concrete subbases were smaller than those under pavements with granular subbases of identical thickness. The data indicate that, depending on subgrade properties, a 100- to 150-mm (3.94- to 5.91-in) thick expanded polystyrene concrete subbase had the same effect on subgrade stress as a 250-mm (9.84-in) thick granular subbase.

SUMMARY AND CONCLUSIONS

Tests were conducted on outdoor slabs to evaluate the performance of pavement sections that incorporate expanded polystyrene concrete subbases. An analytical investigation was made to determine the contribution of expanded polystyrene subbases to subgrade frost protection and pavement load-carrying capacity.

The following are the principal conclusions that were made based on this investigation:

1. Expanded polystyrene concrete subbases effectively reduce or eliminate frost penetration into subgrades under rigid and flexible pavements. A 10-mm (0.39-in) expanded polystyrene concrete subbase is as effective as a 30-mm (1.18-in) granular subbase.
2. Expanded polystyrene concrete subbases reduce pavement deflections and stresses and thereby increase pavement load-carrying capacity. In addition, the effect of an expanded polystyrene concrete subbase on pavement stresses is similar to that of a relatively thicker granular subbase.

From the results of this investigation, it is evident that expanded polystyrene concrete subbases have the potential of reducing pavement damage caused by frost action in those areas where severe winter conditions prevail.

ACKNOWLEDGMENTS

This work was initiated and sponsored jointly by the Portland Cement Association and BASF Wyandotte Corporation. The investigation was carried out in the Transportation Development Section of the Portland Cement Association under the direction of Bert E. Colley.

REFERENCES

1. A. N. Hanna. Properties of Expanded Polystyrene Concrete and Application for Pavement Subbases. Portland Cement Association, Skokie, IL, RD057.01P, 1978.
2. Standard Specifications for Road and Bridge Construction. Department of Public Works and Buildings, Illinois Division of Highways, Springfield, 1971.
3. W. J. Nowlen. Techniques and Equipment for Field Testing of Pavements. Journal of PCA Research and Development Laboratories, Vol. 6, No. 3, 1964, pp. 43-52; Portland Cement Association, Bulletin D83, Skokie, IL.
4. H. P. Aldrich, Jr. Frost Penetration Below Highway and Airfield Pavements. HRB, Bulletin 135, 1956, pp. 145-149.
5. D. M. Burmister. Theory of Stresses and Displacements in Layered Systems and Applications to Design of Airport Runways. Proc., HRB, Vol. 23, 1943, pp. 126-148.
6. A. Jones. Tables of Stresses in Three-Layered Elastic Systems. HRB, Bulletin 342, 1962, pp. 156-214.

Publication of this paper sponsored by Committee on Frost Action.

Some Factors That Affect Thaw Strain

Bernard D. Alkire, Michigan Technological University

Factors that affect the thaw strain of a remolded clay were studied by using conventional isotropic and anisotropic consolidation techniques. A test sequence was developed to simulate a temperature history experienced by soils in many parts of the continental United States. The test sequence was divided into 24-h increments and included isothermal consolidation, undrained freezing, undrained thaw, and isothermal recompression. For all test sequences, the loading applied before the consolidation phase was held constant throughout the test period. The effect of various factors on thaw strain was determined by comparison of results obtained from isotropically and anisotropically loaded samples subjected to confining pressures of 103, 207, and 414 kPa (15, 30, and 60 lbf/in^2). In addition, the effect of repeated freeze-thaw was obtained from tests where the undrained freeze and undrained thaw phase of the test sequence was repeated up to four times. Results from the test indicate that thaw strain increases with increasing principal stress ratio and also increases as the confining pressure decreases. It was also observed that excess pore pressures developed during the undrained freezing and undrained thaw phase of the test and that these pore pressures are closely related to the development of thaw strain.

It is generally maintained that most strains that occur in soils are related to the application of boundary stresses. However, other factors are also capable of causing strain, and in some cases they may be more significant than boundary stresses. Typical examples are strains caused by changes in the physicochemical environment and pore water chemistry. More specifically, some researchers (1, 2) have documented the effect of temperature change on soil compressibility and pore pressure, and others (3, 4, 5) have investigated the strain that occurs when freezing is followed by thawing.

