heave rates in base and subbase layers of less then 3 mm/day.

6. No significant differences in performance relations were noted between the wet coastal areas and the dryer interior areas of Alaska.

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Simulating Frost Action by Using an Instrumented Soil Column

J. INGERSOLL AND R. BERG

The use of an instrumented soil column in tests to develop a mathematical model of the frost-heave process is described. Tensiometers, heat-flow meters, thermocouples, and electrical resistivity gages were installed throughout a soil column filled with Fairbanks silt, Chena Hot Springs silt, or West Lebanon gravel. The column was 100 cm long and about 14 cm in diameter. An open system was used and absorption was monitored during the freezing process. Three freezing tests were conducted on Fairbanks silt, two with a surcharge of 3.45 kPa and one with 34.5 kPa. The water level was held at the 45-cm depth. Eleven tests were conducted using Chena Hot Springs silt. The water level was set at 15-, 50-, or 100-cm depths, and the surcharge was 3.45 or 34.5 kPa. Tests were conducted by using a constant rate of frost penetration, a constant heat-flow rate, or three sequentially lower temperature step changes at the soil surface. Two tests were conducted with West Lebanon gravel. Both included three step changes in the surface temperature and were surcharged with 3.45 kPa. The water levels were at the 15- and 100-cm depths. The soil column has provided critical data for verification of a one-dimensional mathematical model for estimating frost heave. As more soils are tested, this equipment will assist in improving and developing algorithms for the mathematical model and in identifying the most critical parameters that affect frost heave in a given soil—e.g., surcharge, free water level, and hydraulic conductivity. A procedure is also presented for determining the saturated and unsaturated hydraulic conductivity and moisture-retention characteristics of a soil.

In July 1975, the Federal Highway Administration (FHWA), the Federal Aviation Administration (FAA), and the U.S. Army Corps of Engineers initiated a cooperative project to develop a mathematical model of the frost-heaving process. The one-dimensional finite-element computer program resulting from the study was documented (1,2, and Guymon and others elsewhere in this Record).

The results reported here were obtained from an instrumented soil column that was originally constructed to provide data for developing an algorithm describing the effects of overburden (or surcharge) on soil moisture stress. The column was designed so that the water table could also be varied. We have also used data from the soil column to aid in verifying the mathematical model of frost heave (2 and Guymon and others elsewhere in this Record) and have developed a more refined design that will be used with a dual gamma system. With the dual gamma system, we will have the capability of monitoring changes in moisture content and density nondestructively. The dual gamma system, in conjunction with the other instrumentation in the soil column, will permit developing the moisture-characteristic curve and the relation between moisture content (or matrix suction) and hydraulic conductivity for the soil within the column. The soil column will also be used for other portions of the cooperative study—e.g., to assist in developing the thaw-weakening algorithm and in applying the mathematical model to layered pavement systems.

TEST APPARATUS

The soil was molded within a 100-cm-long circular cylinder that had a diameter of about 14 cm. The inside of the upper 15 cm of the cylinder tapered slightly and was lined with Teflon tape to minimize sidewall resistance to heaving. The top portion of the cylinder was detachable from the lower portion (see Figure 1).

Thermocouples were inserted through the cylinder walls and into the soil at intervals of 1 cm in the upper portion and intervals of 2.5-10 cm in the lower portion. Tensiometers were placed at 1.5- to 20-cm intervals depending on the test and the location in the column. In early tests, the uppermost tensiometer was 18 cm below the top of the column whereas later tests had tensiometers at the 5- and 10-cm depths. Additional thermocouples were installed adjacent to the 5- and 10-cm tensiometers.

A linear motion potentiometer (LMP) and a dial gage were used to measure vertical movement of the sample surface. Absorption of water by the soil was visually monitored by using a graduated constant-head reservoir. The reservoir was also used to control the free-water level in the sample; i.e., the tests were conducted in the open-system mode. In several tests, electrical resistivity gages were placed within the upper 15 cm to locate the solidly frozen soil. Surcharge on the soil was created by placing lead weights on a pedestal attached to the surface plate. A heat-flow meter was recessed into the bottom of the surface plate contacting the soil. Data from the thermocouples, the LMP, and the heat-flow meter were monitored hourly by a digital data-collection system.

Electrical pressure transducers were attached to most tensiometers to allow monitoring by the data-collection system and to minimize the amount of fluid movement to and from the soil. Negative pressure dial gages were added to the tensiometers without transducers. The tensiometers with dial gages were usually placed near the bottom of the column and were read daily. Tensiometers within the zone to be frozen were filled with a solution of 30 percent ethylene glycol and water.

Copper electrical resistivity probes were used in most of the tests. These probes were initially spaced 2 cm apart and later at 1-cm intervals from the surface of the column to the 16-cm depth. These devices were used to delineate the solidly frozen zone. They were especially of value when portions of the soil became isothermal. A higher resistance reading indicated frozen soil, whereas a 0°C thermocouple reading does not necessarily indicate the boundary between frozen and unfrozen soil. Observations on the resistivity probes were made manually once a day by using an oscillator and a digital multimeter (see Figure 2). The resistance probes were later omitted because they probably retarded heaving of the soil. We hope they can be used again when a probe with "breakaway" leads can be developed.

Loose cork insulation was placed around the upper 17 cm of the column for the three tests by using Fairbanks silt and to the 50-cm depth for the remainder. Only the top surface was exposed to subfreezing temperatures, which allowed one-dimensional freezing. This was accomplished in early tests by cold-air circulation and later by use of a refrigerated surface plate. The ambient temperature of the room was maintained at +4.5°C.

