic gradients will be generally greater. The effect of the porous stone is to eliminate pressure changes below its own elevation, thereby contributing to the development of high gradients in short specimens concurrent with high levels of hydraulic conductivity (unsaturated permeability). In long specimens (Fig. 1b), the dynamic level of the water table gradually decreases in elevation and hydraulic gradients increase more slowly. In a long specimen, as in most field situations, water will be drawn from lower depths at reduced rates of flow.

Investigations of the effect of water table depth on heaving have been reported elsewhere (1, 2). The results in these reports were tabulated under height of sample (1) and proximity of water table (2). The range of depths investigated was from 4 to 12 in. in the first series and from 12 to 42 in. in the second series; the specimen length in all

cases was the same as the depth to the water table. Freezing rate was approximately 0.25 in./day throughout. An additional unpublished series of tests with two silts from Alaska and Idaho further investigated the effect of the water table at the base of specimens that were 6 to 42 in. in length.

These earlier tests measured the influence of a nonartesian aquifer placed at various distances from a freezing front; the water table was at the upper surface of the aquifer as shown in Figure 1a. Results indicated that heaving could be increased 2 to 3 times by decreasing the distance to the porous stone from 42 to 12 in. A further decrease to 4 in. increased heaving by another 3 to 7 times. It was evident that the proximity of an aquifer to the freezing front had a pronounced effect on heaving.

On the other hand, there are many field situations in which either no aquifer is present or the depth of the aquifer is great. The purpose of the investigation described here was to measure the influence of the water table when an aquifer was sufficiently remote such that moisture tension gradients below the zone of freezing could develop freely. In these tests, the minimum distance between the freezing front and the porous stone at the base of a specimen was 3 ft. Although the arrangement represented a compromise between full simulation and various practical considerations, it was judged to be adequate for the comparatively slow rates of freezing utilized.

There was no evidence to indicate that moisture tension profiles during freezing were restrained by the presence of the porous stones. Because specimen behavior was apparently similar to that of soil layers many feet in thickness, the graphs of heaving results have been extrapolated to indicate the possible effect of water tables at depths in excess of 3.5 ft. Certain tentative conclusions have been drawn from the results of these extrapolations.

MATERIALS AND METHODS

The four soils selected for investigation were a gravelly sand from Pittsfield, Massachusetts; a glacial till from East Boston, Massachusetts; a uniform silt from Manchester, New Hampshire; and a sandy clay from an AASHO Road Test in Illinois. Gradations are shown in Figure 2, and general properties are given in Table 1. Specimen condition before freezing is given in Table 2 for the upper 6-in. portion; water content and degree of saturation were measured after 2 days' drainage under the imposed water table. Complete moisture and density data were obtained for the full specimens, both before and after freezing, but are not shown here.

Sixteen specimens in all were compacted, saturated, and frozen in 42-in. long lucite cylinders (Fig. 3). Half-inch diameter holes along the cylinder length, which were tightly stoppered during the tests, enabled water content samples to be obtained. To minimize wall friction during heaving, we tapered the upper 6-in. portion of each cylinder and lined it with overlapping acetate strips that were lubricated with silicone grease.

Specimens of the first three soils were molded in layers at optimum water content to near-maximum density by using the engineers modified compaction procedure (3) (25 blows of a 10-lb hammer with an 18-in. drop over a 4-in. diameter specimen). Specimens were then saturated from the base with distilled de-aired water under pressure. The clay soil was compacted on the wet side of optimum at 20 percent water content to 90 percent of the engineers standard maximum (25 blows, 5.5-lb hammer, 12.0-in. drop) so that it would correspond to the natural subgrade material. The four specimens of a given soil were then mounted in the freezing cabinet for simultaneous freezing. Thermocouples were inserted at intervals along each cylinder, and water levels were adjusted to the test elevations. After 2 days of drainage, water content samples were taken through the holes in the lucite cylinders. The resulting voids were tightly packed with additional soil of similar water content.

Cabinet temperature was decreased to 33 F and held for 5 days to allow temperature gradients to develop. Freezing was initiated by reducing cabinet temperature to 20 F and by seeding the soil surface with frost crystals. When it was evident that freezing had begun, a precooled steel surcharge weight of 0.5-psi intensity was placed on the

Figure 1. Moisture tension during freezing.

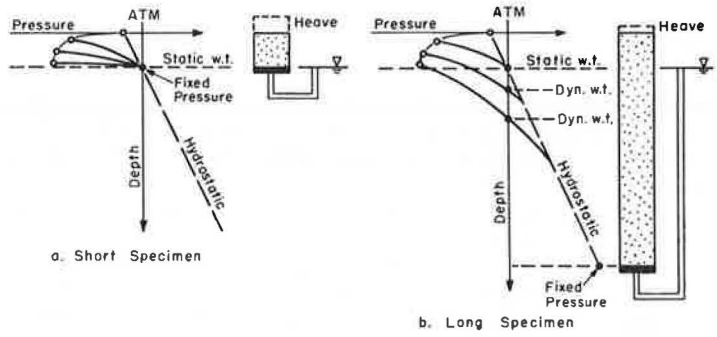


Figure 2. Soil gradations.

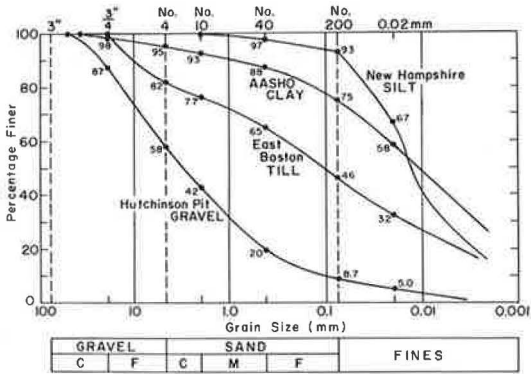


Figure 3. Test arrangement.

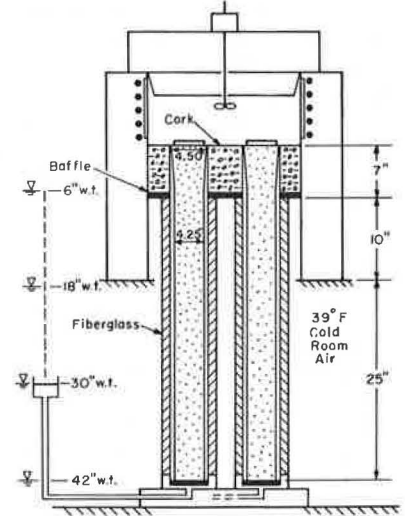


Table 1. General soil properties.

Soil Name	Soil Description	Unified Symbol	Specific Gravity	Atterberg Limits		
				Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index
Hutchinson pit gravel	Gravelly sand	SW-SM	2.75	—	—	0.0
East Boston till	Clayey sand	SC	2.75	22.5	14.5	8.0
New Hampshire silt	Silt	ML	2.71	26.5	20.5	6.0
AASHO clay	Sandy clay	CL	2.74	27.5	15.5	12.0

Table 2. Specimen condition before freezing (upper 6-in. portion).

Soil Name	Specimen Number	Water Table Depth (in.)	Dry Unit Weight (pcf)	Water Content, w _o (percent)	Initial Degree of Saturation, S _o (percent)	Initial Porosity, n _o	Permeability	
							cm/sec × 10 ⁻⁵	mm/day
Hutchinson pit gravel	4	6	141	7.7	96	0.18	—	—
	3	18	143	7.5	98	0.17	—	—
	2	30	141	7.9	99	0.18	—	—
	5	42	144	6.5	96	0.16	—	—
East Boston till	51	6	137	7.6	84	0.20	0.2	1.7
	52	18	134	9.2	90	0.22	0.2	1.7
	53	30	136	8.7	89	0.21	0.2	1.7
	54	42	136	9.0	94	0.21	0.2	1.7
New Hampshire silt	96	6	110	19.2	94	0.35	1.0	8.6
	97	18	110	18.1	92	0.35	1.0	8.6
	95	30	108	20.7	99	0.36	1.0	8.6
	94	42	107	21.4	99	0.37	1.0	8.6
AASHO clay	24	6	109	18.5	91	0.36	0.2	1.7
	22	18	109	19.6	96	0.36	0.2	1.7
	23	30	109	19.5	96	0.36	0.2	1.7
	21	42	106	20.4	96	0.37	0.2	1.7

surface of each cylinder of soil, and cabinet temperature was raised to 28 F. Thereafter, temperature was reduced on a daily basis such that the 32 F isotherm moved downward at a rate of approximately 0.25 in./day. Heave and 32 F depth penetration were recorded daily.

Specimens of the non-clay soils were removed from the cabinet when penetration had reached approximately 6 in. The frozen portion was separated from the column and chilled to 10 F for ease of handling, after which the water content profile was determined by using successive inches of height. Undisturbed samples of the unfrozen portion were cut out at locations that were free of disturbance from previous sampling or the insertion of thermocouples. Complete density and water content profiles were determined.

Water content samples of the unfrozen portions of the clay specimens were taken after penetration had reached 7 in.; the voids were repacked as before. Then the specimens were left in the cabinet for an additional month to investigate heaving under the influence of a stationary freezing front. It was observed that heaving continued virtually undiminished. The final moisture profile showed that further moisture depletion, accompanied by considerable shrinkage and horizontal cracking, had occurred throughout the length of all four specimens.

The results obtained for the frozen portions of the specimens are presented in three sections: rate of heave versus penetration rate, rate of heave versus water table depth, and heave ratio versus water table depth.

RESULTS

Rate of Heave Versus Penetration Rate

The downward progress of the 32 F isotherm was designed to average approximately 0.25 in./day during the test period. It was anticipated, however, that daily and perhaps weekly variations from this average would routinely occur. These variations were utilized in the analysis to provide an estimate of the influence of water table depth for rates of penetration in the range of 0.10 to 0.50 in./day. The indications are that the heave resulting from a given depth of water table depends on the rate of penetration as well as on the type of soil.

The first step in the analysis was to plot penetration of the freezing front p , heave h , and final water content w_f versus days of freezing t for each frozen specimen. Because the temperature of initial freezing is somewhat less than 32 F, total penetration (as measured by the observed height of the frozen column at the end of a test) was always less than the penetration of the 32 F isotherm. Intermediate penetrations of the freezing front were assumed to be in the same proportion. Heave was measured directly, whereas final water content was obtained from the divided frozen column.

Rate of penetration r_p , rate of heaving r_h , and final water content were scaled from these plots. Rates were averaged over periods of 3 to 5 days, except in the New Hampshire silt series where single-day values supplemented the 5-day averages that served as control values. Figures 4 through 7 show the influence of rate of penetration on water content and on rate of heave for each specimen.

In accordance with usual practice, rate of penetration has been expressed as in./day and rate of heave as mm/day; conversion of the latter units to in./day using the approximate value of 25 mm/in. results in an error of less than 2 percent. Intercepts which cross the curves at penetration rates of 0.10, 0.25, and 0.50 in./day mark off the rates of heave that are used in the next section to develop the relationship between rate of heave and water table depth. Numbers placed near the intersections give the corresponding ratio of heave rate to penetration rate in units of mm/in. This ratio is related to the heaving and thawing characteristics of a soil and has been designated the heave ratio (analogous to void ratio) with symbol R . Thus,

$$R = r_h/r_p = (dh/dt)/(dp/dt) = dh/dp \quad (1)$$

where $h = h(t)$ and $p = p(t)$ are the heave and penetration intervals occurring in time t (measured in opposite directions from a common reference level).

Figure 4. Water content and rate of heave as functions of rate of penetration (Hutchinson pit gravel).

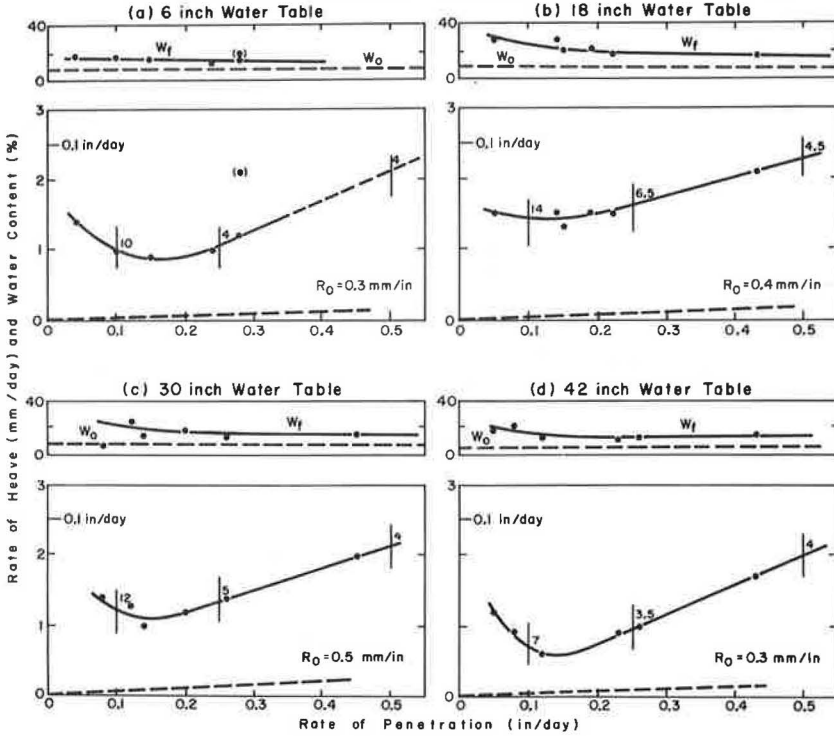


Figure 5. Water content and rate of heave as functions of rate of penetration (East Boston till).

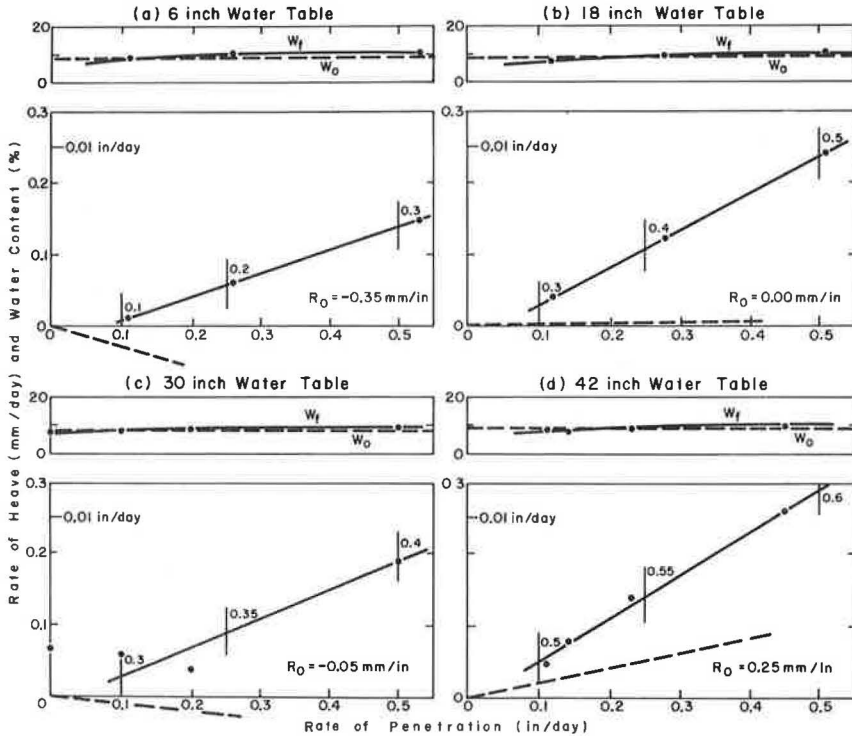


Figure 6. Water content and rate of heave as functions of rate of penetration (New Hampshire silt).