The purpose of the experimental tests described in this paper was to assess the effect of principal stress ratio, confining pressure, and freeze-thaw cycling on the magnitude of thaw strain. This information was then used to develop a mechanistic picture to describe thaw strain.

The general experimental approach involved triaxial consolidation of a test soil under anisotropic or isotropic stress conditions. Each test involved at least a single drained consolidation, undrained freeze, undrained thaw, and drained recompression phase. Various confining pressures and principal stress ratios were used, and some of the tests involved up to four cycles of undrained freezing and

undrained thawing. Results from the test program indicate that thaw strain is substantially influenced by principal stress ratio, number of freeze-thaw cycles (thermal history), and magnitude of confining pressure.

TEST PROCEDURE

All tests conducted during the experimental program were triaxial consolidation tests that followed a set sequence once the sample was mounted in the triaxial device. The basic test sequence involved isotropic consolidation at 49.1 kPa (7.1 lbf/in^2), application of the test stress conditions and isothermal consolidation of the sample, undrained freezing at -6°C (21°F), undrained thawing, and isothermal reconsolidation. Each phase was continued for 24 h. Throughout the test, the confining pressure and axial load applied during the initial isothermal consolidation were maintained, and axial deformation, sample volume change, pore water pressure, and temperature were observed. For some of the tests, the basic sequence was altered by adding one or more cycles of undrained freezing and thawing after the isothermal reconsolidation. These additional phases were also 24 h in duration to allow adequate time for complete freezing and thawing and development of any additional thaw strain.

The equipment used for the tests was a single-cell triaxial test assembly modified for use at low temperatures. Sample volume changes during the drained phases of testing were measured by using a 25-mL (0.007-gal) burette connected to the sample through the base of the cell. Temperature control was achieved by circulating a cooling fluid (50 percent water and 50 percent ethylene glycol) through two layers of polyethylene tubing wrapped around the outside of the triaxial cell. The cooling fluid was cooled by using a constant temperature bath and a portable cooling unit. Temperatures interior to the triaxial cell were measured with a laboratory thermometer (1.0°C division) inserted into a test well fitted to the top of the triaxial cell. Axial displacement was measured with a dial gauge [0.0025 mm (±0.0001 in)], and pore water pressure was measured by using a temperature-compensated pressure transducer connected to a strip recorder.

SAMPLE PREPARATION AND SOIL TYPE

Samples used during the test program were formed by an accelerated sedimentation process. The samples were formed in metal tubes 30.48 cm (12 in) high and 3.49 cm (1.38 in) in diameter and, when ready for testing, were extruded from the tube and cut to a length of 8.0 cm (3.15 in). To make a sample, 150 g (0.331 lb) of oven-dried soil passing a 425-μm (No. 40) U.S. standard sieve was added to water to produce a slurry. The slurry was poured into the tubes and, after 24 h of sedimentation, a plunger was inserted into the tube and the slurry was consolidated to the selected initial water content of 37 percent.

The soil used for the test program was a red silty clay obtained from a roadway cut near Houghton, Michigan. The liquid limit is 38.5, and the plastic limit is 25.5. Thirty percent, by weight, of the particles are smaller than 0.002 mm (7.87 μin) in effective size, and the specific gravity of the solids is 2.78. The classification according to the Unified Soil Classification system is CL.

TEST RESULTS AND DISCUSSION

Freeze-Thaw Tests

A normal test sequence involved several phases and in some cases was over a week in duration. Typical results, presented as vertical strain versus square root of time, are shown in Figure 1 for an anisotropically consolidated sample with a confining pressure of 207 kPa (30 lbf/in^2) and a principal stress ratio of 1.2. It should be noted that the horizontal axis is discontinuous and that each new phase starts at zero. In addition, compressive strain is taken as positive in the upward vertical direction.

The first phase, drained compression, was completed isothermally and on the square root of time plot is similar to the type of curve one would expect for a silty clay. This part of the test sequence was basically a conditioning phase in which the sample was brought to known prefreeze conditions. During this phase, it was possible to check the response of the measuring equipment and to observe the sample for leaks or other undesirable conditions. The results from this phase might also be used to calculate the unfrozen strain modulus, Poisson's ratio, and a three-dimensional coefficient of consolidation.