SOILS TESTED

Three soils have been tested: Fairbanks silt (FBKS), Chena Hot Springs silt (CHSS), and West Lebanon gravel (WLEBG). FBKS and CHSS are quite similar soils, as shown in Figure 3. Both are nonplastic loess deposits that were obtained from the Fairbanks, Alaska, area. WLEBG is a bank-run gravel (Figure 3) obtained in West Lebanon, New Hampshire.

Three tests were run on FBKS, 11 on CHSS, and 2, to date, on WLEBG. For each series, the 85.0-cm-long base section was first compacted with soil at slightly above the optimum water content by using a uniform compactive effort for each of several layers. The molded density was then determined and was used as a criterion in the molding of subsequent top segments.

Figure 1. Soil column.

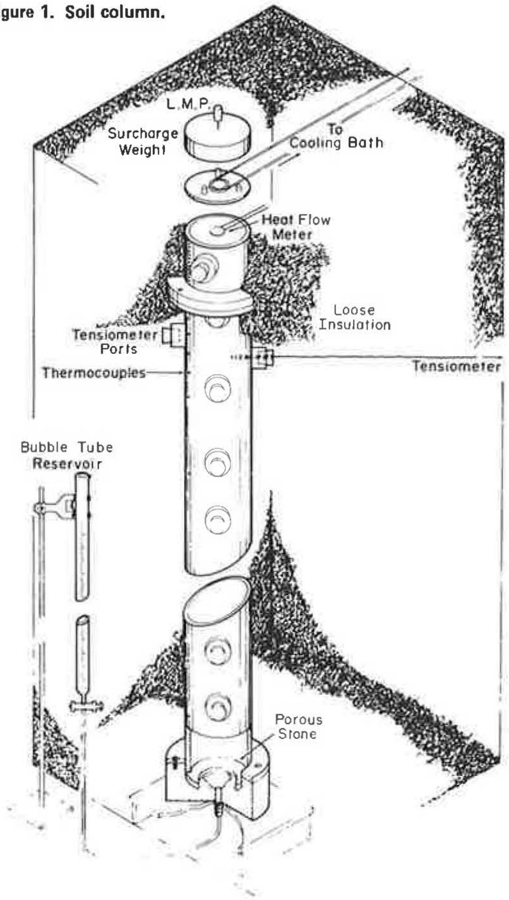
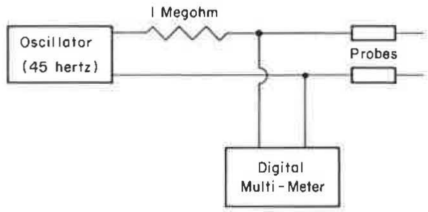


Figure 2. AC resistance probe.



At the conclusion of each test, the upper 15-cm section of the cylinder, which contained the frozen soil, was removed. The frozen soil was ejected from the cylinder, split vertically to expose ice lenses for photographs, and cut horizontally into sections about 2.5 cm thick. Final moisture distribution and density were measured by using these sections (see Figures 4 and 5). Density was measured by coating the sections with ice and displacing their volume in iso-octane maintained at -5°C. The upper cylinder was then compacted with new soil and placed on the column.

Prior to freezing, the water source was adjusted to the level desired for the free-water table. Water absorption and moisture tension were monitored until equilibrium was reached. Due to the relatively high hydraulic resistance of the soil within the column, the free-water level in the soil column usually dropped dramatically during most of the FBKS and CHSS tests. Recently, we have periodically adjusted the height of the water reservoir to maintain the water table at the desired location within the soil column. When freezing was initiated, a rapid drop in temperature was applied to the surface of the soil until nucleation was apparent. The temperature was then raised to the desired subfreezing temperature. This procedure was necessary to stop supercooling to a significant depth before nucleation. Supercooling made it very difficult to control frost penetration in the early stage of the tests.

Three methods of freezing the soil were used: (a) a nearly constant rate of frost penetration, (b) a nearly constant rate of heat flux from the surface, and (c) three step changes at the surface. All three methods were attained by adjusting the surface temperatures, and each of the step changes in method 3 was maintained until frost penetration ceased.

Figure 3. Grain-size distribution of the three test soils.

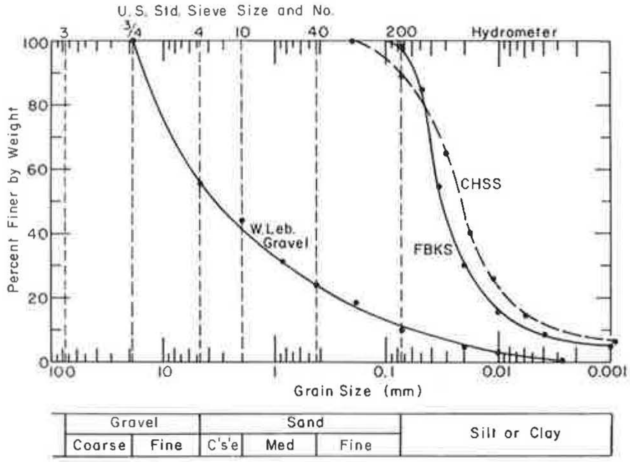


Figure 4. Final moisture and density distribution for CHSS test 11.