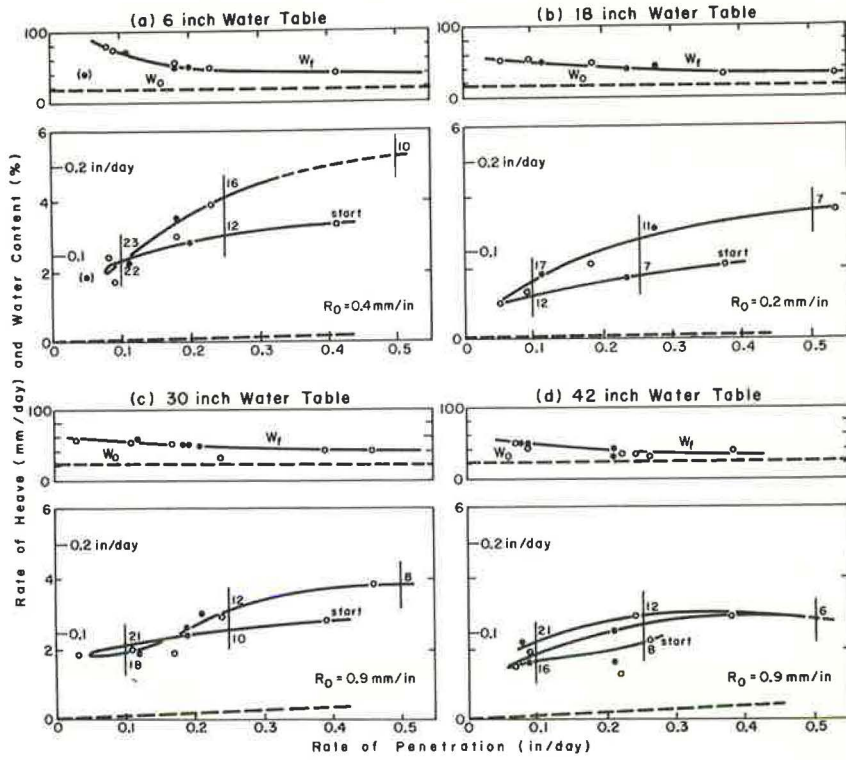
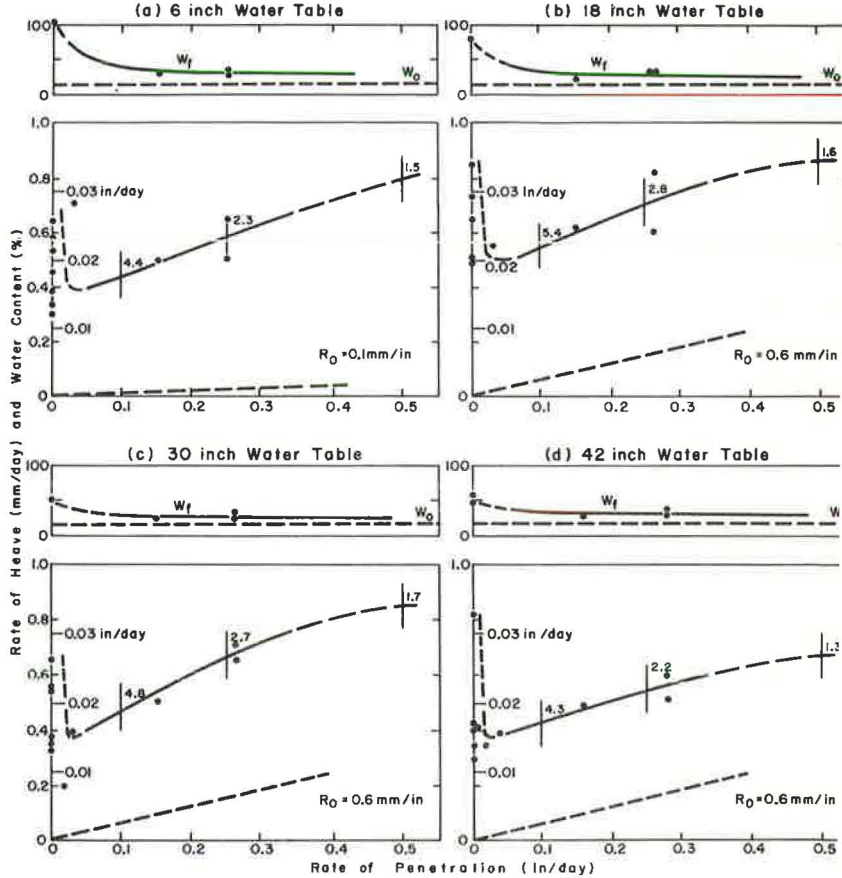


Figure 7. Water content and rate of heave as functions of rate of penetration (AASHO clay).



The relationship between heave ratio and water table depth is developed in a subsequent section. However, a remark concerning R_o is pertinent here. R_o is the heave ratio resulting from the expansion of the initial in situ water upon freezing; one expression for its calculation can be shown as

$$R_o = n_o[(S_o/S_f) - 1] \quad (2)$$

where n_o is the initial porosity and S_o and S_f are the initial and final degrees of water saturation respectively. Total expansion ($R_o \cdot r_p$) is indicated by a dashed line at the bottom of the graphs shown in Figures 4 through 7.

Values of n_o and S_o are given in Table 2. The maximum value of S_f for an ice-saturated specimen would be 91.7 percent. However, S_f has been taken to be 90 percent in the calculations to allow for a small volume of air entrapped in the frozen soil.

The test data indicate that the rate of heave generally increases with the rate of penetration but to a smaller degree with deeper water tables. There is also a region at very low rates of penetration where heaving appears to be governed by factors other than penetration rate. Presumably, the silt would have shown a similar region had rate of penetration been sufficiently slow. On the other hand, it is not so likely with East Boston till, inasmuch as the heave rate had dropped to zero while the penetration rate was still finite.

Rate of Heave Versus Water Table Depth

Figures 8 through 11 show the results for the rate of heave versus the depth to water table. Relationships are shown for three rates of penetrations: 0.10, 0.25, and 0.50 in./day. As expected, rate of heave generally decreases with water table depth for a single rate of penetration. The figures also show, as in the previous section, that the rate of heaving at a given water table depth changes with the rate of freezing.

Two plots of the East Boston till data are given. The data in Figure 9a are taken directly from Figure 5 and give the impression that the rate of heave is increasing with the water table depth. This impression is caused by a discrepancy in the initial degrees of saturation among the four specimens. From Figures 5a and 5c, it can be observed that R_o was calculated to be negative because S_o was less than 90 percent. The interpretation in such a case is that the expansion of the in-situ moisture can be more than accommodated by the initial voids; heaving will not begin until an appreciable water content change has occurred. Specifically, raw heaving rates for this soil are not directly comparable because they are of the same order of magnitude as the heaving due to the initial in situ moisture and because the in situ moisture varies among specimens. For this reason, the data plotted in Figure 9b have been adjusted for an assumed initial degree of saturation of 90 percent. It is then clear that an increase in the water table depth decreases the heaving rate for this soil.

In Figure 11, the data for the specimen with the 6-in. water table have been adjusted to an S_o value of 96 percent on identical grounds.

The results shown in Figure 8 for Hutchinson pit gravel are interesting in that the maximum rate of heave apparently occurs with the 18-in. water table. The variation in initial conditions is negligible for this soil, and the effect is apparently a real one. It is possible that such a relation is common to certain soil types, inasmuch as velocity of flow to a freezing front is the product of hydraulic conductivity and moisture tension gradient, factors that vary inversely one to the other but at different rates.

As previously mentioned, the data have been extrapolated graphically to show the approximate water table depths required to control or eliminate heaving for each soil, all other factors being equal. Within the experimental error, these depths vary with penetration rate. For the gravel, the greatest depths are required with fast freezing. The silt and the clay require the greatest depths with slow freezing.

The result for the fine-grained soils is apparently consistent with the anomaly observed by Higashi (4) in 1958, when he restricted upward flow in fine-grained soil specimens by using textile filaments. Higashi found that, with a simulated deep water table, the slower he allowed freezing to progress, the more heaving he observed. Figures 10 and 11 show a similar effect with the deeper water tables.

Figure 8. Influence of water table depth on rate of heave (Hutchinson pit gravel).

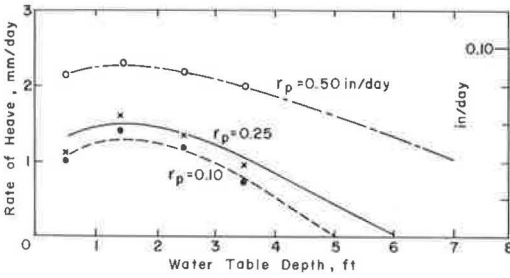


Figure 9. Influence of water table depth on rate of heave (East Boston till).

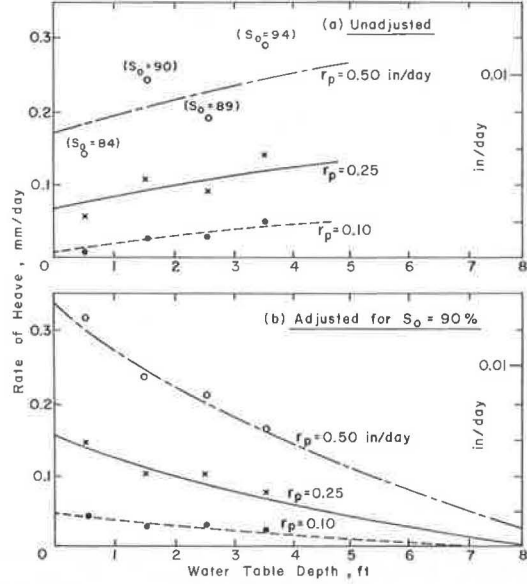


Table 3. Comparison of heave rates (6-in. water table, penetration 0.25 in./day).

Soil Name	Standard Tests (mm/day)	Present Tests (mm/day)	Ratio
Hutchinson pit gravel	6.0	1.0	6
East Boston till	4.5	0.15	30
New Hampshire silt	13 to 26	3 to 4	5
AASHTO clay	7.0	0.6	11

Figure 10. Influence of water table depth on rate of heave (New Hampshire silt).

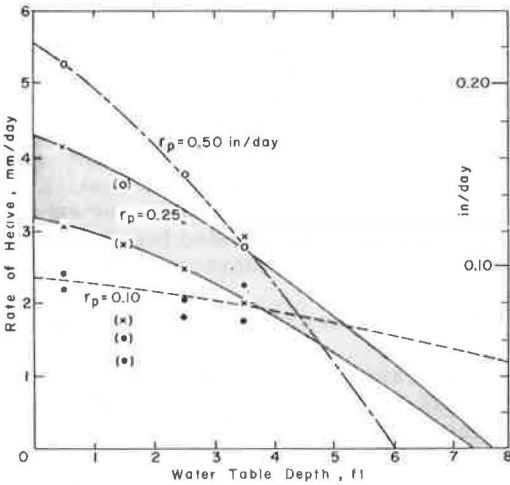
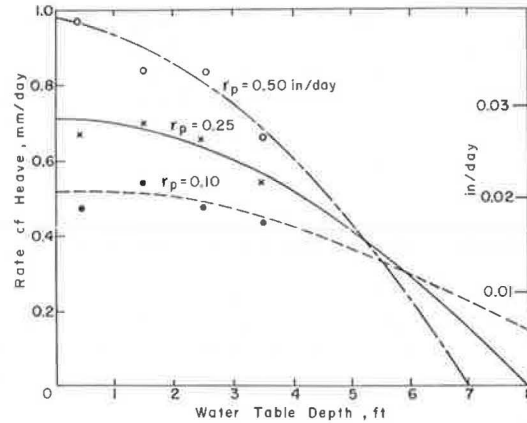


Figure 11. Influence of water table depth on rate of heave (AASHTO clay).



The rates of heave obtained in the present tests for the 6-in. water table and a penetration rate of 0.25 in./day are compared in Table 3 with average values previously obtained with the Corps of Engineers standard test (5), which was intentionally designed to represent a severe freezing condition. Because the only difference in the test procedures is that the standard test utilizes a specimen length of 6 in., the comparison is exactly as shown in Figure 1. In effect, the standard test magnifies rate of heave by a factor between 5 and 30, the magnification factor being greater for the smaller rates of heave. The values obtained here may be considered more nearly representative of heave rates to be expected under average field conditions where the soil column is non-layered.

Heave Ratio Versus Water Table Depth

Figures 12 through 15 show the results for heave ratio versus water table depth. The data shown in Figure 13 are again adjusted for an initial degree of saturation of 90 percent. Relationships are shown for the three rates of penetration. Although the general picture appears similar to that for the rate of heave, there is an important difference. Except for East Boston till, the curves for a given soil are in the reverse order; whereas a slow rate of penetration of 0.10 in./day produced the lowest rates of heave, the same rate of penetration produces the highest heave ratios.

The heave ratio parameter has appeared from time to time in the literature on frost behavior; however, its meaning and importance will likely be unfamiliar to some. A word of explanation may therefore be useful. The quantity R may be expressed in any convenient units of length/length, for example, mm/in. or in./in. It may also be expressed nondimensionally, either as a percentage or as a decimal.

Physically, heave ratio is a useful indicator of the frozen condition because heave ratio, frozen water content, and final porosity are related in a unique way independently of penetration rate. Thus, with a given initial condition, a higher heave ratio always means a higher frozen water content w_f , as a study of the graphs of water content in the first section (Figs. 4 to 7) will verify; a higher final porosity n_f will also result.

A careful study of Figure 5 will reveal that although both the water content and the heave ratio for East Boston till increase in the same direction, the direction is opposite to that for the other three soils. A graphical clue to this anomalous behavior is the intercept on the x-axis when the curves are extended toward the low rates of penetration. Physically, the anomaly is undoubtedly due to a specific combination of moisture tension, degree of saturation, and hydraulic conductivity.

The relations among heave ratio, frozen water content, and final porosity may be derived as follows. From the volume change associated with heaving, it may be shown that the void ratio in the frozen condition is given by

$$e_f = e_o + R(1 + e_o) \quad (3)$$

from whence,

$$w_f = (S_r/G) [e_o + R(1 + e_o)] \quad (4)$$

or equivalently,

$$w_f = (S_r/G) [(n_o + R)/(1 - n_o)] \quad (5)$$

and,

$$n_f = (n_o + R)/(1 + R) \quad (6)$$

In these expressions, S_r is the water-equivalent degree of saturation (here taken to be 90 percent), and G is the specific gravity of the solids (Table 1). R must be entered in decimal form.

Water content w_f and porosity n_f are shown in Figures 12 to 15 as auxiliary scales to aid in the understanding of parameter R . It may be noted that these two factors also describe the condition of a newly thawed soil, prior to drainage or compression. Thus, final porosity is a direct indicator of the average denseness of the granular skeleton, whereas frozen water content is a measure of the drainage required to reach a more stable condition. An estimate of the remolded strength of the initially thawed soil may be

Figure 12. Variation of heave ratio with water table depth (Hutchinson pit gravel).

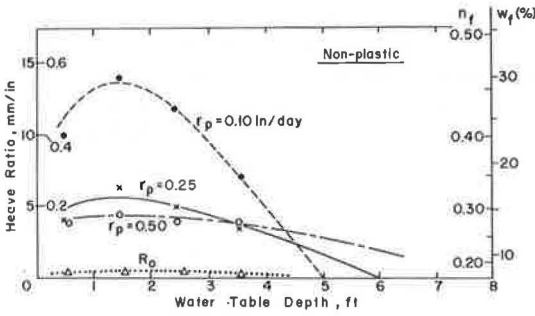


Figure 13. Variation of heave ratio with water table depth (East Boston till, adjusted for an S_o value of 90 percent).

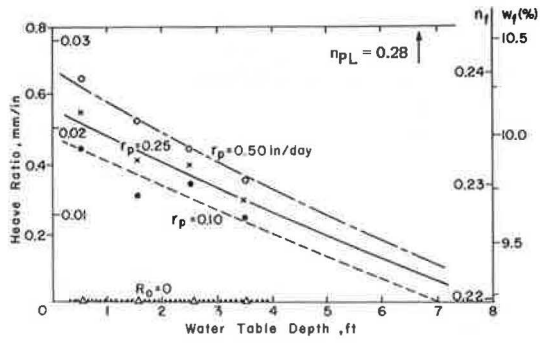


Figure 14. Variation of heave ratio with water table depth (New Hampshire silt).

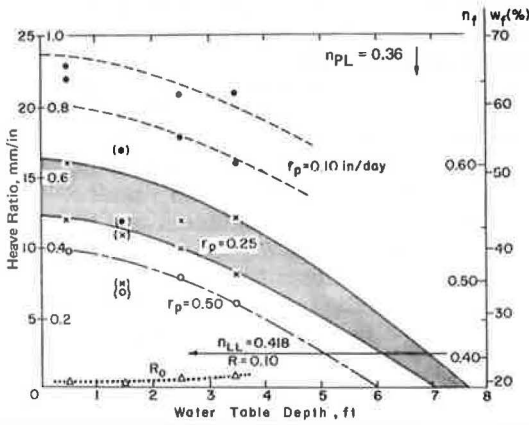
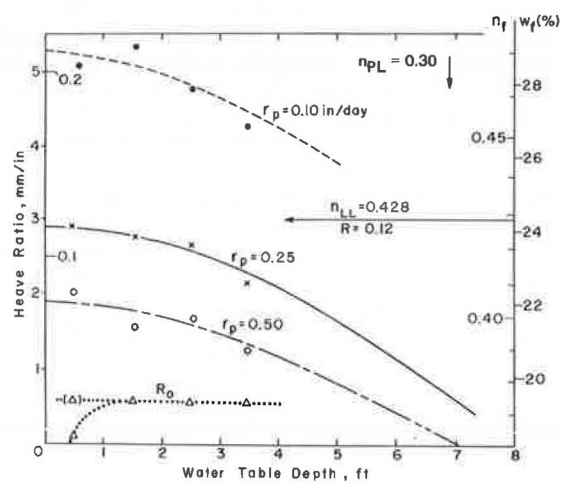


Figure 15. Variation of heave ratio with water table depth (AASHO clay).



obtained by a comparison of n_r with the porosity corresponding to the liquid or plastic limit for the soil. These porosities are indicated in the figures.