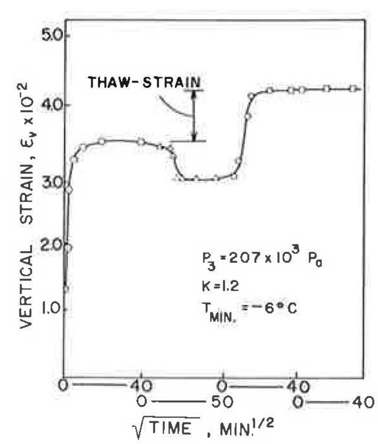
At the end of the compression phase, the drainage lines to the sample were closed and the temperature was lowered. Typically, the sample responded with a slight expansion as the temperature decreased toward freezing and then a large expansion as freezing of the pore water and its associated phase change occurred. Once the pore water was frozen, sample expansion decreased or continued at a low rate because of creep of the frozen soil.

Undrained thaw followed a strain versus square root of time path that was essentially the reverse of the undrained freezing. As the temperature increased, the sample slowly began to contract until the temperature approached 0°C (32°F), and then the sample rapidly contracted as pore water phase change took place. Note, however, that more contraction occurred during this phase than expansion during the freeze cycle; the difference is the thaw strain associated with an undrained freeze-thaw cycle. During the thaw cycle, excess pore pressures developed that, when allowed to dissipate during the recompression phase, caused additional vertical strain (thaw consolidation). All tests conducted on the clay exhibited similar behavior; any difference between tests was a function of stress level, test type, and ambient conditions.

Effect of Applied Stresses

As shown in Figures 2 and 3, the magnitude and ratio of the applied stresses have a substantial in-

Figure 1. Typical development of vertical strain during test to determine thaw strain.



fluence on the development of vertical strain. In these figures, the strains were normalized by subtracting the total strain at the end of the isothermal compression phase from the total strain at any time during the freeze, thaw, or recompression phase. Thus, the zero point in time shown in the figures is the end of the compression phase, and the plotted strains show only the influence of temperature change. The last plotted point on each curve gives the net strain that occurred throughout the freeze, thaw, and recompression phases.

For the anisotropically consolidated samples shown in Figure 2, it can be observed that strain during the freezing phase was approximately equal for the samples and that the lower consolidation stress resulted in the largest amount of strain during the thaw cycle. Thus, the higher the confining pressure is, the lower is the thaw strain.

From Figure 3 it can be seen that, for samples subjected to the same confining pressure, the largest thaw strain is associated with the anisotropically consolidated sample, there is little recompression of the anistropically consolidated sample, and the vertical strain associated with freezing was much larger for the isotropically loaded sample.

Freeze-Thaw Cycling

In an attempt to determine the influence of repeated freeze-thaw, several samples were subjected to a number of freeze-thaw cycles. The results are shown in Figure 4. In this figure, only the normalized strain at the end of each cycle is shown. In addition, cycles 2, 3, and 4 have been corrected to remove the strain that occurred during the drained recompression after the first freeze-thaw cycle. Note that there are two groups of curves: the upper curves, which represent the locus of points determined from the normalized strain at the end of a given thaw cycle, and the lower curves, which represent the normalized strain at the end of a given freeze cycle. As shown in the figure, the upper and associated lower curves for a given test can be used to separate the total vertical strain during a freeze-thaw cycle into a

Figure 2. Effect of consolidation stress conditions on development of vertical strain.

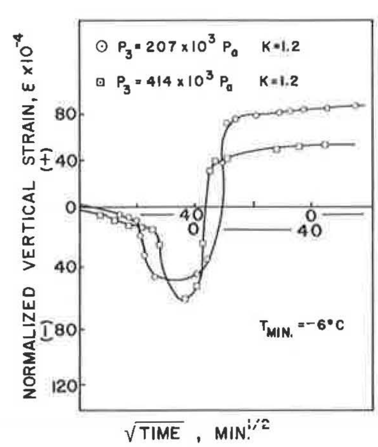


Figure 3. Effect of principal stress ratio on development of vertical strain.