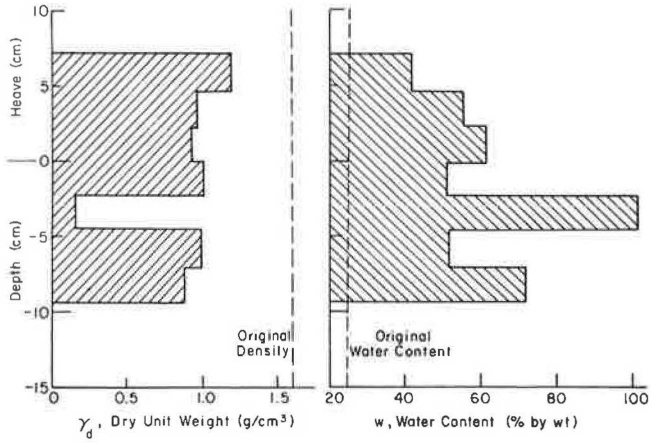
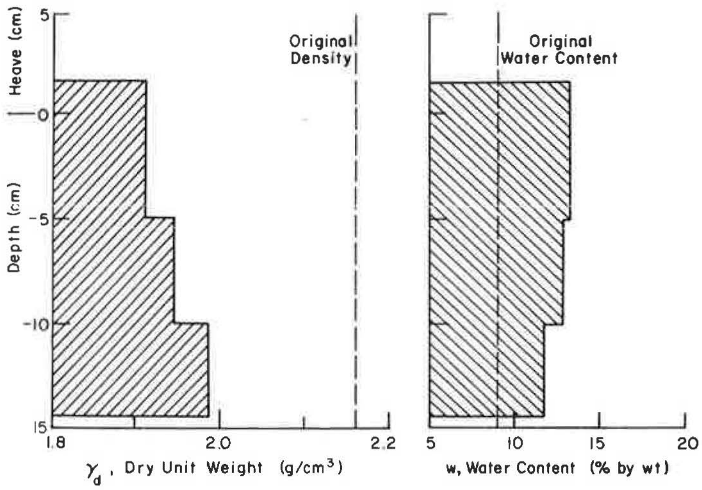


Figure 5. Final moisture and density distribution for WLEBG test 1.



SOIL MOISTURE PROPERTIES

A method for coincidentally determining the moisture-retention characteristics, saturated hydraulic conductivity, and unsaturated hydraulic conductivity was developed by combining a constant-head permeameter with a pressure extractor (see Figure 6). The procedure and equipment make it possible to determine these three parameters simultaneously by using the same sample. The test method is a modified version of the unsaturated hydraulic conductivity test procedure described by Klute (3). The technique was verified by comparing our results with those obtained by using a method developed by Green and Corey (4). The procedure is described in greater detail by Ingersoll (5).

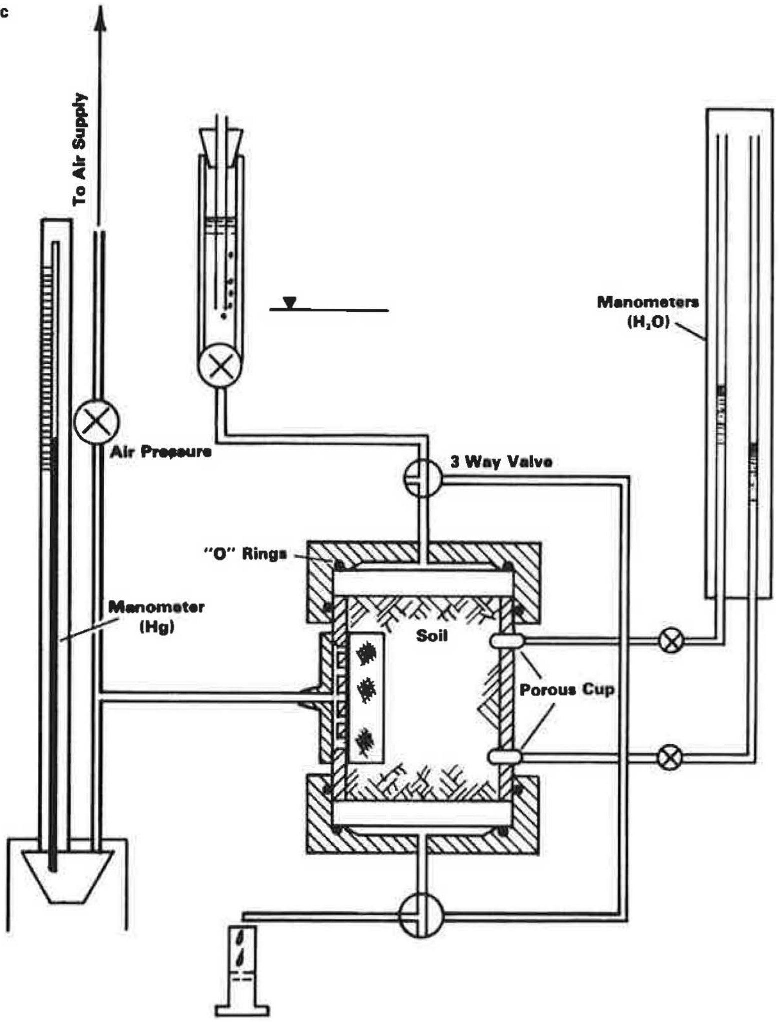
The basic equipment includes a clear plastic cylinder for the permeameter cell, the top and bottom end caps of which contain porous stones that require pressures in excess of 100 kPa before the air-entry value is reached. (The air pressure necessary to force water from the pores of the stones is termed the air-entry value.) Water will flow through the stones readily, however. The end caps were designed to contain the maximum air pressure administered during the test.