The extreme loss of strength that often occurs in silt and clay soils upon thawing may be predicted from data shown in Figures 14 and 15 by comparison with the porosity at the liquid limit. For both soils, the liquid limit porosity corresponds to a heave ratio of approximately 0.10. When the overburden pressure is low, as in the tests reported here, only very fast freezing or a deep water table will prevent this heave ratio from developing. It is immediately obvious that the silt soil will be especially sensitive to detrimental freezing.

SUMMARY

The influence of water table depth on the freezing characteristics of four soils has been investigated in the laboratory. The four soils tested were a gravelly sand, a clayey sand, a silt, and a sandy clay. Water table depths were in the range of 0.5 to 3.5 ft, with a specimen length of 3.5 ft. Rate of frost penetration varied between 0.1 and 0.5 in./day. Relationships were found among rate of heave, rate of penetration, heave ratio, and water table depth. Extrapolations were made graphically to water table depths greater than 3.5 ft.

Rate of heave and heave ratio were found to vary with water table depth and with frost penetration rate. At a single rate of penetration, deeper water tables limited both rate of heave and heave ratio. As rate of penetration increased, rate of heave increased while heave ratio decreased.

Heave ratio and water content after freezing varied inversely with rate of penetration and with water table depth. A decrease in frozen water content always accompanied a decrease in heave ratio.

An apparent anomaly was found for fine-grained soils with deep water tables. As observed by Higashi, when water flow was restricted, rate of heave decreased with an increase in freezing rate. As he also observed, heave ratio continued to decrease.

Heave ratio was shown to be related to the porosity, water content, and stability of a newly thawed soil. The consistency of the undrained thawed soil can be estimated by comparing the porosity after freezing with porosities corresponding to the liquid and plastic limits for the soil.

ACKNOWLEDGMENT

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INFLUENCE OF FREEZING RATE ON FROST HEAVING

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The major thermal influences on the frost-heaving process in soils are reviewed. Laboratory experiments that are designed to predict the frost susceptibility of soil in the field are shown to be strongly influenced by the freezing procedure. It is believed to be misleading to compare the frost susceptibility of different soils based on freezing tests that are carried out at the same rate of frost line penetration. Applying the same rate of heat removal is thought to give more meaningful comparisons of frost susceptibility. In general, increasing the rate of heat removal causes the heaving rate to rise to a maximum followed by a reduction that intercepts the in-place pore water phase change expansion line. Arakawa's concept of ice segregation efficiency is introduced, and its usefulness for assessing frost susceptibility is discussed. The ice segregation efficiency ratio, E , gives the fraction of the heat removed from the freezing front in the soil that is directly attributable to ice lens formation. When $E = 1$, the total heat evolved is from the phase change involved in ice lens formation; when $0 < E < 1$, only a part of the heat evolved is derived from ice lens formation; and when $E = 0$, no ice lensing occurs. Finally, suggestions are made for the improvement of frost-susceptibility tests in the laboratory.

•A SATISFACTORY laboratory test method has not yet been devised that serves as a reliable basis for the assessment of frost susceptibility of soils for all conditions. The variability of the natural environment complicates simulation of the conditions. The procedure up to the present has been to create favorable conditions for frost heaving in the laboratory, where it has not been possible to simulate the desired conditions effectively. However, by using this method, soils can be distinguished that are borderline with respect to frost susceptibility and that may cause unexpected frost action problems in the field.

The nature of the porous media, water supply, and thermal conditions are the three principal frost action factors and, of these, the first factor is probably the easiest to simulate. Representative samples may be taken from the site in question, pretreated, and compacted to simulate the field condition. This can be done easily when the freezing zone is confined within earth structures such as fills and embankments but may be more difficult for undisturbed soils.

Because the amount and rate of lensing is dependent on water supply, the most favorable condition for heaving is normally simulated. Laboratory specimens are usually presaturated before freezing, and in the case of open systems additional moisture is made available at the base of the soil sample. The ice lensing system in the field may operate as an open system from a high water table or as a closed system if the water table is at great depth. These are the extremes, which can be created easily in the laboratory, but the ordinary cases are more difficult to simulate.

This paper is particularly concerned with the third frost action factor—thermal conditions. The importance of the rate at which the soil specimen is frozen and its effect on mobilizing moisture flow is an essential factor that influences frost-susceptibility assessments for field conditions. This is supported by many experiments carried out recently in various laboratories. Finally, a method for quantifying the influence of heave rate with respect to the thermal condition imposed is discussed. This method is based on an efficiency ratio concept recently introduced by Arakawa (1).

The dependency of heave rate on the freezing rate has not always been recognized. Beskow (2) found that, for a constant pressure (load on the soil), "the rate of heaving is independent of the rate of freezing." He further stated that "it should be noted, however, that this is completely valid only for relatively permeable soil." Beskow recognized that, for soils where the frost line was stationary, various heave rates were possible and dependent on heat flow (a relatively unique condition), thus this dependence did not hold for a penetrating frost line (more common in the field). Similarly, the U. S. Army Corps of Engineers (9) stated that "rate of heave has been found to be relatively independent of rate of freezing over a range of freezing rates employed in the investigation." This was the basis for their frost-susceptibility classification that is valid for freezing rates between $\frac{1}{4}$ and $\frac{3}{4}$ in. of frost penetration per day.

Higashi (4) studied the rate of heaving in the laboratory and found an inverse relation with frost-line penetration, a result quite contradictory to previous findings. Although this is difficult to understand, the author believes that the highly restrictive water supply influenced the results attributed to thermal conditions by Higashi. In more recent laboratory experiments by Penner (7) and Kaplar (5, 6), the heaving rate was found to be directly dependent on heat flow, i.e., increasing net heat flow and frost-penetration rate increased the heave rate.

Haas (3) concluded from results of field studies that, when the frost line was not penetrating, the heave rate was essentially proportional to the heat conduction difference between the frozen zone and the unfrozen zone. This observation is similar to Beskow's findings. Field data were also presented for the case when the frost line was actively penetrating, and the heave rate was found to be inversely proportional to the frost penetration rate. In a published discussion to the paper by Haas, Penner (8) was able to show that no statistical significance existed between the two variables, frost-penetration rate and heave rate, because of scatter in the data.

HEAT FLOW AND FROST PENETRATION STUDIES

The method used by Penner (7) to study the influence of the rate of heat flow on the heave rate was to measure heat in and heat out with calibrated heat transducers placed at opposite ends of a 3-in. long and 6-in. diameter specimen. At the same time, heave rate and moisture-influx rate were also measured under conditions of unidirectional heat flow. The relevant aspects of this investigation are summarized as follows.

Three soils were studied: a clay (Leda clay) with 64 percent clay-size particles and 36 percent silt-size particles; a silt (PFRA silt) with 9 percent clay-size particles, 43 percent silt-size, and 48 percent sand-size; and a sandy soil (Lindsay sand) with 7 percent clay-size particles, 13 percent silt-size, and 80 percent sand-size, based on the MIT grain-size classification. The dry densities were 91, 110, and 137 lb/ft³ respectively; all the samples were presaturated before freezing. The saturated moisture contents averaged 33.2 percent, 19.2 percent, and 8.2 percent respectively.

The prefreezing procedure was to impose a thermal gradient of about 0.35 C/in. across the water-saturated specimen with the cold end near 0 C. When thermal equilibrium was established, the temperature of the cold end was reduced to just below 0 C and crystallization of the water was artificially induced. The heat flow was then controlled to give a constant rate by manual adjustment of the temperature on the cold side. When a constant heave rate was established, the thermal gradient was increased to give a different rate of heat extraction by lowering the cold-side temperature.

Figure 1 (from a previous paper by the author, 7) shows the results of three rates of heat removal from one of the soils, Lindsay sand. The figure shows the net heat removal (difference between heat in and heat out) and the moisture influx values expressed in terms of the heat released upon freezing. All three soils showed a positive increase in heaving rate when the heat flow was increased.

The results shown in Figure 2 (from a previous paper by the author, 7) were obtained somewhat differently but lead to the same conclusions. In this series of experiments, a new sample was prepared for each heat flow rate; at the same time, the moisture flow rates into the sample were also measured. As before, moisture flow was plotted in terms of latent heat of fusion by using standard values of 80 cal/g of water or 144

Figure 1. Cumulative values of net heat flow and moisture flow versus time.

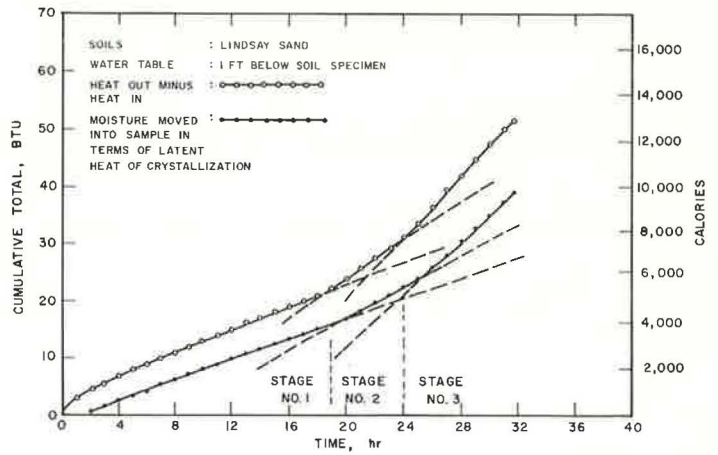
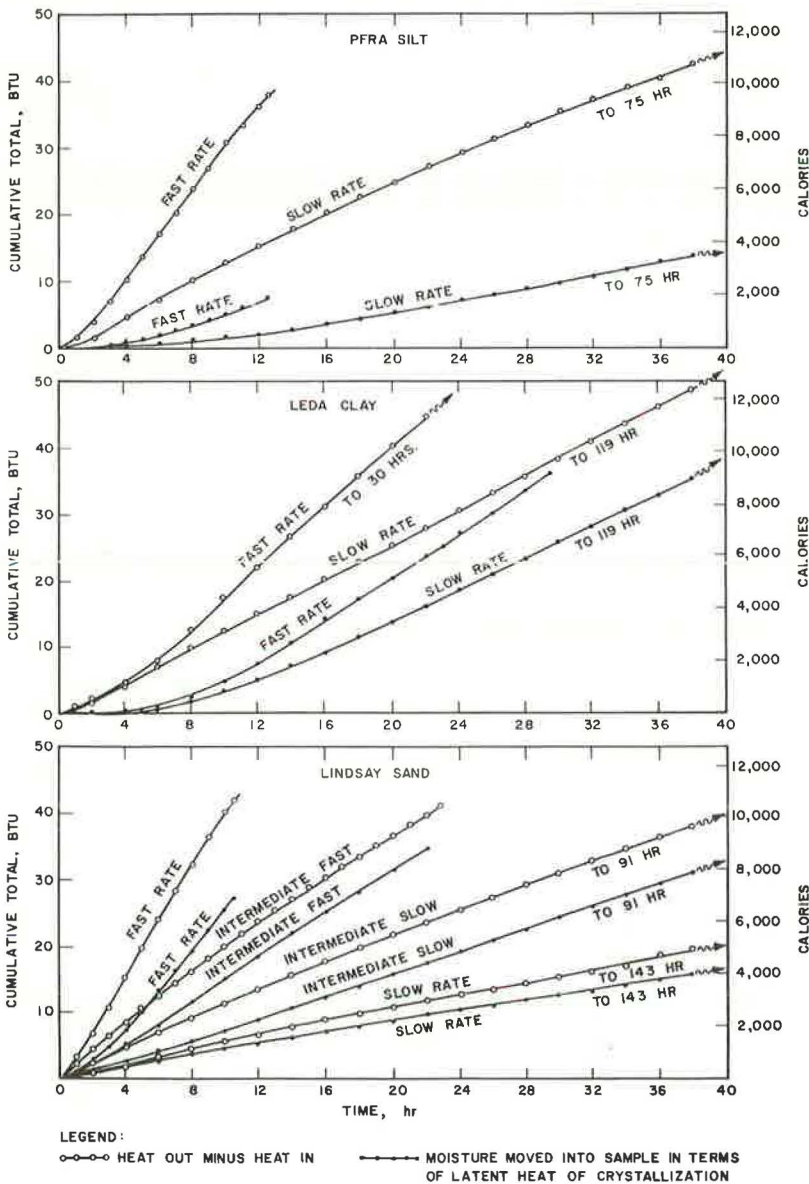


Figure 2. Cumulative value of net heat flow and moisture flow versus time.



Btu/lb of water. It was possible to establish and maintain a constant heat extraction rate after a short period, as may be seen from Figure 2. The apparatus was dismantled, and the thickness of the frozen layer was measured to calculate the average frost-penetration rate.

The influence of rate of frost-line penetration on heave rate has been summarized for these studies in Figure 3. The upper curves show the total heave due to (a) the expansion of the in situ water as the frost line penetrated and (b) the additional water moved into the freezing zone. The lower curves give the heave rates that can be assigned to the in-place expansion of in situ water when it freezes.

Kaplar's (5) recent studies support the positive influence of frost-penetration rate on frost-heave rate (Fig. 4; from Kaplar, 6) in laboratory-conducted experiments. Experiments were also carried out by Kaplar (6) where the frost-heaving rate was measured for different but constant freezing temperatures in the freezing cabinet above the samples. Summary results are shown in Figure 5 (from Kaplar, 5) for four different soils, from which Kaplar concludes "that the heave rate is dependent on or controlled by the rate of heat extraction (up to some unknown critical rate dependent upon the availability of water and the capability of the soil to conduct the water)."

The frost-line penetration rate is, therefore, an important consideration when deciding on the freezing procedure to be used for determining the frost susceptibility of soils in the laboratory. Figures 3, 4, and 5 show, however, that the dependence of the heave rate on heat extraction rate (or frost-line penetration rate) is different for each soil. This dependence cannot as yet be predicted and must apparently be determined experimentally. Yet the criterion currently used exclusively to evaluate quantitatively frost susceptibility is the rate of heaving. It is unfortunate that this important criterion is so sensitive to the rate at which freezing is carried out. Some effort has been made to understand the degree of frost susceptibility in terms of other factors such as heaving pressures, but further work is necessary before these can be applied with confidence to field problems.

ICE SEGREGATION EFFICIENCY RATIO

The author, in previous studies (7), measured the induced water flow into the sample during ice lensing as well as the net heat flow (heat out minus heat in). Arakawa (1) termed the ratio of these quantities the ice segregation efficiency ratio, which is defined by the following equation:

$$E = \sigma L / [K_1(\partial T_1 / \partial x) - K_2(\partial T_2 / \partial x)] \quad (1)$$

where σL gives the heat evolved at the freezing front based on the moisture flow rate (ice lens growth rate) and the denominator is the net heat flow out of the sample, that is, heat out minus heat in. The symbols in the equation are defined as follows:

σ = ice segregation rate, mass of ice per unit area per unit time at the frost line;

L = latent heat of fusion;

K_1 = thermal conductivity, frozen layer;

K_2 = thermal conductivity, unfrozen layer;

$\partial T_1 / \partial x$ = thermal gradient in frozen layer; and

$\partial T_2 / \partial x$ = thermal gradient in unfrozen layer.

The ice segregation efficiency ratios have been calculated from the results shown in Figure 2. In the first instance, the efficiency ratio has been plotted as a function of net heat flow (Fig. 6). In Figure 7, the ratio has been plotted as a function of rate of frost penetration for the same experiments with the aid of Figure 3. Figures 6 and 7 both show the same tendency for the ice segregation efficiency ratio to decrease as the rate of heat removal or frost penetration rate increases. Within the range of rates of heat extraction of 0 to 20 Btu/hr ft² or frost penetration rates of 0 to 3 in./day, the rate of moisture flow into the sample (hence the heaving rate) increased when the rate of heat

Figure 3. Heave rate versus frost-penetration rate.