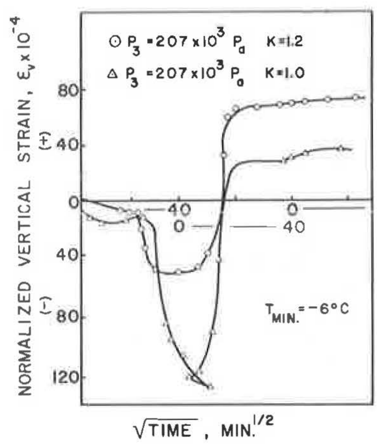


Figure 4. Effect of freeze-thaw cycles on development of thaw strain.

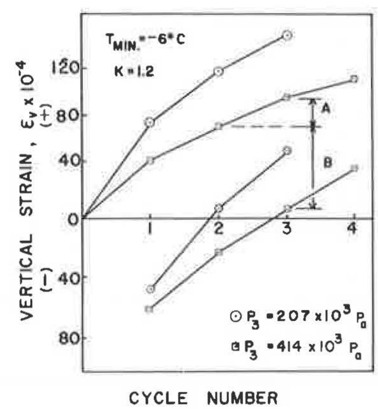


Table 1. Vertical strains observed during test phases.

Sample	Confining Pressure (kPa)	Test	Vertical Strain: Compression Phase	Freeze Phase	Thaw Phase	Recompression Phase	Thaw Strain
3F	207	Isotropic	0.000 240	-0.000 147	0.000 175	0.000 010	0.000 028
5F	207	Isotropic	0.000 094	-0.000 102	0.000 182	0.000 006	0.000 080
5F			NA	-0.000 128	0.000 170	NA	0.000 042
7F	414	Isotropic	0.000 266	-0.000 208	0.000 205	0.000 031	-0.000 003
9F	104	Isotropic	0.000 160	-0.000 207	0.000 242	0.000 014	0.000 035
9F	207	Isotropic	0.000 092	-0.000 143	0.000 173	0.000 004	0.000 030
9F	414	Isotropic	0.000 087	-0.000 144	0.000 168	0.000 002	0.000 024
4F	207	Anisotropic	0.000 409	-0.000 064	0.000 216	0.000 034	0.000 152
6F	207	Anisotropic	0.000 348	-0.000 055	0.000 130	0.000 005	0.000 075
6F	207	Anisotropic	NA	-0.000 066	0.000 106	NA	0.000 040
6F	207	Anisotropic	NA	-0.000 062	0.000 090	NA	0.000 028
8F	414	Anisotropic	0.000 216	-0.000 071	0.000 117	0.000 006	0.000 046
8F	414	Anisotropic	NA	-0.000 077	0.000 098	NA	0.000 026
8F	414	Anisotropic	NA	-0.000 064	0.000 087	NA	0.000 023
8F	414	Anisotropic	NA	-0.000 056	0.000 071	0.000 004	0.000 015

Note: 1 kPa = 0.145 lbf/in^2.

component caused by phase change (B) and a component caused by undrained thaw strain (A). For example, the difference between the vertical coordinates on the upper curve for cycles 2 and 3 is the thaw strain that occurred during cycle 3.

Several factors appear to be occurring during freeze-thaw cycling. First, there is a general increase in total vertical strain as the number of cycles increases. This appears to be true for both isotropically and anisotropically loaded samples. Second, there is a general decrease in additional thaw strain per cycle as the number of cycles increases. Finally, the nearly straight lines that connect the end of the freezing phase (lower curves) suggest that phase change during each cycle is nearly constant.

A summary of the vertical strain results for all tests is given in Table 1. The values listed for each phase were obtained by subtracting the vertical strain at the end of each phase from the vertical strain at the beginning of the phase. Negative values are associated with sample expansion. Also shown are the thaw strains for each test. These values are obtained by algebraically adding the results of the freeze phase to the thaw phase. If the strain from the recompression phase is added to the thaw strain, the total temperature-induced vertical strain can be obtained. The values in this table support the observations already made in the discussion of the various test factors.