Two porous ceramic cups, each connected to a water-filled manometer, were inserted into the soil through a hole in the cylinder wall and were epoxied into place. The cups also had air-entry values in excess of 100 kPa. The manometers were used to measure the head loss between them.

Each end cap had a three-way valve connection. A plastic tube was attached between the valves to allow water to bypass the soil during the extraction stages of each test. A constant-head reservoir (bubble tube) was connected to the top valve and an outflow tube to the bottom valve. This completed the permeameter setup.

The end caps purchased were base plates for large Tempe cells used in moisture-retention testing. The end caps as well as the porous ceramic stones and cups are available from the Soilmoisture Equipment Corporation in Santa Barbara, California. The remainder of the apparatus is normal laboratory equipment.

Figure 6. Pressure-cell permeameter for determining hydraulic conductivity and moisture-retention characteristics.



The cell was a clear plastic cylinder about 10 cm long with an inside diameter of 7.6 cm and an outside diameter of 8.9 cm. Air pressure was applied through several small holes drilled through the cylinder wall. A fine mesh (no. 200) screen was placed between the soil and the cylinder wall at the hole location. Grooves were etched on the outer cylinder surface between the holes to evenly disperse the air. The holes were then covered by a cylinder segment that had the same inside diameter as the outside diameter of the cell. An air fitting was attached to this segment, and the outer segment was epoxied to the cell wall.

HYDRAULIC CONDUCTIVITY TEST PROCEDURE

With the molded soil in place, a saturated constant-head permeability test is run. The three-way valves are turned to allow water to flow through the soil and out the bottom valve into a receiving flask. Flow is continued until input, output, and manometers reach equilibrium. Permeability (K) is calculated by using the following formula:

$$K = QL/dAt \tag{1}$$

where

Q = quantity of outflow (cm^3),
L = distance between porous cups (cm),
d = difference of manometer elevations (cm),
A = area of sample (cm^2), and
t = time (desired unit).

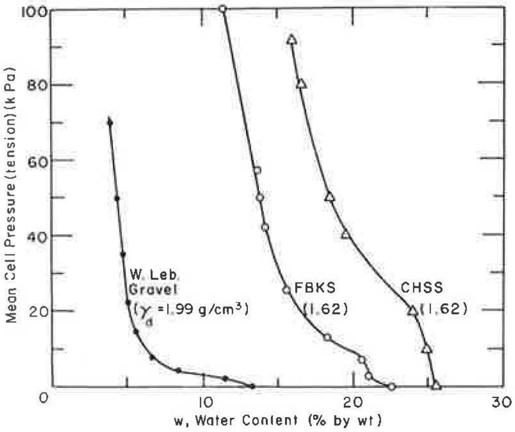
It is important to keep a record of all water going into the sample and coming out of the sample during the test. The water collected during subsequent pressure increments is used to calculate moisture retention.

When the saturated hydraulic conductivity test (permeability test) has been completed, the top and bottom three-way valves are turned to allow water to flow out through the top and bottom plates. The system is then in the extraction mode, and the first air pressure increment--say, 3 kPa--is applied. When equilibrium has been reached, the amount of extracted water is recorded.

The second hydraulic conductivity test is then run with the soil remaining under pressure. At this point, water from the larger pores has been displaced by air at the applied pressure so that the largest passages are unavailable for moisture movement and moisture passes through the sample at a reduced rate. The hydraulic conductivity is calculated as before, and the procedure is repeated for any number of pressure increments up to approximately 80 kPa. Hydraulic conductivity values can be expected to drop four or five orders of magnitude by this point, and the test apparatus would require modification for lower flow rates.

When the final hydraulic conductivity increment has been completed, the soil sample is removed and

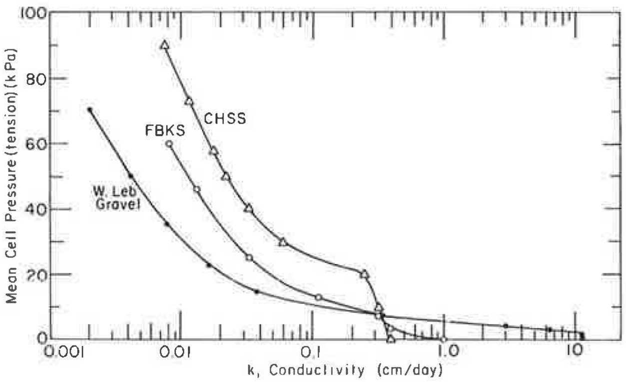
Figure 7. Moisture-retention characteristics of test soils.



its moisture content is determined. The volume of water collected from the preceding extraction points is then incrementally added to the final volume to develop the moisture-retention-characteristic curve (see Figure 7).

It is generally accepted that pressure against the soil surface produces the same effect as tension or suction applied through the soil base. In view of this, a plot can be constructed by using soil tension versus hydraulic conductivity. By using the water-extraction data, another plot can be made that shows degree of saturation versus hydraulic conductivity, if so desired. A plot that shows relations between tension and hydraulic conductivity for the three soils discussed here is included (see Figure 8).

Figure 8. Hydraulic conductivity of test soils.



Since most soils exhibit hysteresis between the desorption (drying) curve and adsorption (wetting) curve, it may be of value to rewet the soil by using a reversed procedure. This technique has worked satisfactorily for several soils tested.

Table 1. Summary data on sample preparation and testing.