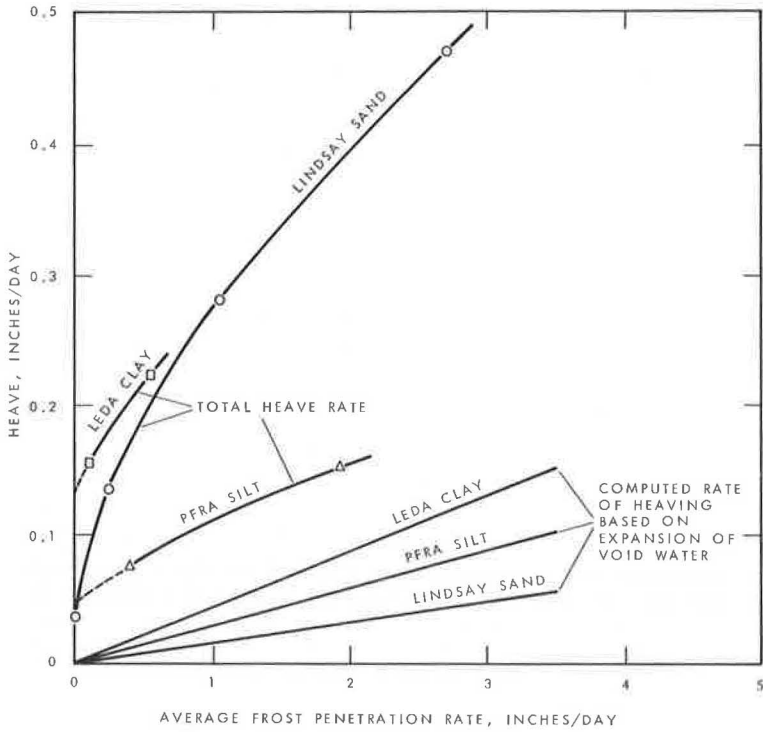


Figure 4. Heave rate versus rate of frost penetration.

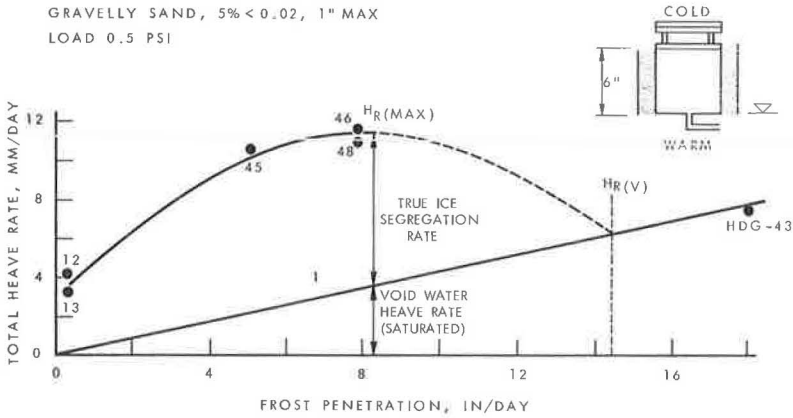


Figure 5. Summary of multi-ring freezing tests.

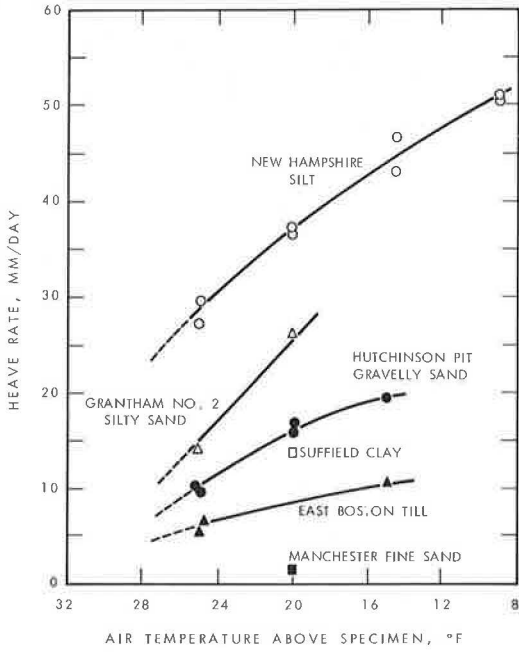


Figure 6. The ice segregation efficiency ratio as a function of net heat flow (heat out minus heat in).

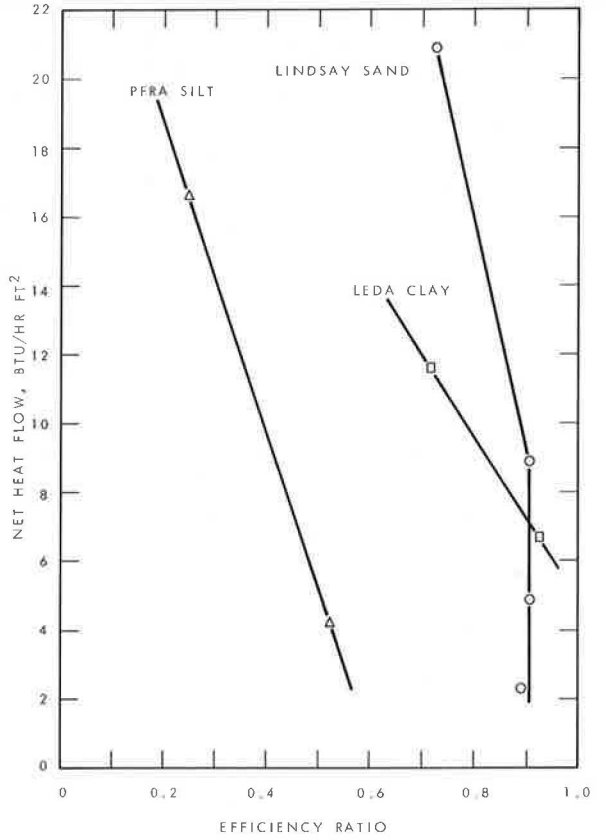


Figure 7. The ice segregation efficiency ratio as a function of frost-penetration rate.

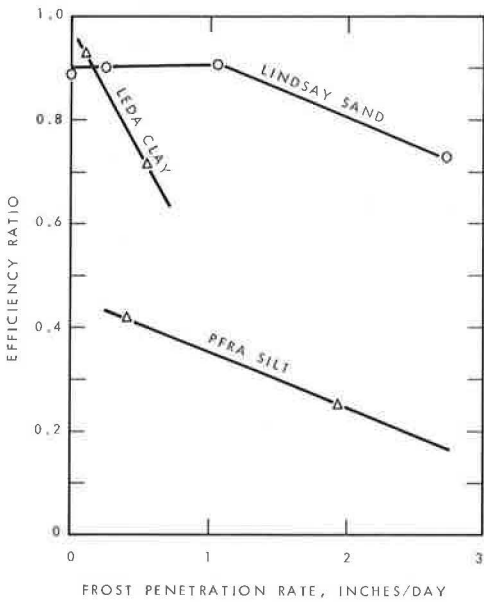
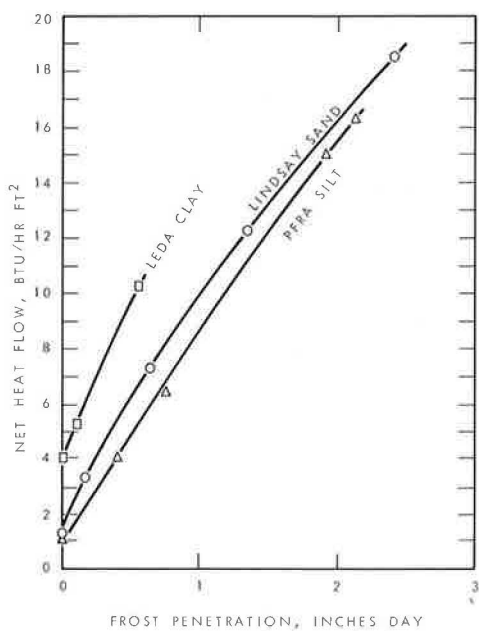


Figure 8. The influence of net heat flow on frost penetration.



removal was increased (Fig. 2). Doubling the rate of heat removal, however, does not double the heave rate. This decrease in ice lensing efficiency is depicted in the ice segregation efficiency ratio and is an indication that the moisture permeability, even in saturated samples, is too low to hold the efficiency ratio constant for some soils as the freezing rate is increased. In Lindsay sand it appeared to remain relatively constant up to 1 in./day of frost penetration (in this case equivalent to about 9 Btu/hr ft²) and then decreased. Sufficient data are not available for the other soils to determine whether this trend applies more generally.

There is a special case of frost heaving with a stationary frost line and, as pointed out by Haas (3) and others, it may occur over a range of values of net heat flow. If we ignore the small amount of heat extraction due to cooling of the water as it moves in the thermal field, $\partial T_2/\partial x$, the efficiency ratio E equals 1, that is,

$$K_1(\partial T_1/\partial x) - K_2(\partial T_2/\partial x) = \sigma L \quad (2)$$

despite the fact that the heaving rate may vary depending on the net heat flow. Arakawa calls it perfect segregation when $E = 1$ and imperfect segregation when $0 < E < 1$. For a soil that does not heave, E equals 0 because σ is zero when no ice segregation occurs.

There is one component of the total heave that is not taken into account in the ice segregation efficiency. In a frost-susceptible soil, the expansion of the existing pore or void water adds to the total heave if the frost line is penetrating. The amount of heave in this case results simply from the phase change of pore water to ice, which gives about a 9 percent volume increase. Kaplar and Penner have both shown the amount this contributed to the total heave (Figs. 4 and 3). The calculation of heave response to the expansion of the void water when it freezes has been based on all the water freezing at 0 C. This is known to introduce an error in the total heave, but other experiments would have to be undertaken to evaluate this correctly for all three soils. Although the latent heat of fusion is the same for both the void water and "outside" water moved to the freezing zone, the amount of heave resulting is vastly different. The amount of heave resulting simply from phase change expansion of the void water may be, in some soils, a considerable portion of the total heave. For Lindsay sand at a frost penetration rate of 3 in./day, it amounts to about 9.5 percent as shown in Figure 3. At present there does not seem to be a satisfactory way of incorporating this contribution into the ice segregation efficiency ratio. Nonetheless, it should not be overlooked in the total assessment of frost susceptibility.

The heave rate tends to increase to a maximum as the rate of freezing is increased and then falls off and intersects the heave-rate curve, which results solely from the void water phase change. This trend is shown in Figure 4. Under very high rates of frost penetration, mobilization of water apparently is not possible. It is also unlikely that under field conditions the frost penetration rate would be high enough to produce the maximum heave rate attainable in the laboratory. The average frost penetration rate is more likely to be less than 1 in./day in areas of seasonal frost in Canada.

Finally, it is misleading to compare the frost susceptibility of various soils based on freezing tests carried out at the same rate of frost-line penetration. Different soils exposed to the same freezing conditions will freeze at different rates. This is shown for three soils (Fig. 8) for which heat flow data and frost-penetration rates were available. As an example, for a heat extraction rate of 10 Btu/hr ft², the average frost penetration in the laboratory was 0.5 in./day for Leda clay, 1 in./day for Lindsay sand, and 1.20 in./day for PFRA silt. The samples were fully saturated and had a moisture content of 33.2 percent, 8.2 percent, and 19.2 percent respectively. Under similar thermal conditions, the main factors that determine the frost penetration rate in frost-susceptible soils are the in situ moisture content and the ice segregation rate. Other lesser influences placed on the foregoing samples are such factors as density and the thermal conductivity and specific heat of the soil solids.

DISCUSSION AND SUMMARY

The major influences on frost heaving that have been stressed in this paper can be attributed directly to the thermal aspect of freezing tests conducted in the laboratory for frost-susceptibility evaluation. It has been shown that, for saturated soils studied in an open system, the following apply:

1. The heat-removal rate influences the heaving rate (Figs. 1 and 2).
2. With increasing heat-removal rates, the heaving rate increases (Figs. 3, 4, and 5) and appears to rise to a maximum; it then decreases and intersects the heave rate based on the phase change expansion of the pore water (Fig. 4).
3. The rate of frost penetration is not the same for different soils at the same rate of heat extraction (Fig. 8).
4. The concept of the ice segregation efficiency ratio, E , is a useful indicator of the frost susceptibility of a soil; as the rate of frost penetration and heat removal is increased, the ratio is reduced (Figs. 6 and 7).
5. The heave-rate response to increasing heat-removal rates is not the same for all frost-susceptible soils (Figs. 3, 4 and 5).
6. A considerable portion of the measured heave may be due to the in situ freezing of the pore water (Figs. 3 and 4), but the present concept of the ice segregation efficiency ratio does not take this into account.

CONCLUSIONS AND SUGGESTIONS

The work reviewed in this paper suggests that the thermal conditions imposed on laboratory samples during laboratory frost-susceptibility experiments are an important element of the testing procedure. These studies indicate that the rate of freezing used should be related to the thermal conditions in the field for which the tests are being performed. Because thermal conditions will vary from year to year in the field, it would be helpful to carry out the freezing tests at two rates, one at less than the minimum rate of freezing and the other at somewhat faster than the maximum rate of freezing expected. One test conducted with an arbitrary rate of freezing is not sufficient to evaluate the response to a different freezing rate because this varies with soil type.

The use of heat meters at both ends of the soil sample (along with measurements of moisture influx, in situ moisture content, and depth of freezing) allows the calculation of a thermal balance. The ice segregation efficiency ratio can be calculated from these results. Such a test procedure, although desirable, may be too costly, and the information obtained from exposing the soil samples to different surface temperatures to impose different rates of freezing may be sufficient. If rates of frost penetration are both slower and faster than the expected field values, the results should permit a good evaluation of frost susceptibility of the material under test.

There does not seem to be a need to carry the freezing experiments beyond a 24-hr (or even shorter) period. It can be seen from the data in Figure 2 that relatively constant flow and heaving conditions are established in a few hours. More useful information can be obtained by spending extra time and effort on conducting heaving experiments at various rates rather than relying on prolonged measurements at one rate.

Finally, the heaving response to different thermal conditions has been evaluated for only a few soils under relatively simple conditions of in situ moisture, moisture supply, and sample density. This work does indicate, however, that improvements can be effected in frost-susceptibility testing procedures used in the laboratory.

ACKNOWLEDGMENTS

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FROST AND SORPTION EFFECTS IN ARGILLACEOUS ROCKS

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As a result of research on aggregate materials, it seemed possible that the formation of ice in some mineral aggregates might be a much less important process of destruction than wetting and drying. Differential thermal analysis in the range of about -20 C to as low as -40 C disclosed that carbonate rocks that were considered to be unsound as a result of standard New York State engineering tests (magnesium sulfate and freeze-thaw) tended to contain relatively little or no water that was freezable in the temperature ranges used. Conversely, in the sound rocks water froze readily and rapidly in all cases. The nature of the contained water was found by determining the amount of water adsorbed in 72 hours at 100 percent relative humidity at 30 C, the amount of water added on immersion for 24 hours under ambient T and P, and the amount of water added after 6 hours in vacuum saturation at ambient T. The rocks that adsorbed the most water had the least freezable water; i. e., the water that could not be detected as freezing was mostly the adsorbed component. It is concluded that sorptive interactions with water vapor or liquid water are far more destructive of shales, siltstones, and argillaceous carbonate rocks than is freezing and thawing. It is further concluded that destruction of rocks (excluding frost wedging) in New York State by the formation of intergranular ice may be a relatively rare phenomenon, at least during initial stages of deterioration. Deterioration of concrete or other highly heterogeneous systems is believed to be caused by both the formation of ice and by sorptive forces, the influence of each depending on many variables. Sodium chloride in water apparently enhances the efficiency of the sorptive forces in rock and may also do so in concrete.

•RESEARCH on mineral aggregates was started at Rensselaer Polytechnic Institute in 1958. Most of the initial work was directed toward understanding carbonate rocks and their included cherts. In 1960, after extensive logging of the faces and cores of many operating quarries, more than 80 representative blocks of carbonate rock and graywacke of lower to middle Paleozoic era were collected from quarries in New York State (6). Table 1 gives the data that were collected at that time.

As part of the research program, an attempt was made to correlate some of the parameters with rock soundness as defined by the standard engineering tests used by the New York State Department of Transportation (formerly Department of Public Works). However, for the most part, the lack of correlation between soundness and other measured parameters was striking. It was apparent that the soundness of the rocks studied was not related in a simple way to most of the many parameters studied.