Test	Dry Unit Weight		Porosity (in)	Water Content (percent by weight)		Water Uptake (cm^3)		Surcharge (kPa)	Free Water from Surface (cm)	Method of Freezing	Max Freeze Depth (cm)	Freeze Rate (cm/day)		Heave (cm)	
	Pounds per Cubic Foot	Grains per Cubic Centimeter		Original	Avg Final	Total	Max Daily					Avg	Max	Expected from Water-Uptake Freezing	Total
FBKS															
1	95	1.52	44.3	21.3	38.4	717	29.5	3.45	45	CP	15.0	0.35	1.1	4.6	5.9
2	95	1.52	44.3	20.0	30.0	514	26.0	34.50	45	CP	15.0	0.80	1.8	3.3	3.2
3	95	1.52	44.3	21.0	27.4	410	28.0	3.45	45	CP	15.0	1.0	2.0	2.7	2.2
CHSS															
1	99	1.58	43.5	20.1	26.5	120	36.8	3.45	100	CP	Sample went isothermal			0.8	1.1
2	95	1.52	45.7	22.0	30.2	315	48.0	3.45	100	CP	18.0	1.0	2.1	2.0	1.9
3	98	1.57	43.9	19.6	26.2	261	45.5	3.45	100	CP	21.0[a]	2.1	2.7	1.7	1.8
4	97	1.56	44.2	20.1	32.0	426	61.0	3.45	50	CP	19.5[b]	1.7	1.7	2.8	2.2[c]
5	100	1.60	42.9	19.0	27.3	268	37.0	34.50	100	CP	15.0	1.2	2.3	1.7	1.65
6	99	1.58	43.6	19.5	29.2	232	33.0	34.50	50	CP	16.0	2.0	2.5	1.5	0.99
7	101	1.62	42.2	20.0	28.6	498[d]	60.0[d]	3.45	15	CHF	16.5	1.6	2.7	3.2[d]	2.3
8	104	1.67	40.4	16.4	35.7	610	59.8	34.50	15	CHF	15.0	1.25	1.7	3.9	3.9
9	102	1.63	41.6	19.3	35.3	529	94.0	3.45	15	CHF	15.5	2.2	4.0	3.4	4.1[e]
10	105	1.68	40.0	17.6	NA	80	58.0	3.45	15	CHF	14.0	8.5	12.0	0.5	0.5
11															
Step 1						241	52.0				3.8	1.9	2.5	1.6	1.6
Step 2						407	56.0				7.0	0.75	1.5	2.6	2.5
Step 3						310	51.0				10.5	0.56	1.0	2.0	2.5
Entire test	100	1.60	42.9	24.0	62.0	958		3.45	15	SC					
WLEBG														6.2	6.6
1															
Step 1						168	41				7.5	3.5	5.0	1.1	0.8
Step 2						95	24				12.0	1.0	1.8	0.6	0.4
Step 3						114	16				14.5	0.3	2.2	0.8	0.5
Entire test	136	2.16	21.3	9.0	12.8	377		3.45	15	SC				2.5	1.7
2															
Step 1						39	12				5.5	1.4	5.5	0.3	0.6
Step 2						85	19				11.0	0.9	4.1	0.6	0.5
Step 3						95	13				13.0	0.3	0.5	0.7	0.4
Entire test	131	2.10	23.6	9.0	13.9	219		3.45	100	SC				1.6	1.5

Notes: CP = constant frost penetration; CHF = constant heat flux; SC = step changes.
Standard Proctor moisture density as given below:

Material	Max Density		Optimum Water Content (%)
	Pounds per Cubic Foot	Grams per Cubic Centimeter	
FBKS	106	1.70	11
CHSS	105	1.68	20
WLEBG	135	2.16	7

[a] Irregular freeze depth: 16-23 cm.
[b] Irregular freeze depth: 16.5-19.3 cm.
[c] Irregular heave: 2.1-3.1 cm.
[d] Base leaked water at start and end of test.
[e] Average heave at end of test.

SOIL-COLUMN RESULTS

Table 1 gives data on sample preparation and procedures for all tests conducted on the three soils. Also listed for comparison are selected test results, including maximum frost heave, average surface heat flow, frost penetration, and water uptake. Normally, data were recorded hourly on perforated paper tape and stored for future use. Once each day, all data were manually recorded and plotted to follow test progress (see Figure 9).

The next-to-last column in Table 1 gives the "ice segregation ratio". This is a relation of the change in height to total frost penetration:

$$I_s = h/(x + h) \qquad (2)$$

where

I_s = ice segregation ratio,
h = frost heave, and
x = frost depth.

The first three tests were on Fairbanks silt. The results were quite satisfactory; however, tension measurements and temperature control were quite erratic, since the procedures were still in the development stage. In all three tests, the free-water level was initially at the 45-cm depth. In FBKS tests 1 and 3 a 3.45-kPa surcharge was used, and in FBKS test 2 a 34.5-kPa surcharge was used. Tension values were not measured within the freezing zone in tests 1 and 2 because the top tensiometer was at the 18-cm depth. The top 15 cm of the column was insulated, whereas the bottom 85 cm was exposed to +4.5°C ambient air. There was no measurement of heat flow during these tests. All three tests were frozen by circulating cold air over the exposed top surface of the sample.

The next test series was on Chena Hot Springs silt. The samples in the first six tests (CHSS 1-6) were frozen by circulating cold air at the surface, and those in CHSS tests 7-11 were frozen by circulating a cold liquid through a heat exchanger on the sample surface. This latter method gave us much better control for both frost penetration and vertical heat flow. All CHSS tests were insulated to the 50-cm depth, and the bottom 50 cm was exposed to +4.5°C air. The adjustable free-water reservoir was set at three different elevations: 15, 50, and 100 cm from the top surface. Eight of the tests used surcharges of 3.45 kPa, and three used 34.5 kPa.

In CHSS tests 1-6, we attempted to control the rate of frost penetration, while in CHSS tests 7-9 heat flow at the surface was controlled at approximately 0.02 cal/cm²•min. In CHSS test 10, the controlled heat flow was doubled, and the entire top 15 cm froze in two days with relatively little heave.

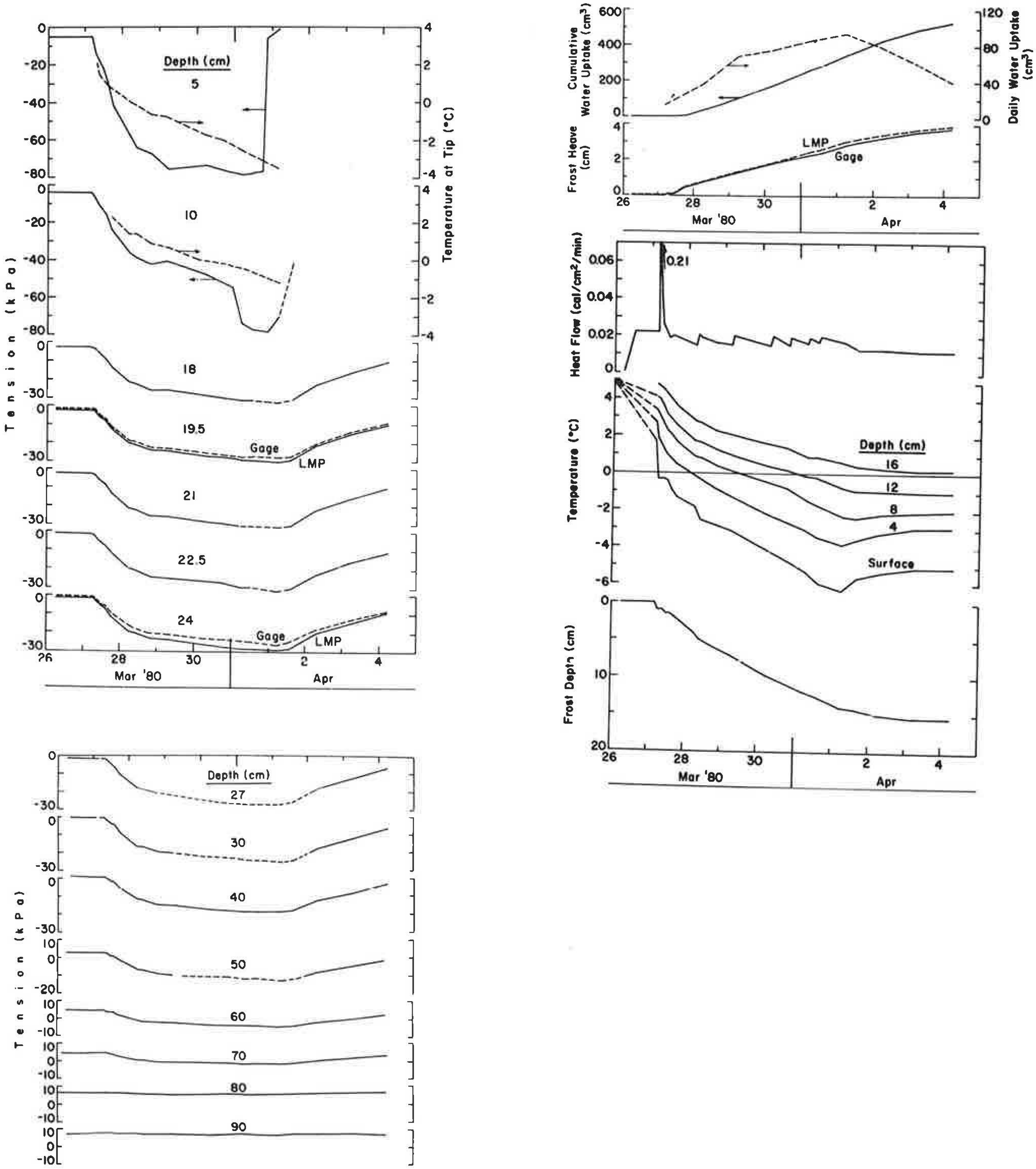
Temperature step changes were applied to CHSS test 11. The initial surface temperature applied was -2.0°C; this was followed by steps to -3.5°C and -5.5°C. Each step change in temperature was main-