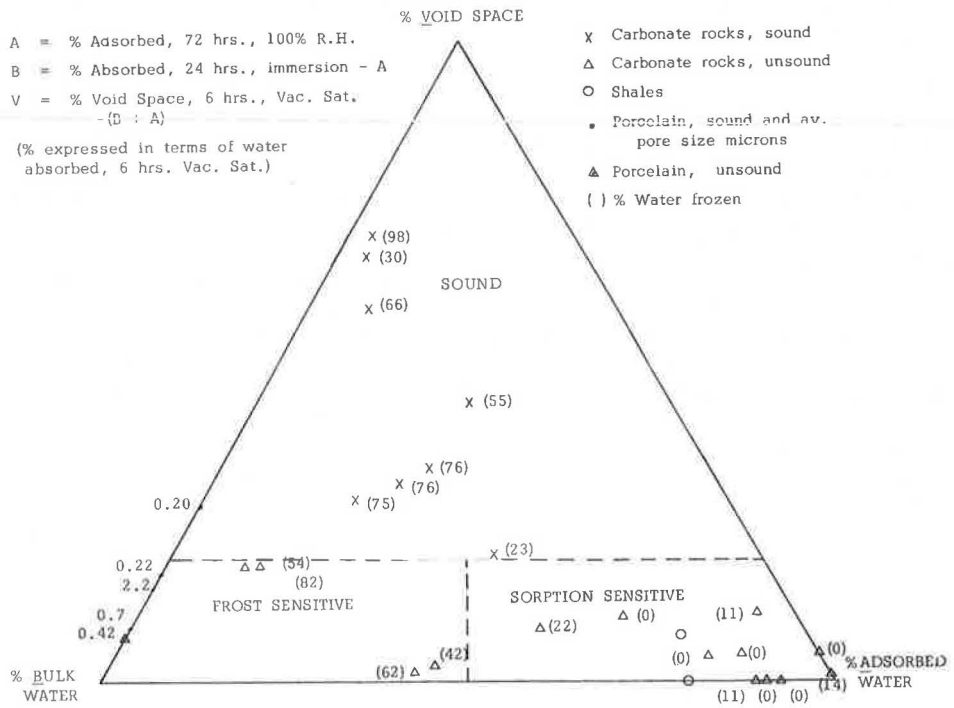
However, some measurements made at the end of the initial research seemed to indicate a tendency for the carbonate rocks having high freezing and thawing losses (as determined by the department's 25-cycle test on unconfined aggregate in 10 percent NaCl solution) to take on water rapidly on immersion and to take on only a little more under vacuum saturation conditions. These rocks also tended to hold water strongly under vacuum drying. Conversely, those with low freeze-thaw test losses tended to take on water more slowly and to take on considerably more when immersed in water under vacuum. They also tended to lose water more rapidly on vacuum drying.

The experiments were not refined because of lack of time, and the results were not wholly consistent. (Later work seems to indicate that refinement of the procedures might be valuable because, so far as carbonate rocks are concerned, reaction to the

Table 1. Carbonate rock and graywacke data.

Type of Rock	Characteristic	Test	Number of Samples Tested	Comment
Carbonate	Chemical	Chemical analysis	80	
		Normative analysis	80	
		X-ray, clays	80	
		Insoluble residue	80	
		Dedolomitization	80	
		Dedolomitization, pH	80	
		X-ray, dedolomitized specimens	3	
	Petrographic	Description		All formations Typical units
	Photomicrographs			
	Physical	Specific gravity, bulk dry	79	
		Specific gravity, bulk saturated	79	
		Specific gravity, apparent	79	
		Specific gravity, true	79	
		Porosity	80	
		Compressional wave velocity	79	
		Shear wave velocity	20	
		Poisson's ratio	20	
		Thermal expansion	24	
		MgSO ₄ soundness	79	
		Na ₂ SO ₄ soundness	79	
Deval abrasion		75		
Water absorption		79		
Wet-dry sensitivity	10			
Rate of absorption and loss of water	15			
Service failures checked	4			
Graywackes	Chemical	Chemical analysis	5	Remarkable similarity Typical samples
		X-ray, clays		
	Physical	Water absorption	34	145 hr
		Rate of water absorption	12	
		Freeze-thaw	18	9 in duplicate
		Pore-size distribution	14	
		MgSO ₄ soundness	18	9 in duplicate (many additional small samples run at RPI)
		Reproducibility, MgSO ₄ test	2	
		Particle shape versus MgSO ₄ test	85	Mostly nonstandard tests

Figure 1. Sorption characteristics versus soundness.



freezing and thawing test is apparently related to the quantity and ratio of adsorbed or tightly held water relative to bulk water.)

Two other critical observations were made during this initial work.

1. Several homogeneous carbonate rocks not visibly or excessively argillaceous deteriorated on wetting and drying alone. These "wet-dry sensitive" rocks were sensitive to the freeze-thaw test also.

2. It is known that the department's 25-cycle freezing and thawing test on unconfined mineral aggregate in 10 percent NaCl solution is far more severe than freezing and thawing in pure water, just as severely salted sections of concrete deteriorate more rapidly than nonsalted or less salted sections. Yet salt probably reduces the total quantity of ice that forms in concrete and in aggregate and lowers the temperature at which ice forms (thus probably decreasing the number of freezing cycles).

The question arising from these considerations was, If these carbonate rocks deteriorate on wetting and drying alone and if they really deteriorate more rapidly with less ice (e. g., in the NaCl solutions), then is ice necessarily a major requisite for the deterioration process?

Work was then started to study the process of ice formation in sound and unsound rocks. Quantitative differential thermal analysis in the freezing ranges was performed on most of the aforementioned rocks, based on the concept that the amount of heat given up on freezing of contained water is directly related to the amount of ice that forms.

This paper summarizes the results of the study and provides related research and concepts. Detailed summaries of the work at Rensselaer are given elsewhere (3, 5-9, 11, 13, 14, 20, 22, 30).

SELECTION OF TEST SAMPLES

The rock test samples studied by Hudec and Dunn consisted mostly of the carbonate rocks referred to previously. Basically, the rocks were categorized as sound or unsound by their percentage of loss in the department's freeze-thaw test. At the time of the research, a loss of more than 3 percent was considered a failure by the department (the limit is now taken as 10 percent). The test is highly significant for carbonate rocks in New York because, from personal observations by the authors and other observers of virtually every carbonate layer quarried in the state, no carbonate rock that deteriorates rapidly on natural exposure has ever been found to have low losses in the test. The cutoff between high and low is necessarily vague but is somewhere between 5 and 10 percent.

However, two points should be stressed: (a) The department's freeze-thaw test results have no similar correlation with noncarbonate rocks with the possible exception of shales and siltstones; and (b) the results of the test do not necessarily correlate with the performance of carbonate aggregate in concrete. Thus, the research on frost action in carbonate rocks was significantly related to natural carbonate rock weathering (and probably argillaceous rocks in general), and no further correlations should be made without further research on other rocks or on concrete.

The rock samples thus were from a very well-understood and relatively narrow rock system, i. e., primarily carbonate rocks with varying amounts of clay (mostly illite, with some kaolinite and chlorite), quartzose sand, and silt. They were, as well as could be ascertained, typical of most significant carbonate rock strata exposed in quarries of New York State from Utica west and from Utica north. The rocks were from the Cobleskill, Rondout, Manlius, Onondaga, Lockport, Pamelaia, and Lowville formations of Paleozoic era; most formations were sampled several times in an effort to test all typical rock variations. The individual blocks were fresh, massive, and from a single bed in all cases and, so far as possible, each block was homogeneous in color and lithology. Zones that were obviously shaly or had other obvious surfaces of weakness were eliminated. The test samples taken from the blocks were mostly cores $\frac{3}{4}$ in. in diameter and 2 in. in length and drilled from slabs 2 in. thick, which were cut parallel to the bedding planes of the test blocks. The method gave a minimum

variation of physical and chemical characteristics between the various samples of the same block tested; statistical analysis made of sorption properties showed relatively minor variations of character among samples from the same block.

Porcelain cylinders of various and known pore-size distribution and several clays were investigated to try to develop certain additional parameters.

The rocks studied were categorized primarily by their percentage of loss in standard engineering tests for rock soundness (Table 2).

DIFFERENTIAL THERMAL ANALYSIS IN THE COLD RANGE

Equipment to measure the heat released on the freezing of water in rock specimens was made in 1963 by Hudec (8). The problems of construction were rather severe because of (a) the need to screen out minor stray voltages and (b) the need to cool the unknown and the standard cylinder at the same rate within the critical minimum temperature differences. Ultimately, at maximum sensitivity, a temperature of approximately 0.001 C and a heat release of approximately 0.001 calorie were distinguishable between the standard and the unknown. The quantity of water being frozen was determined by using freezing substances with known heat release on freezing, by electro-calorimetric means, and by correlating the peak heights and the areas under the curves of the recorder traces. It is noteworthy that the sensitivity of the equipment was nearly 1,000 times greater than that required to measure the freezing of water in some samples in which little or no freezing was detected.

Freezing runs were routinely taken from ambient temperature to a temperature of about -20 C with selected samples taken to about -40 C. No freezing of water was ever detected at temperatures lower than -12 C. For most specimens, the inception of freezing was at temperatures from -5 to -7 C, and freezing occurred as a single pulse; i. e., all of the water that could freeze froze simultaneously. The total water that froze in 30 carbonate rocks varied between 0 and 90 percent; for reference, in porous porcelain cylinders 95 to 99 percent of the water froze.

RESULTS OF DIFFERENTIAL THERMAL ANALYSIS RUNS

Cooling rocks that were saturated in 10 percent NaCl solution for 24 hours had the following effects relative to the freezing behavior of pure water: (a) The total non-freezable water was reduced an average of about 50 percent, (b) freezing began at from 3 to 7 C lower temperatures, and (c) freezing occurred over a wider temperature range. The addition of ethyl alcohol caused some increases and some decreases in heat release. The addition of formamide, a large polar molecule, for the most part increased the quantity of water that could freeze. (The reason for adding formamide was to see if a breakup of a hypothetically neatly packed adsorbed water structure would increase the amount of freezable water.) Partial results of the differential thermal analysis measurements are given in Table 2.

Much of the water in many rocks was obviously not freezable, and those rocks with the most nonfreezable water appeared to be those that had the highest losses in the department's freeze-thaw test.

SORPTION RESEARCH

Previous research had seemed to indicate that the water that was tightly held (i. e., was low vapor pressure water) was most abundant in rocks that failed in the freeze-thaw test. Further, on theoretical grounds, it seemed likely that the vapor pressure of most adsorbed water was such that it should not freeze at normal freezing temperatures. Because the DTA measurements indicated that a surprisingly large quantity of water was not freezable at normal winter temperatures, it seemed important to sort out the quantities and various states of water in the rocks studied.

The sorption measurements that were made included adsorption isotherms in water vapor at 30 C, vacuum saturation in water, and 24-hour absorption in water. The full details of the procedures and the results are summarized elsewhere (8, 9, 13). We feel that adsorption isotherm measurements, despite their limitations, are critical

Table 2. Results of differential thermal analysis measurements.

Sample Number	Group Number*	Number of Runs	Liquid Medium	Weight-Percent Liquid Absorbed, 24 Hours, Ambient Pressure	Freezing Temperature (C)	Lowest Temperature of Run (C)	Water Frozen (percent)
P-A-1	1	4	Water	1.58	—	-40	0
		1	3% alcohol	1.58	—	-25	0
		1	10% NaCl	1.05	—	-20	0
		1	5% formamide	1.46	-6	-20	28
P-B-2	1	2	Water	0.54	—	-40	0
		1	3% alcohol	0.52	-5	-20	6
		1	10% NaCl	0.43	—	-20	0
		1	5% formamide	0.37	-12	-24	1
24 EC	1	2	Water	0.04	—	-20	0
RA	1	1	Water	4.06	-3	-20	54
		1	3% alcohol	4.4	-7	-20	27
LO 3	1	1	Water	0.33	-11	-30	11
		1	3% alcohol	0.32	—	-30	0
M	1	1	Water	1.20	-4	-40	11
42	2	1	Water	0.35	—	-18	0
		1	10% NaCl	0.36	—	-20	0
R-1-1	2	2	Water	0.08	—	-20	0
P-M-2	2	2	Water	0.41	—	-40	0
		1	3% alcohol	0.41	—	-20	0
		1	5% formamide	0.28	-8	-20	14
OD	3	1	Water	2.81	-3	-20	82
		1	3% alcohol	2.81	-7	-20	97
C-1-1	3	1	Water	0.93	-6	-40	22
		1	3% alcohol	1.23	-8	-20	31
		1	10% NaCl	1.43	-9	-20	12
2c	3	1	Water	1.63	-6	-40	14
		2	3% alcohol	1.54	-7	-20	46
4a	3	2	Water	2.81	-2	-16	62
		2	3% alcohol	1.02	-5	-16	85
5b	4	2	Water	0.73	-5	-20	42
		2	3% alcohol	0.66	-9	-20	45
5a	4	1	Water	0.27	—	-40	0
		1	3% alcohol	0.32	—	-20	0
		1	5% formamide	0.19	-8	-20	18
42JO	4	2	Water	0.04	-12	-20	20
P-A-2	4	2	Water	0.09	—	-20	0
CR	4	1	Water	0.08	-5	-18	51
		1	10% NaCl	0.12	-12	-20	22
K-2	4	1	Water	0.30	-5	-20	25
		1	3% alcohol	0.34	-5	-20	16
26EA	5	1	Water	0.07	07	-20	90
26J	5	1	Water	0.23	-5	-20	47
EA 1	5	1	Water	0.29	-6	-20	23
P-B-1	5	2	Water	0.12	-7	-20	19
		1	3% alcohol	0.09	—	-20	0
3b	6	1	Water	0.15	-6	-20	66
		1	10% NaCl	0.54	-10	-20	47
8b	6	1	Water	0.31	-6	-20	98
		1	10% NaCl	0.89	-10	-20	66
4b	6	1	Water	0.69	-4	-18	30
		1	3% alcohol	1.14	-5	-18	66
		1	10% NaCl	0.89	-10	-18	25
2e	6	1	Water	0.45	-6	-20	76
		2	Water	0.48	-5	-20	76
6b	7	1	3% alcohol	0.33	-8	-20	30
		1	10% NaCl	0.44	-9	-20	22
		2	Water	1.11	-5	-40	75
7a	7	2	Water	1.43	-7	-20	39
		1	10% NaCl	1.65	-9	-20	23
		1	Water	0.52	-5	-40	55
9	7	1	Water	0.52	-5	-40	55
		1	3% alcohol	0.64	-6	-20	56
Porcelains							
2.2 ^b		2	Water	37.0	-5	-20	83
		1	3% alcohol	38.6	-9	-20	61
1.5		1	Water	45.0	-6	-20	92
0.7		1	Water	23.0	-6	-20	89
		1	10% NaCl	25.0	-9	-20	44
0.42		1	Water	26.0	-5	-20	94
0.30		1	Water	22.0	-6	-20	83
0.22		1	Water	13.0	-5	-20	81
0.20		1	Water	11.0	-6	-20	100
0.15 Not observed							

*Group 1 = high freeze-thaw (25%), high Na₂SO₄ (10%), high MgSO₄ (20%)

Group 2 = high freeze-thaw (25%), low Na₂SO₄ (0), low MgSO₄ (10%)

Group 3 = high freeze-thaw (25%), high MgSO₄ (20%), low to high Na₂SO₄ (1-6%)

Group 4 = intermediate freeze-thaw (5-25%), Na₂SO₄ (11%), MgSO₄ (12%)

Group 5 = low freeze-thaw (3.2%), low Na₂SO₄ (1%), low MgSO₄ (3%) with high calcite (90%)

Group 6 = low freeze-thaw (3.8%), low Na₂SO₄ (2%), low MgSO₄ (5%) with high dolomite (87%)

Group 7 = low freeze-thaw (1%), low Na₂SO₄ (1.3%), low MgSO₄ (3%) with varying proportions of calcite-dolomite-clay

^bAverage capillary radius in microns, as given by Selas Fibronics, Springhouse, Pennsylvania, the supplier.

to understanding the nature of adsorbed water and its potential freezability. However, the results of the isotherm measurements will not be discussed further in this paper.

The relationship among the water that is adsorbed in a humid atmosphere, the added water taken on in contact with water, and the water that could only be taken on under vacuum conditions is highly instructive. Table 3 summarizes some of the data.

Figure 1 shows the types of water for carbonate rocks with more than 0.30 percent absorption. The lower absorption limit of 0.30 percent was taken purely because below this level results become inconsistent. (Rocks with very low absorption may adsorb most of their water, but the quantities are so small that the forces may or may not be disruptive depending, perhaps, on distribution; i. e., evenly distributed small quantities of water may create potentially disruptive forces, but the elasticity of the rock is probably sufficient to absorb them.)