		Max Tension in Freeze Zone (kPa)	Depth of Measurement of Max Tension (cm)	Heat Flux ($cal/cm^2/min$)		Avg Ice Segregation Ratio at End of Frost Penetration	
Total as Percent of Frost Depth	Max Daily			Avg	Max After Initial Stage		Remarks on Test
39.3	0.42	NA	NA	NA	NA	0.282	Only top 15 cm insulated; did not freeze through tensiometers
21.3	0.27	NA	NA	NA	NA	0.176	Only top 15 cm insulated; did not freeze through tensiometers
14.3	0.90	62.0	15	NA	NA	0.128	Only top 15 cm insulated
–	1.0	68.0	18	NA	NA	NA	Poor test; cold trapped in top of cabinet; redesigned
10.5	0.5	76.0	10	0.012	0.019	0.005	First test using resistance gages
8.6	0.56	69.0	10	0.015	0.028	0.079	Total heave measured at end of test
11.3	0.48	76.0,47.0	10,14	0.010	0.018	0.101	
11.0	0.32	83.3	10	0.026	0.036	0.099	Total heave measured at end of test
6.2	0.24	84.0,42.0	10,13	0.020	0.030	0.058	Total heave measured at end of test
13.9	0.80	80.0,44.0	10,13	0.020	0.024	0.122	Total heave measured at end of test; maximum uptake okay
26.0	0.50	80.2,57.0	5,10	0.022	0.024	0.206	No resistance gages; total heave measured at end of test (uneven)
26.5	0.60	78.0,78.0	5,10	0.017	0.020	0.209	No resistance gages; replicate test 7
3.6	0.90	87.0,82.0	5,10	0.040	0.048	0.034	No final density on water table; results on first 48 h only
42.0	0.6	NA		0.006	0.028	0.296	Temperature of surface plate = -2.0°C
78.1	0.8	62.5	5	0.010	0.022	0.561	Temperature of surface plate = -1.5°C
100.0	0.6	NA		0.013	0.026	0.462	Temperature of surface plate = -5.5°C
62.9						0.403	
							Entire column insulated
10.6	0.28	NA		0.020	0.033	0.096	Temperature of surface plate = -2.0°C
8.9	0.08	79.5		0.020	0.026	0.082	Temperature of surface plate = -3.0°C
20.0	0.04	63.0		0.020	0.022	0.170	Temperature of surface plate = -3.5°C
11.7						0.105	
10.1	0.13	NA		0.025	0.030	0.098	Temperature of surface plate = -2.0°C
9.0	0.11	70.0	5	0.022	0.028	0.083	Temperature of surface plate = -3.0°C
20.0	0.03	61.5		0.021	0.021	0.170	Temperature of surface plate = -3.5°C
11.5							

tained until frost penetration essentially stopped and the thermal gradient stabilized. At these points, heaving usually continued as a large, essentially pure ice lens developed. The lenses that formed during each temperature step were very small initially and became progressively larger until the last large lens formed. The test was terminated, due to equipment malfunction, before the third large lens developed fully. Table 1 includes data from each step as well as from the entire test.

WLEBG test 1 was also a step-change test. A 3.45-kPa surcharge was applied to the surface, and the external water level was maintained at the 15-cm depth. WLEBG test 2 was identical to test 1 except that the external water level was maintained at the 100-cm depth. The results of these two tests showed that a variation of the free-water depth does not significantly affect the total amount of heave or the heave rate for this soil. The pressure (tension) gradients within the two samples were nearly identical beneath the freezing zone. The primary difference was that pressures in test 2 were about

Figure 9. Plots of CHSS test 9 formulated by using data collected from daily readings.



0.8 kPa lower (more negative). This result was unexpected, since the hydraulic conductivity decreases about two orders of magnitude between saturation and a tension of about 0.8 kPa. We are evaluating the reasons for this behavior.

A third test, currently in progress, is exhibiting substantially less heave because it is being subjected to a 34.5-kPa surcharge. The external water level in test 3 is at the 15-cm depth.

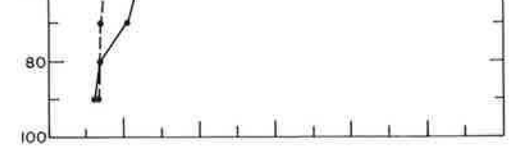
A striking difference between the gravel tests and the previous silt tests was the low soil-tension measurements ahead of the freezing zone in the gravel. Values of soil moisture tension in the freezing and frozen zones are given below:

Temperature (°C)	Tension at 5-cm Depth (kPa)	
	CHSS Test 7	WLEBG Test 1
0.0	60	2.0
-0.1	64	2.0
-0.2	65	2.0
-0.3	66	2.0
-0.4	66	2.0
-0.5	67	6.0
-0.6	69	19.0
-0.7	73	56.0
-0.8	75	74.0
-0.9	76	76.0
-1.0	76	-

Tensions within the freezing zone and within the frozen material in all three gravel tests increased to approximately 76 kPa but did not increase significantly until the temperature at the tensiometer reached about -0.4°C. Tensions in the unfrozen soil immediately below the 0°C isotherm showed only a slight increase in tension in the gravel (see Figure 10). In the CHSS soil, the tension gradient directly in front of the 0°C isotherm was much greater. The more gradual increase in tension at subfreezing temperatures for the CHSS soil, indicated in the table above, is due to drying in front of the freezing zone. Had the suction at the 15-cm depth in the CHSS soil been maintained at about zero as in the WLEBG soil, the behavior might have been more similar and undoubtedly more heaving would have been observed. It is not clear if this is peculiar to a soil of low frost susceptibility, but further testing is planned in this regard. A comparison of the tension gradients of CHSS and WLEBG at a rapid rate of frost penetration is shown in Figure 10.

At the conclusion of each test series, the soil contained in the 85-cm-long base section was removed and density and water content were determined. The density remained unchanged from the original molded density, and water contents averaged about 80 percent saturation. Only the FBKS and CHSS bases have been analyzed to date; testing of WLEBG is continuing.

Figure 10. Soil-tension profile comparing CHSS and WLEBG during frost penetration.



CONCLUSIONS

In all tests, upon the initial nucleation of ice, the soil moisture stress immediately became more negative throughout the length of the column. The tensions in the unfrozen zone generally reached their highest values after two to four days of freezing and then decreased slowly but steadily until the particular test or temperature step was terminated. This was much more apparent in the highly frost-susceptible silt than in the less-frost-susceptible gravel. When the 0°C isotherm passed a tensiometer location, the tension increased rapidly and remained at a high value until termination of the test or thawing of the soil. A very steep pressure (tension) gradient occurred behind the freezing front--i.e., in the freezing zone--in all three soils. Water was transported through the WLEBG material rapidly enough to preclude significantly increased soil moisture tensions in advance of the 0°C isotherm, but the two silty soils did exhibit increases in tension in front of the 0°C isotherm.

Data from the soil subjected to freezing temperatures lend further credence to the existence of a freezing zone, as proposed by Miller (6), rather than a "frost line", as commonly pictured.

Because the gradient of soil moisture tension appears to be very steep within the freezing zone, we were unable to completely achieve our original objective of developing an improved mathematical algorithm to include the effects of surcharge. The data do not invalidate our present algorithm, however. In it we simply compute, from the moisture-characteristic curve (or an extrapolation of it), the soil moisture stress acting at the freezing front and deduct the overburden (surcharge) stress to obtain the effective stress on the pore water.

The depth of the free-water level apparently has a greater effect than surcharge on the heave potential of the two silts. The reverse appears to occur in the gravel.

These data also suggest that, since the pressure gradient is extremely steep in the freezing zone, we may need to use a very small mesh spacing in numerical models to adequately represent phenomena that occur in this region. To more closely represent the actual freezing situation, perhaps we should also include freezing zones in our models rather than freezing fronts. These concepts will be further evaluated by using the soil column on a wider variety of soils, especially those for which we also have field observations of frost heave and frost penetration. Using the dual gamma system in conjunction with the soil column should greatly aid in refining our current algorithms. That system will also help us to develop the "thaw-weakening algorithm". We plan to use the dual gamma system in tests on at least three of the soils from a test site in Winchendon, Massachusetts, and also on two subgrade soils from the Albany County Airport in Albany, New York.

Although the instrumented soil column was initially intended for verification of a mathematical algorithm of surcharge effects, some other applications are to be investigated:

1. The effect of layered soil types under pavements and airfields,
2. Thaw weakening and recovery of soils under pavements,

3. The influence of water-table location with respect to the freezing zone,
4. The influence of freezing rate on frost heave and thaw weakening, and
5. The influence of a partially thawing or frozen layer on total heave and thaw weakening (although they are not part of the present study, we could also study frost buildup behind retaining walls or exposed culverts and/or frost action caused by below-freezing pipelines buried in thawed soils).

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Comparative Evaluation of Frost-Susceptibility Tests

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Methods of determining the frost susceptibility of soils are identified and presented. More than 100 criteria were found; the most common were based on particle-size characteristics. These particle-size criteria are frequently augmented by information such as grain-size distribution, uniformity coefficients, and Atterberg limits. Other types of information, such as permeability, mineralogy, and soil classification, have also been required. More complex methods that require tests based on pore-size distribution, moisture tension, hydraulic conductivity, heave stress, and frost heave have also been proposed. However, none have proved to be a universal test for determining the frost susceptibility of soils. Based on this survey, four methods are proposed for further study: the U.S. Army Corps of Engineers Frost-Susceptibility Classification System, the moisture-tension/hydraulic-conductivity test, a new frost-heave test, and the California bearing ratio after-thaw test.

The search for a reliable method to evaluate the frost susceptibility of soil has gone on for at least the past 50 years. More than 100 methods have been proposed since Taber's treatise on the mechanism of ice segregation in soils (1) and Casagrande's conclusion that "under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in nonuniform soils containing more than three percent of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm" (2). Even though there has been almost continuous research on frost heave since then, there has been little success in developing comprehensive criteria that more successfully predict the frost susceptibility of soil than does the Casagrande criterion. This has occurred in spite of the probability that Casagrande never intended that his grain-size criterion be universally applied.

Evidence of the lack of success in developing a comprehensive method to determine the frost susceptibility of soils is the plethora of methods reported in the literature. Obviously, each new method set forth has been developed because others have proved to be unsatisfactory for one reason or another. In many cases, the new criteria have been successful for specific but limited purposes. In most cases, however, there is little evidence as to the degree of success achieved.

This paper attempts to identify index tests for determining the frost susceptibility of soils and to select a few of these for further study. The period of time surveyed reaches back to the early work of Taber (1), Casagrande (2), Beskow (3), and Ducker (4) and includes methods reported through 1980. Although an attempt was made to identify all index test methods developed or reported during this period, some may have been missed. The most serious omissions may be from the Asian and Eastern European nations because of the difficulty in gaining access to their literature.

TYPES OF FROST-SUSCEPTIBILITY TESTS

This survey has identified five fundamentally different categories of tests for determining the frost susceptibility of soils and granular base materials. They include tests based on (a) particle-size characteristics, (b) pore-size characteristics, (c) soil-water interaction, (d) soil-water-ice interaction, and (e) frost heave. A brief description of each of the test methods follows.

Particle-Size Characteristics

Particle-size classification methods are by far the most extensively used tests for determining the frost susceptibility of soils. The most basic of these tests include only particle size as the determining factor. The most widely used, the Casagrande criterion (2), requires the determination of the percentage finer than 0.02 mm and the uniformity coefficient.

More complex classification systems, such as the U.S. Army Corps of Engineers criterion (5), are related to the Unified Soil Classification system (6),