By dividing Figure 1 into three fields, it is possible to include all unsound carbonate rocks in two fields. (It should be stressed that sorption here correlates very well with the freeze-thaw test. No such correlation exists for the sulfate soundness tests.) One field, which is around the sorption corner (Fig. 1), includes all rocks that may be considered to be sorption sensitive; i. e., those in which little ice forms. Included in this group are all rocks known to be sensitive to alternate wetting and drying. The other field includes rocks in which ice forms and which readily approach saturation by soaking in water (about the bulk water corner). Presumably, such rocks are truly frost sensitive. The only carbonate rocks of the study group, which are known to cause deterioration of concrete in New York, are in the latter field. Incidentally, the dolomitic rocks that dedolomitize in concrete apparently lie in the sorption-sensitive field.

Note that the rocks that are sensitive to the freeze-thaw test are below the 81 percent saturation point (relative to the vacuum saturation figure), which is in line, generally, with the critical saturation concept.

It is quite apparent that the rocks that adsorb the most water have the least freezable water. It is, therefore, concluded that the nonfreezable water is mostly the adsorbed component. It is also probable that the addition of NaCl, in further reducing the freezable water (Table 2), in a sense drives the sorption characteristics toward the adsorption corner, i. e., increases the sorption sensitivity.

If the sample population is large enough to be significant, then the results would suggest that, so far as carbonate rocks are concerned, sorption measurements alone might be a way to determine the degree of natural sorption sensitivity and/or freeze-thaw sensitivity. Ultimately, such tests might indicate the potential for water in carbonate rocks to destroy concrete.

SUBSEQUENT RESEARCH

Since the first presentation in 1965 and 1966 of many of the concepts and research methods summarized here, research has been carried out primarily at Pennsylvania State University under the direction of Larson and Cady (15-19). Some research has been conducted at Rensselaer Polytechnic Institute in which various aspects of the original work have been checked and expanded.

Following is a brief summary of parts of the work at Penn State and some comments.

1. Differential thermal analysis in the subfreezing range: Larson, Cady, et al., verified that the use of differential thermal analysis at cold temperatures is a valuable tool for studying the freezing behavior of water, not only in rocks but in concrete mortar bars as well.

The method was modified by the researchers to fit concrete systems, and many different types of aggregate were used with results generally similar to those obtained at Rensselaer.

2. Sorption and frost sensitivity: In referring to the theories proposed by the authors as against those proposed by Powers, et al., Larson and Cady (18) concluded that "the two hypotheses appear to be in direct opposition regarding the role of water in the destructive process." We did not feel then nor do we now that the theories are necessarily in opposition. However, we do feel that the frost sensitivity model does not describe all that occurs in rock.

Larson and Cady seem to have shown that, for concrete systems, some force other than ice and hydraulic pressures related to its formation is an important factor in the deterioration of some concretes. Expansion of concrete in the cold ranges was correlated with the heat released by the freezing of ice. Expansion was found to continue in several cases well after the release of heat from the freezing of ice. Further, the expansion after freezing stopped was inversely proportional to the percentage of water that froze. Larson and Cady suggested a dual mechanism to account for the destruction of concrete—an ice mechanism similar to that proposed by Powers, et al., of the Portland Cement Association, and a sorption mechanism similar to that proposed by the authors.

They proposed modifications of both concepts, however, in order to be consistent with their research and with certain thermodynamic considerations. Their concept that the sorption expansion produces hydraulic pressures similar to those produced by ice is certainly possible. However, if all of the water in such a system is either frozen or adsorbed, it is all tightly held, not particularly mobile, and may not be capable of moving enough to create such pressures.

3. Adsorption—absorption relationships: The general character of the adsorption isotherms that we described seems to be verified. However, Larson and Cady found that the use of the 3-component diagram (Fig. 1) is apparently not justified for rocks other than the types that we originally studied. This is not surprising because we were working with a narrow and special rock system for maximum experimental control. (Work by Han (11) at Rensselaer also indicated that, for graywacke sandstones, the diagram is not applicable.) However, we feel that the use of such sorption measurements should not be abandoned. The sorption measurements were correlated at Rensselaer against the department's freeze-thaw test on unconfined aggregate in 10 percent NaCl solution. Conversely, soundness in the Penn State work was defined by the freezing and amount of expansion of vacuum saturated particles in mercury. The extent of the correlation between the two methods of determination of soundness is not known, and, therefore, conclusions cannot be made without further work.

OBSERVATIONS RELATED TO SORPTION SENSITIVITY

Engineers working with argillaceous rocks in tunnels or other excavations have long known the importance of sealing such rocks quickly to minimize expansion from alternate wetting and drying or humidity changes. Sorption deterioration of argillaceous rocks is readily observed and, at least for shales and siltstones, is so effective that true frost deterioration may be minor or absent. In our experience, the process of sorption deterioration is so efficient that it is difficult to get large, fresh blocks of shale even from quarries in shale. Figures 2 and 3 are examples of sorption sensitivity of argillaceous rocks. The core shown in Figure 3 from the Nonesuch formation at White Pine, Michigan, was fractured by drying the shale, putting it into the steel press, and wetting the shale (23). Later research by the Copper Range Company at White Pine indicated that the amount the Nonesuch shale expands is proportional to the relative humidity.

Work reported by Ollier (21) indicated the importance of wetting and drying as a destructive agent for many rocks, humidity changes and wetting as an expansive force, and the enhancement of the force in the presence of NaCl. [But Carroll (4) does not mention wetting and drying (or sorptive forces) as an agent of rock weathering.]

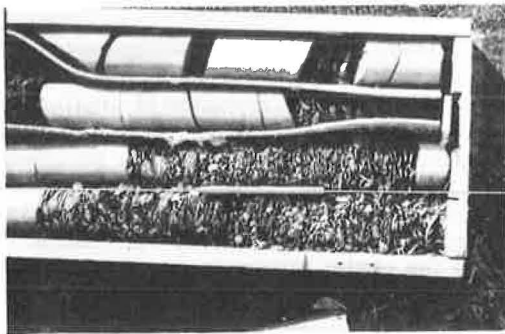
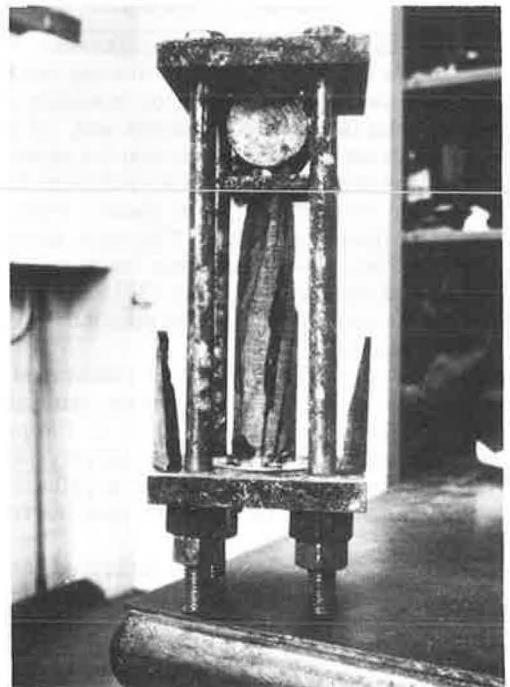
Carbonate rocks that contain argillaceous parting tend to split along the partings, and the mechanism is probably similar to the deterioration of the shales already discussed.

Rhoades and Mielenz (29) indicated the importance of wetting and drying as a mechanism that causes the deterioration of certain rocks, presumably rocks that are similar to some of those reported on here.

Dolomitic rocks with disseminated argillaceous material are somewhat different in character. Sorption-sensitive dolomites appear to be sound when fresh because the clay is often not megascopically visible but is disseminated through the rocks. Petrographic observations revealed that the dolomite crystals are clear and that their

Table 3. Sorption data.

Sample Number	24-Hour Absorption, (percent)	Adsorbed	
		24-Hour Absorption	$\times 100$
			$\frac{24\text{-Hour Absorption}}{\text{Vacuum Saturation}} \times 100$
M	1.40	91	100
P-A-1	1.30	90	95
P-B-2	0.67	92	100
24 EC	0.14	60	69
LO 3	0.30	95	88
RA	3.83	13	82
42	0.43	100	95
R-1-1	0.77	93	99
P-M-2	0.42	89	95
OD	2.91	15	81
C-1-1	1.34	61	91
2c	1.61	100	99
4a	2.81	43	98
5b	0.79	46	97
5a	0.31	74	90
42JO	0.70	80	64
P-A-2	0.03	100	87
CR	0.08	30	99
K-2	0.19	60	100
26 EA	0.09	42	73
26 J	0.28	62	87
E-A-1	0.32	55	80
P-B-1	0.20	70	73
8b	0.65	8	29
4b	0.85	10	34
3b	0.58	18	42
2e	0.47	37	68
6b	0.54	43	66
7a	1.48	29	71
9	0.66	51	56

Figure 2. Summer-weathered shale core alternating with sandstone.**Figure 3. Shale core after drying, pressing, and wetting.**

boundaries are cloudy (20). This observation led to the concept of the "rejection texture," (9) in which it was postulated that dolomite crystals during their growth in limestone tended to reject the clays to the grain boundaries, making the clays more or less continuous and, hence, placing them in a wettable position. Such rocks are highly adsorbent. It was further postulated that each clay particle held as much water around it as the lithic pressure during the last crystallization of the dolomite would allow. Figure 4 shows the textural relationships. Hadley (10) has excellent photomicrographs of such a texture. The electron micrographs of Figures 5 and 6, which were taken during research on certain argillaceous illite and some kaolinite dolomites in Illinois, seem to verify the details of the texture including the postulated space around the clay particles.

Figures 7, 8, and 9 show typical examples of sorption-sensitive dolomites as seen in quarries. Precaution should be taken, however, in automatically labeling such materials as being sorption sensitive because truly frost sensitive dolomites such as samples RA and OD may deteriorate similarly.

The clays in the argillaceous dolomites that were investigated were found to be mostly illite with minor kaolinite and chlorite and hence do not take on interlayer water (6). The water taken into the rocks, in other words, is presumably adsorbed water only, i. e., taken onto the exterior surfaces of the clay and other mineral grains.

DISCUSSION

It should be made clear that we do not feel that the excellent and carefully conceptualized models to explain the deterioration of rock and concrete during winter (involving a moving ice front, growing ice masses, and hydraulic forces 25, 26, 31) are necessarily jeopardized by this research. The research described here was done within a narrow system—primarily carbonate rocks and certain other clay-rich rocks—and should not, without considerable caution, be extrapolated to other major rock types or to concrete systems.

Conversely, the models previously used to explain rock deterioration and deterioration of concrete must certainly be reexamined and modified to accommodate the data described here. We feel that the dual-mechanism concepts of Larson and Cady (18), which describe concrete failure as resulting from a combination of sorptive forces and forces related to the formation of ice approach, are an accurate explanation of the real mechanisms. We feel, further, that the classic concepts of frost deterioration in concrete are most generally applicable to concretes in which the coarse aggregate is highly weathered and porous, as in many gravels (which may or may not have natural frost sensitivity). These classic mechanisms certainly must be modified to explain frost phenomena in concretes made from freshly crushed stone or from argillaceous rocks in general.

Existing soundness and freeze-thaw tests must be acknowledged as being generally insufficient to describe sorption sensitivity. Based on this work, on the experience of the New York State Department of Transportation, on the recent research done at Pennsylvania State University, and on the previous work on frost action by many workers, the following types of observations may be sufficient to define the three components of physical soundness of mineral aggregates for purposes of use in concrete.

1. Detection of sorption sensitivity (largely clay-related): (a) petrographic description: identification of clay-rich rocks with clay in wettable position; and (b) sorption measurements of carbonate rocks to determine the quantity and nature of water.

2. Detection of frost sensitivity (largely pore related): (a) petrographic description: identification of most likely porous rock types; and (b) sorption measurements to determine the quantities of absorbed water and the degree of saturation.

3. Detection of NaCl enhanced-sorption sensitivity: (a) petrographic analysis to determine likely candidates; and (b) freezing and thawing in NaCl-water solutions.

Figure 4. Argillaceous dolomite with rejection texture.

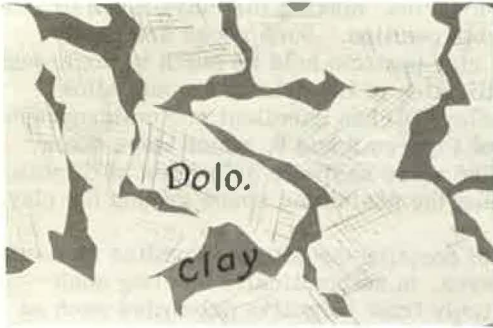


Figure 5. Electron micrograph of rejection texture showing clay plates around dolomite crystals, polished and etched in 0.1 normal HCL.



Figure 6. Electron micrograph of rejection texture showing clay plates around dolomite crystals.



Figure 7. Weathered, alternating sorption-sensitive and sound dolomitic strata.



Figure 8. Sorption-sensitive dolomitic layer after summer weathering.



Figure 9. Typical massive deterioration of argillaceous dolomite with rejection texture.



EXPLANATION OF SORPTION SENSITIVITY

It is obvious that several explanations of the sorption and related phenomena observed may be made at several levels of abstraction and from varying perspectives and yet still not be contradictory.

The original hypothesis to explain the disrupting forces in the rocks—where ice was not involved—was that the expansive forces are the forces of adsorption of water on the surfaces of clay minerals in carbonate rocks.

According to the concept, the force that the adsorbed water could exert was equal to the lithic pressure at the time of the dolomitization process (or the maximum burial pressure if shaly zones are involved). The expansive force is similar to those adsorptive forces that cause soils to expand when wet and to contract when dry.

The presence of the dolomitization process and the clay-rejection phenomenon during dolomitization is a very special situation, which produces a special type of texture not found in other rock types and which partially explains why this sort of noncrystallization of water is not necessarily directly applicable to other rock types. Graywackes, for instance, may be strongly argillaceous; however, on compression in the earth's crust, the sand grains bridge the interstices that contain the clays, thus protecting the clays from maximum lithic pressures. We believe, however, that some graywackes may be made sorption sensitive by the addition of salt (11). In the original hypothesis, the forces pushing the clay particles apart were attributed to a process in which adsorbed water was taken onto the clay surfaces in a quasi-crystalline form, possibly approximating the ice structure. When vapor pressures or available water were increased or when the system cooled, the thickness of the surface-controlled layers increased; thus, such rocks could be broken down—according to the hypothesis—both by temperature and by humidity changes.

At a lower level of abstraction, any concept that accounts for an expansive mechanism in rock must be consistent with modern observations and concepts in surface chemistry. It must also explain why NaCl increases the efficiency of the destructive process. Any complete explanation must be speculative in detail, but we feel that the following hypothesis, as a possible guide to further research, is important.

The expansion of the volume of a solid as a result of its having adsorbed extraneous gases or vapors has been demonstrated for a number of materials, and the particular case of the swelling of wood charcoal on sorption of vapors has been studied extensively by Bangham (1) and by Razouk (27). It was established that the linear expansion that a charcoal rod undergoes when it adsorbs gases or vapors is directly proportional to its adsorbed-film pressure or to its surface-free-energy lowering.

The mechanism of the effect is that sorption, whether of vapor or of liquid, is a spontaneous process whereby the surface tension of the adsorbent is reduced. Surface tension is a constrictive force on a solid, equivalent to an external pressure of several hundred atmospheres. When the surface tension is reduced, it is as if this large pressure were partially relieved; and the solid expands as a consequence. The expansion is anisotropic and will also depend on the internal forces of cohesion, just as do the modulus of elasticity and other cohesive properties of the material. The expansion is therefore more pronounced in directions where the cohesion is weakest; for example, it is normal in the layer planes of a layer-lattice structure. Minerals with marked planes of cleavage, such as clays, are particularly susceptible to this effect.

Presumably, the polarity of water molecules and viscosity changes in water with decreasing temperature enhance the expansion of the adsorbing solid. Presumably also, the high internal surface area of many argillaceous rocks increases the expansivity.

Any effects of dissolved electrolyte on the reduction of surface tension of a solid on immersion in water have not been reported. However, it is reasonable to assume that a solid with an ionic or polar surface, such as have many oxides or silicates, would interact at the solid-liquid interface with ions of opposite charge in the solution. Any such additional interaction at an interface increases the adhesion of the two phases

and also further lowers the surface tension. The expansion of a solid, which is proportional to this effect, would therefore be increased by virtue of these electrostatic interactions.

The effects described are reversible; the solid shrinks proportionately as the surface tension increases on desorption of the vapor or on removal from the liquid.

The relative importance of the order-disorder hypothesis and the surface-tension hypothesis is not known at this time and needs further research.

CONCLUSIONS

The following conclusions can be made as a result of this study.

1. Many common carbonate rocks formerly believed to deteriorate by frost action had little or no ice form within them when saturated and cooled from ambient temperature to temperatures of -20°C and -40°C . Such rocks are considered to be sorption sensitive and may deteriorate by wetting and drying or warming and cooling in contact with water or possibly by humidity changes alone.

2. Many shales and siltstones are also sorption sensitive, probably for somewhat similar reasons to those for the carbonate rocks.

3. In all sorption-sensitive types of rock studied, clay (largely illite) is the apparent common denominator. The forces that cause sorption deterioration of rock are considered to be similar to those that cause adsorptive clay systems to expand and contract on wetting and drying. Because of the nature of the clays, interlayer absorption is not considered to be a major expansive force for the rocks studied.

4. The water that does not freeze is largely the adsorbed component.

5. Deterioration of certain rocks by wetting and drying or by humidity changes has been observed by others. However, the importance of the phenomenon has been underestimated. We feel now that true frost deterioration of fresh, unweathered bedrock may be a less important phenomenon than sorption deterioration. In fact, fresh bedrock that has proven sensitivity to true frost action is difficult to find in New York State.

6. The deterioration of concrete due to its aggregate component is probably related both to sorptive effects and to ice effects. Sorptive effects are most generally likely to be critical when unweathered crushed stone is used; ice effects are most likely to be critical when weathered, porous gravels are used.

7. NaCl in water apparently enhances the sorptive effects in many rocks, whereas it decreases the amount of ice that can form within them. Similarly, salting concrete during winter may increase sorption sensitivity in many cases.

8. Tests designed to determine the physical soundness of rocks or concrete should specifically determine sensitivity to sorption related forces, ice related forces, and NaCl-enhanced forces in order to fully define their soundness.

ACKNOWLEDGMENTS

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PRESSURE-TIME RELATIONSHIP IN LATERALLY STRESSED FROZEN GRANULAR SOILS

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A frozen sand-ice layer, when subjected to lateral pressure at a constant temperature, shows reduction in stress with time. Investigations were carried out to study the general tendency of this reduction in lateral stress and also to investigate the effects of certain parameters influencing this behavior. Two different types of sand were used to prepare the sand-ice specimens with a variety of initial porosity and degree of ice saturation. They were tested at various constant temperatures under selected initial pressures. The magnitude of pressure retained by a frozen sand sample at any time, after commencement of the experiment, was measured by means of a pressure gauge attached to the "pressure unit." The reduction in lateral stress, expressed as a percentage of initial pressure applied to a frozen sand layer, was studied as a function of five variables; initial pressure, the initial porosity of the sand, the degree of ice saturation, temperature of sample, and time elapsed after the application of initial pressure. Experimental results were plotted in the form of pressure-time curves to study the time-dependent behavior of the sand-ice system under applied load. Also reduction in stress was plotted against each of the previously mentioned variables to show their influence on the reduction in lateral stress. A general equation to predict the value of lateral pressure retained by the sand-ice layer at any time after the application of a known initial pressure was derived empirically in terms of the five variables.

•A CONFINED frozen soil layer is known to exert lateral thrust on retaining structures during freezing and also during an increase of its temperature in the freezing range. Experimental evidence shows that the pressure developed in the latter case by a sand-ice layer, having high ice saturation and low initial soil temperature, can reach a magnitude that no longer may be disregarded by the design engineer (1). This behavior is further complicated by the fact that the magnitude of the lateral thrust developed by the frozen soil, caused by the temperature increase, will not remain constant but will change with time even if the soil layer itself is no longer subjected to temperature rise. Naturally, to a design engineer seeking a more economical design, the maximum instantaneous pressure developed between frozen soil and a retaining structure will be of interest as well as the soil behavior under applied pressure (2, 3). This in turn requires a knowledge of the magnitude of the reduced pressure that will be retained by the frozen sand layer after any given period of time.

Accordingly, the object of the investigation presented in this paper was to study the general tendency in which the lateral stress in frozen sand samples that have been subjected to radial pressure at constant temperatures diminishes with time. The investigation was further extended to correlate the reduction in lateral stress with some relevant parameters, i. e., temperature, degree of ice saturation, porosity, initial pressure, and time. Granular material was selected for this experimental work because it is frost nonsusceptible and is used in areas subjected to frost action.

SOILS STUDIED

Two different types of sand were used to prepare test specimens of various combinations of initial porosity and degree of ice saturation. Sand No. 1 was a crushed

uniform sand from Ottawa, Illinois (uniformity coefficient of 1.5 and specific gravity of 2.65); sand No. 2 was a natural variety of well-graded sand from Paris, Ontario (uniformity coefficient of 3.8 and specific gravity of 2.67). The grain-size distributions are shown in Figure 1.

APPARATUS

The main apparatus, a pressure unit (Fig. 2), consisted of a steel cylinder with a cylindrical rubber diaphragm positioned inside and sealed to both ends of the steel cylinder. The small void between the $\frac{1}{8}$ -in. thick diaphragm and cylinder was filled with oil and connected to a hydraulic pressure pump so that any desired amount of initial pressure could be applied to the frozen specimen. The oil pressure was measured by means of an external pressure test gauge connected to the pressure line in which a high pressure valve was installed between the gauge and the pump. The steel cylinder was insulated all around except for the top surface of the sample so that the freezing and thawing of the sand-ice system followed the pattern existing in nature.

To measure the horizontal radial deformation taking place in the sample under applied pressure, we used BLH Type A-9 bonded resistance strain gauges. They were embedded in the sand-ice specimen and connected to a portable digital 8-channel strain indicator (Strainert Model TN8C). Six copper-constantan thermocouples connected to the temperature potentiometer through a multipolar rotary switch were used to measure the temperature of a frozen soil and the air temperature inside the freezer.

PREPARATION OF SAMPLES

To prepare each sand-ice specimen, we thoroughly mixed a known weight of dry sand and a measured amount of water to provide the desired degree of ice saturation when frozen. The moist sand thus prepared was placed in the pressure unit in four equal, gently compacted layers to form finally a $9\frac{5}{8}$ -in. diameter and 4-in. high sample of a certain void ratio. Two 6-in. long strain gauges (previously described) were placed crossways, horizontally halfway down the sample's depth, and symmetrically about the specimen's centroidal axis. Further details on the application and preparation of the strain gauges used can be found elsewhere (1).

The pressure unit with the sand sample was then placed inside the freezing chamber where, after completion of the necessary wire connections to the strain indicator and temperature potentiometer, the specimen was frozen to the required temperature. Before each test, the frozen sample was left at a selected constant temperature for not less than 6 hours to ensure uniform temperature distribution throughout the frozen sample.

TEST PROCEDURE

At the beginning of each test, a selected initial pressure was applied to the frozen specimen, and immediately after that the pressure controlling valve was closed to guard against pressure reduction due to oil outflow. The selected initial pressure indicated by the pressure dial was recorded against zero hour. Afterward, the decrease in pressure shown by the pressure gauge and the change in strain displayed by the strain indicator were recorded after definite intervals of time. The temperature of the specimen, which was kept constant, was frequently checked to ensure no variation during the test. After 24 hours, one set of readings was completed; some tests were continued up to 36 hours or more, but no appreciable change in readings was observed. On completion of a test, the pressure retained by the specimen was released, and, in the case of specimens that were to be reused, the sand-ice system was allowed to recover at 32 F for not less than 36 hours.

TEST RESULTS

The lateral radial pressure σ_r retained by the sand-ice specimen at any time after application of initial pressure was investigated as a function of five variables: initial lateral pressure σ_i , temperature of frozen soil T_f , initial soil porosity n , degree of ice saturation S_i , and elapsed time t .

More than 75 specimens were included in the program. For each investigated specimen, the initial pressure was selected from 25, 50, 75, or 100 psi; temperature was limited to 30, 25, 15 or 0 F; and degree of ice saturation used was 33.3, 50, 66.6, or 100 percent. Initial porosities chosen were 46 percent or 40 percent for specimens of sand No. 1 and 36 percent or 30 percent for specimens of sand No. 2 because they could be obtained with ease without excessive compaction effort. Combinations of these variables were planned in such a manner that maximum information could be obtained from the 75 experiments conducted. Test data were compiled and graphs were drawn to investigate the time-dependent behavior of the sand-ice system under applied pressure as well as the influence of the previously mentioned parameters on this behavior.

Figure 3 shows the relationship obtained between the lateral pressure σ_t retained by frozen sand specimens and the corresponding elapsed time t for indicated values of n , S_i , and T_f . The values of initial pressure σ_i applied to the investigated specimens on commencement of experiments can be read on the ordinates when $t = 0$ hour. All the pressure-time curves show the same tendency of rapid decrease in pressure during the first few hours and then a slow rate of this decrease with time.

For convenience, a term "reduction in stress" (R_t), expressed as percentage of initial pressure, is introduced and defined as

$$R_t = (\sigma_i - \sigma_t) / (\sigma_i) \times 100 \quad (1)$$

where

R_t = reduction in lateral stress after time t , expressed as a percentage of initial pressure;

σ_i = initial pressure at $t = 0$, psi; and

σ_t = actual pressure retained after time t , psi.

In the conducted investigation, the decrease in the pressure retained by a frozen soil layer was associated with a nonlinear increase in lateral deformation (strain). Hence, the common rheological term "relaxation" is not applicable in such cases, and the term "reduction in stress" was adapted. Typical experimental strain-time curves showing the change of strain with time are shown in Figures 4 and 5. It can be seen that the strain increases rapidly during the first few hours (causing a decrease in the diameter of the investigated specimen) and then very slowly after approximately 24 hours. The effect of the temperature of the frozen soil on the magnitude of the developed strain can be observed by comparing Figure 4 with Figure 5.

Figures 6 to 9 show some typical "reduction in lateral stress versus time" curves obtained for different combinations of n , S_i , T_f , and σ_i . Figure 6, magnitudes of n , S_i , and σ_i are kept constant to show how the " R_t versus t " curves tend to change at different temperature levels. Similarly, Figure 7 shows the effect of the different initial pressures on the behavior of the R_t versus t curves. Figure 8 shows the effect of porosity, and Figure 9 shows the effect of the degree of ice saturation on the previously mentioned curves.

For each " R_t versus t " curve, the relationship between reduction in lateral stress (expressed as a percentage of initial pressure) and the corresponding time could be approximated by the following general equation.

$$R_t = A \exp (-B/t) \quad (2)$$

where

$A = f(n, S_i, T_f, \sigma_i)$ = reduction factor,

$B = f(S_i, \sigma_i, T_f)$ = time factor, and

t = time in hours.

Every " R_t versus t " curve was obtained as a function of n , S_i , T_f , and σ_i ; therefore, for each combination of these variables, there are unique values for reduction factor A and time factor B . If we place $t = 0$ into Eq. 2, it follows that $R_t = 0$, which satisfies the initial boundary condition. When t is taken to be very large, R_t approaches A , which defines the maximum expected reduction in lateral stress for a given frozen soil.

Figure 1. Grain-size distributions.

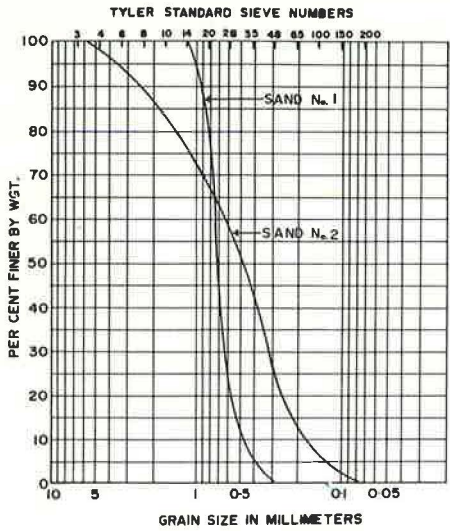


Figure 2. Pressure unit showing steel cylinder with rubber diaphragm.

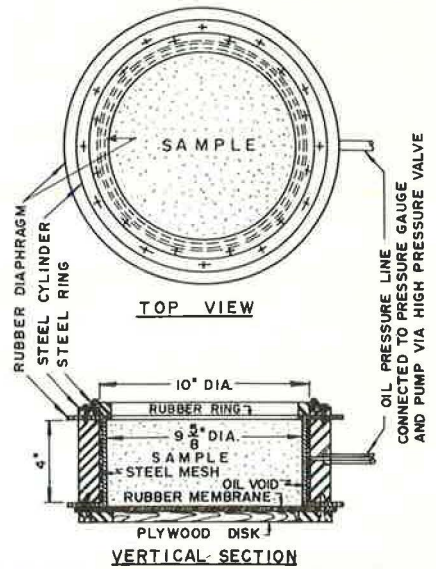


Figure 3. Pressure-time curves for specimens at 0 F having 100 percent ice saturation under three different initial pressures, 25, 50, and 100 psi.

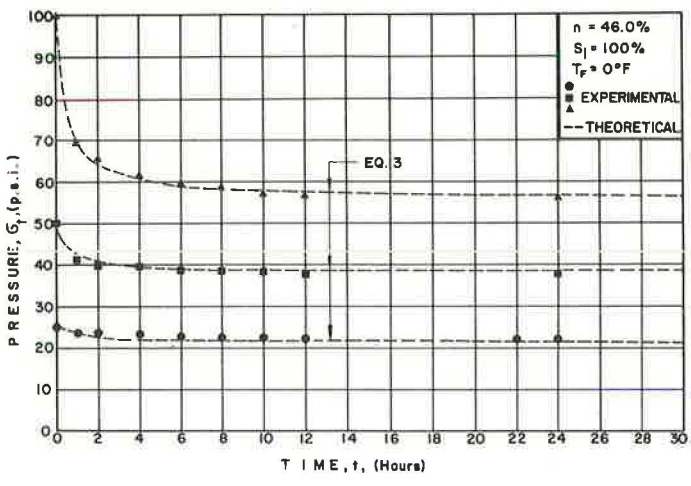


Figure 4. Experimental curves showing strain-time relationship for three different initial pressures under indicated conditions.

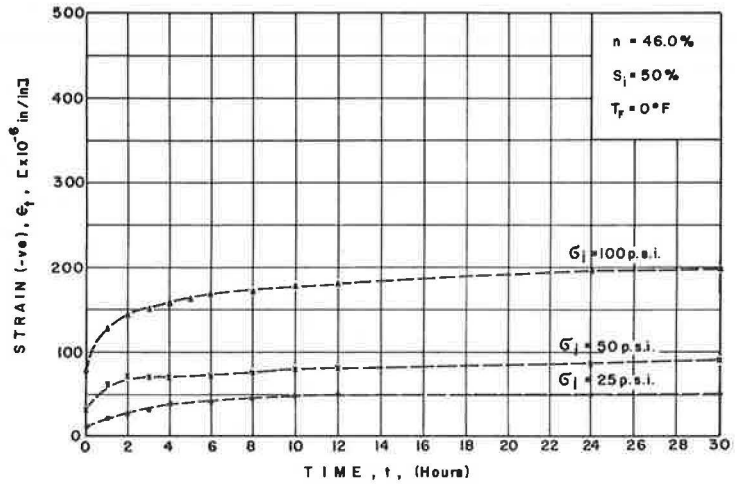


Figure 5. Experimental curves showing strain-time relationship for three different initial pressures under indicated conditions.

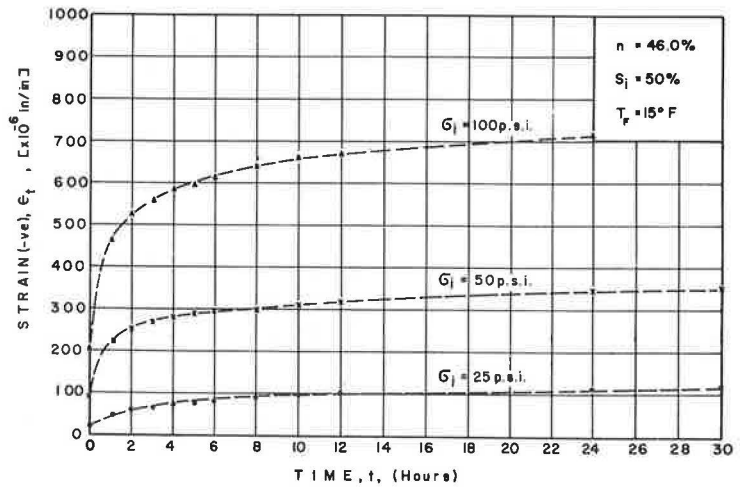


Figure 6. Reduction in lateral stress versus time for different constant temperatures under indicated conditions.

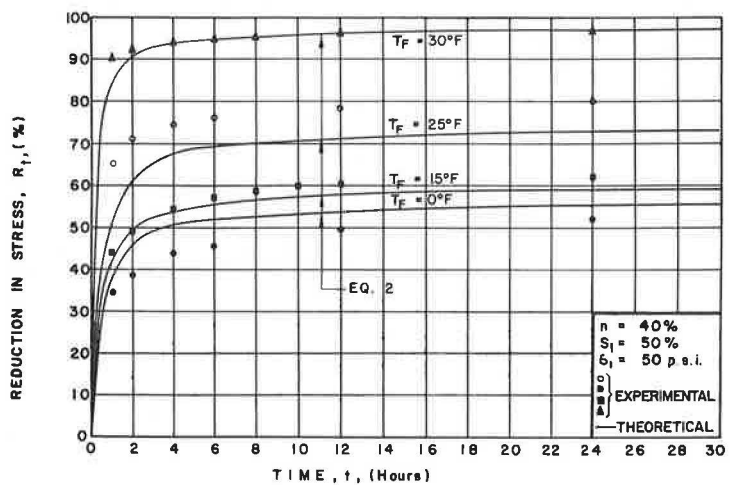


Figure 7. Reduction in lateral stress versus time for different initial pressures under indicated conditions.

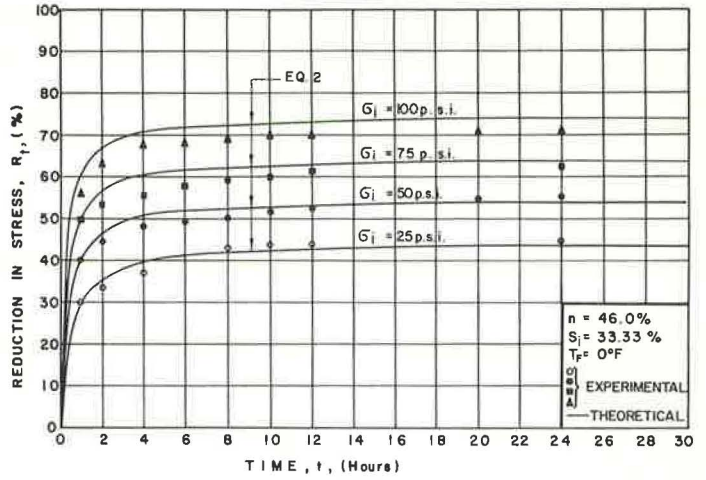


Figure 8. Reduction in lateral stress versus time for different initial porosities under indicated conditions.

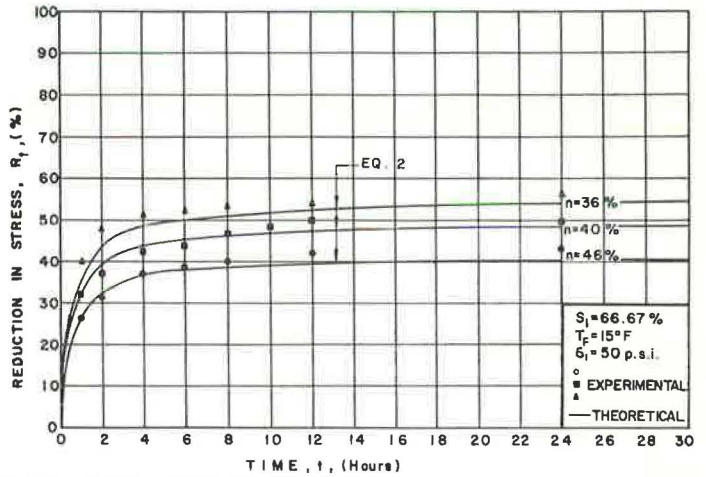
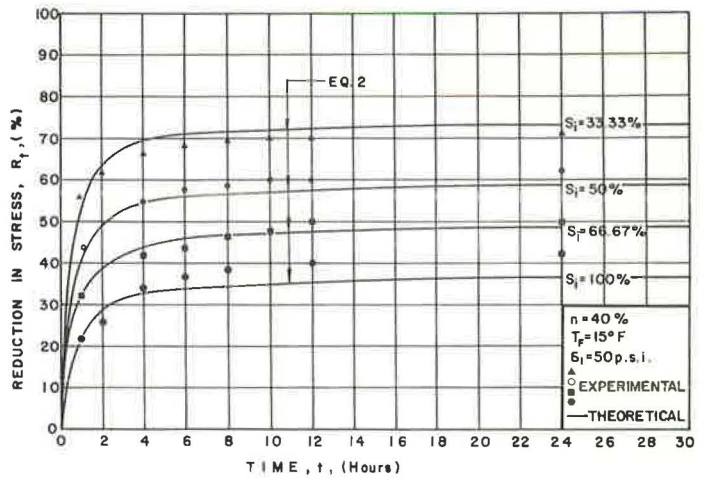


Figure 9. Reduction in lateral stress versus time for different degrees of ice saturation under indicated conditions.



The magnitude of the pressure retained by the frozen soil layer after time t , can be obtained by equating Eqs. 1 and 2 and solving for σ_t , which gives

$$\sigma_t = \sigma_1 - C\sigma_1 \exp(-B/t) \text{ [psi]} \quad (3)$$

where $C = A/100$.

Based on the experimental results obtained, values of reduction factor A and time factor B can be expressed in the form of the empirical equations that were derived by searching systematically for unknown functions by means of an electronic computer. The reduction factor may be computed from the following equation:

$$A = M\sigma_1 + 100 \exp(K) + 0.08 T(46-n) \quad (4)$$

with a maximum value of $A = 100$ percent

where

$$M = 0.245 \log_{10} T + 0.35;$$

$$K = -0.00717 S_1 (\sqrt{T} - 1.15);$$

$$T = 32 - T_f = \text{temperature below freezing point, deg F};$$

$$T_f = \text{temperature of frozen soil, deg F}; \text{ and}$$

$$n = \text{initial soil porosity in percent.}$$

The time factor may be estimated from the relation

$$B = \left[0.46 (S_1)^{0.185} - 0.33 \log_{10} \sigma_1 \right] N \quad (5)$$

where

$$S_1 = \text{degree of ice saturation, percent};$$

$$N = 1 \text{ for } T_f \leq 27 \text{ F}; \text{ and}$$

$$N = T/5 \text{ for } 27 \text{ F} \leq T_f \leq 31 \text{ F.}$$

If, for a given frozen sand layer, the value of n , S_1 , T_f , and σ_1 are known, factors A and B can be easily calculated from the equations already given. If we put the values of A and B into Eq. 2, the percent of reduction in lateral stress can be estimated as a function of time. Equation 3 can be used to calculate the magnitude of reduced pressure retained by a sand-ice layer at any given period of time after application of initial pressure. From Eq. 3, taking t to be very large, the long-term stable lateral stress capacity (σ_{end}) of the sand-ice system can be estimated as follows:

$$\sigma_{\text{end}} = \sigma_1 - C\sigma_1 = (1 - C) \sigma_1 \quad (6)$$

The theoretical curves shown by the dotted lines in Figure 3 and by the solid lines in Figures 6 to 9 were obtained by using Eq. 2 and Eq. 3 respectively. They show good agreement with the points that were obtained experimentally.

The experiments indicated that, 24 hours after the application of initial pressure to the frozen sand layer, there was very little or practically no reduction in the lateral stress taking place in the frozen soil. To be more specific, it should be stated that, on the average, 98 percent of the value of A was reached in the first 24 hours, where A , the reduction factor, defines the maximum long-term reduction in lateral stress for a given frozen sand. Therefore, to demonstrate the effects of porosity, ice saturation, temperature, and initial pressure on the decrease of lateral stress in frozen soil, we took as a basis for comparison the percentage of reduction in stress that took place at the time $t = 24$ hours (R_{24}).

Figure 10 shows the influence of the magnitude of initial pressure σ_1 on the stress reduction R_{24} as a function of T_f , S_1 , and n . Figure 10a indicates the effect of soil temperature on "R₂₄ versus σ_1 " curves, whereas Figures 10b and 10c show the effect of the degree of ice saturation and the initial porosity respectively on the aforementioned curves. Within the experimental range, the relationship between R_{24} and σ_1 is approximately linear; this is true for all combinations of the variables used.

Figure 10. Stress reduction after 24 hours versus initial pressure showing the influence of (a) temperature, (b) degree of ice saturation, and (c) porosity.

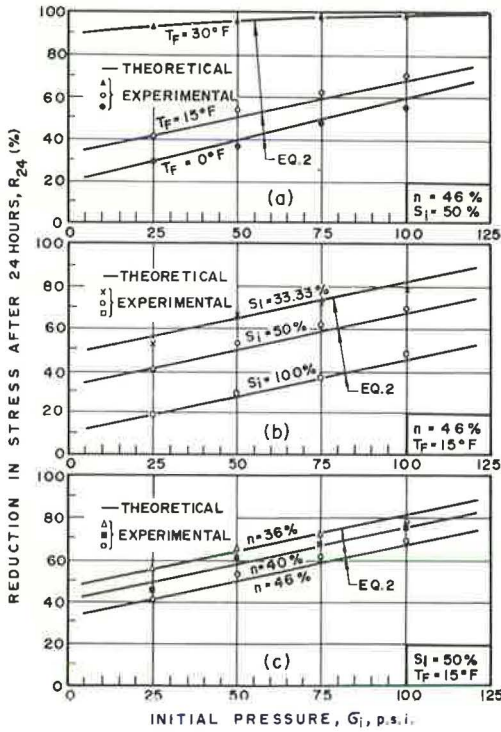


Figure 12. Reduction in stress after 24 hours versus degree of ice saturation showing the influence of (a) initial pressure, (b) temperature, and (c) porosity.

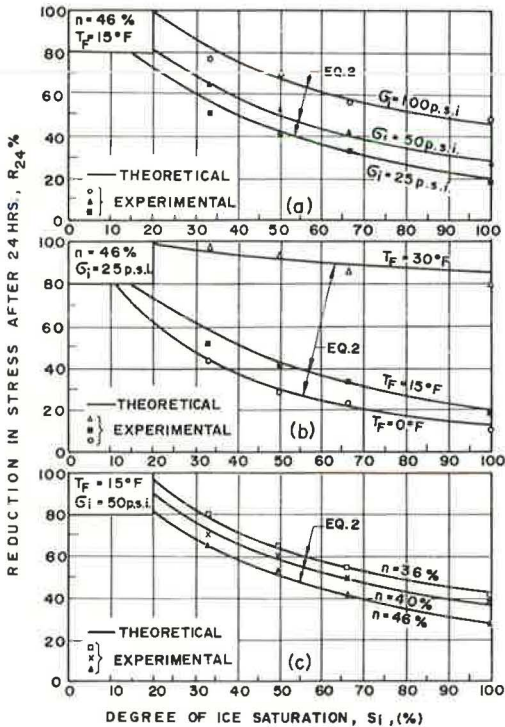


Figure 11. Reduction in stress after 24 hours versus temperature showing the influence of (a) initial pressure, (b) degree of ice saturation, and (c) porosity.

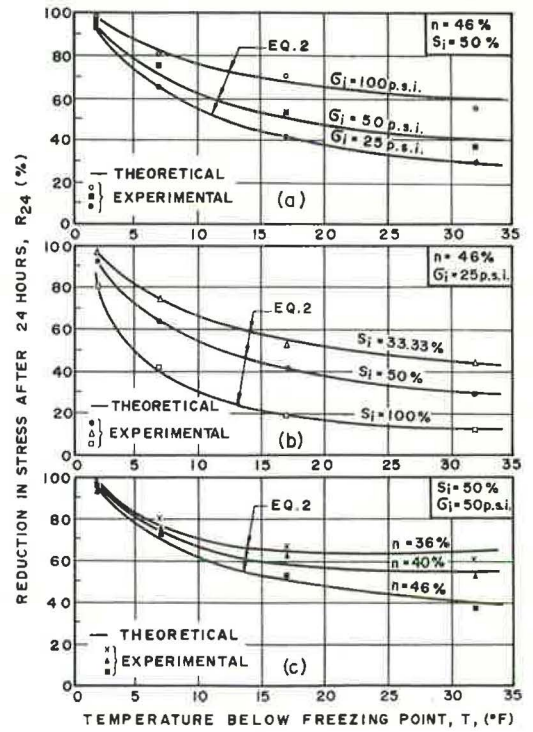


Figure 13. Reduction in stress after 24 hours versus initial porosity showing the influence of (a) initial pressure, (b) temperature, and (c) degree of ice saturation.

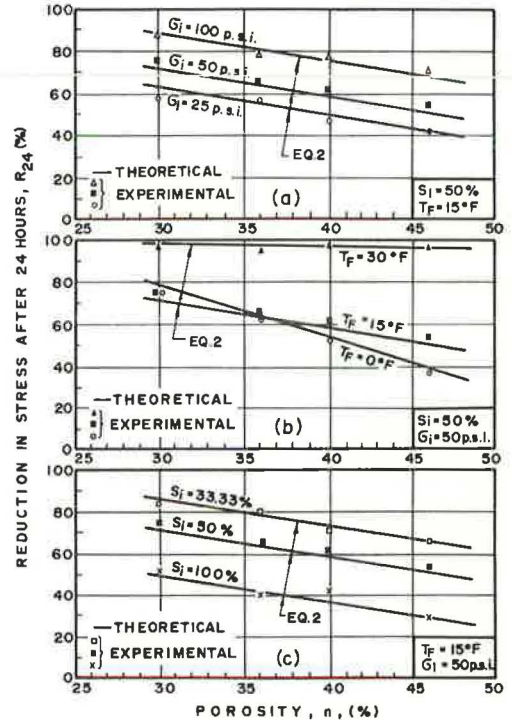


Figure 11 shows the variation of R_{24} with temperature below freezing point T as well as the effects of initial pressure (Fig. 11a), degree of ice saturation (Fig. 11b), and porosity (Fig. 11c) on " R_{24} versus T " curves. All three figures indicate that at low values of T , the rate of reduction in stress is rather rapid, but as value of T increases, the rate becomes smaller.

Figure 12 shows the variation of R_{24} with the degree of ice saturation S_i and also the effects of σ_1 , n , and T_f on the " R_{24} versus S_i " curves. These curves reveal that, in general, the magnitude of R_{24} decreases with an increase in ice saturation. It can be seen that, when low ice saturation occurs simultaneously with high initial pressure or high frozen soil temperature (i. e., 30 F), 100 percent of the long-term reduction in stress can occur even before the elapse of a 24-hour period.

Another general relationship is shown in Figure 13, which indicates that reduction in stress R_{24} is inversely proportional to the initial soil porosity. The aforementioned figure also shows that an increase in initial pressure (Fig. 13a) or a decrease in ice saturation (Fig. 13b) shifts the " R_{24} versus n " curves upward in a parallel manner. An increase in the temperature of the frozen soil (Fig. 13c) moves the curves upward and also tends to decrease their slopes.

The solid lines shown in Figures 10 to 13 are not the lines of best fit for experimentally obtained points but represent Eq. 2 when the value of $t = 24$ hours.

CONCLUSIONS

Lateral stress developed in a confined sand-ice layer due to temporary temperature increase or to application of external pressure decreases with time if, after development of the initial pressure, the temperature of the frozen soil remains relatively constant. A more economical design can be achieved when the reduction behavior of the lateral thrust developed between a frozen soil layer and a retaining structure is known. The empirical equations introduced in this paper may serve as a guide in this case.

In general, the major part of the long-term reduction in lateral stress occurs during the first few hours after application of initial pressure. In fact, more than 85 percent of the total stress reduction occurs during the first 5 hours, and approximately 98 percent occurs during 24 hours.

The magnitude of reduction in lateral stress is highly influenced by the temperature of the sand-ice system. At a temperature of 30 F, the stress reduction R_{24} measured 24 hours after application of the initial pressure to a frozen sand was found to be more than 90 percent of the initial pressure, whereas at 0 F it was approximately half of this value.

Initial soil porosity and degree of ice saturation are also two major factors affecting the stress reduction process. An increase in both will result in a decrease in the value of R_t , seemingly because of the increase in ice content.

Initial pressure applied to a sand-ice system at a constant temperature also shows a significant effect on the magnitude of stress reduction; R_{24} increases almost linearly with the increase in initial pressure.

The influence of the type or grading of sand, in this particular case, seemed to be negligible. However, this conclusion may need further experimental evidence.

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