Pore Pressure

Throughout the test period, pore pressures were monitored as long as the pore fluid remained unfrozen. Because of the limitations of the test equipment, pore pressures were measured only at the base of the sample. Nevertheless, some trends appeared that are worthy of discussion.

Considering the entire test sequence, excess pore pressures measured during the isothermal drained consolidation phase dissipated in the conventional manner and at the end of this phase were zero. Once sample drainage was prevented and temperatures decreased, pore pressures dropped into the negative region. The magnitude of the decrease varied somewhat from test to test but was in the range of 3 to 30 kPa (0.5 to 4 lbf/in^2). Pore pressures continued to drop during this phase until the sample began to freeze, and then the pore pressures rapidly increased (because of the volume change that occurs during freezing).

During the thaw phase, pore pressure began to increase and continued to increase until the sample returned to laboratory temperature. Typical increases in pore pressure during this phase were 13 to 80 kPa (2 to 11.5 lbf/in^2). After the undrained thaw phase, the excess pore pressures that developed during thaw were allowed to dissipate during the recompression phase.

A summary of pore pressure data obtained during the undrained freeze and undrained thaw phases is given in Table 2. For undrained freeze, U_{max} is the pore pressure at the beginning of the phase, and U_{min} is the last pore pressure reading before freezing of the pore fluid. During undrained thaw, U_{max} is the last pore pressure reading taken during the phase. The change in pore pressure ΔU that occurred during undrained freeze is $U_{min} - U_{max}$ and for undrained thaw is $U_{max} - U_{min}$ (from the undrained freeze phase). In general, it can be observed that pore pressures decreased as the temperature decreased and increased as the temperature increased. The magnitude of the change in pore pressure during undrained freezing increases with increasing confinement and with increased cycles of freeze-thaw. The same observations are true for undrained thaw, the difference being the sign of the change in pore pressure.

Thaw-Strain Mechanism

Before a mechanistic picture of thaw strain can be proposed, it is necessary to assume some conditions. Thaw strain as defined here includes only the net vertical strain (+ or -) in excess of phase change that occurs during the undrained freeze and thaw phases. In addition, the soil is normally consolidated, the thermal gradient applied to the soil is such that migration of pore water toward the freezing front (at least on a local basis) can occur, and the phase change associated with water-ice transformations is reversible. If these conditions are assumed, it is possible to propose a mechanism for thaw strain similar to that proposed by others for strains that occur without a change in boundary stress (1, 6, 7, 8).

The explanation for thaw strain can be developed by using Figure 5. This figure shows typical curves of volumetric and vertical strain versus effective stress that describe the compressibility characteristics of a soil. Along these curves a given soil might be consolidated isothermally to a point represented by A or A'. At this point, drainage is pre-

Table 2. Pore pressure change observed during undrained freeze and undrained thaw phases of testing.

				Test Phase				
				Undrained Freeze			Undrained Thaw	
Sample	Cycle	Confining Pressure (kPa)	Test Type	U_{max} (kPa)	U_{min} (kPa)	ΔU (kPa)	U_{max} (kPa)	ΔU (kPa)
6F	1	206.9	Anisotropic[a]	0	-3.45	-3.45	27.6	31.0
	2			6.90	-6.90	-13.80	6.90	13.80
	3			3.45	-10.30	-13.80	0	10.30
8F	1	413.9	Anisotropic[a]	0	-6.90	-6.90	44.83	51.73
	2			6.90	-3.45	-10.35	31.04	34.49
	3			31.04	13.80	-17.24	37.94	24.14
	4			37.94		-24.14	34.49	20.69
4F	1	206.9	Anisotropic[a]		-[b]			
9F	1	103.5	Isotropic	6.90	-13.80	-20.69	37.94	51.73
	2	206.9		13.80	0	-13.80	27.59	27.59
	3			20.69	6.90	-13.80	27.59	20.69
7F	1	413.9	Isotropic		-[b]			
5F	1	206.9	Isotropic	6.90	-20.69	-27.59	51.73	72.42
	2			13.80	6.90	-6.90	34.49	27.59
3F	1	206.9	Isotropic	13.80	6.90	-6.90	82.77	75.87

Note: 1 kPa = 0.145 lbf/in^2.

[a]K = 1.2. [b]No results.

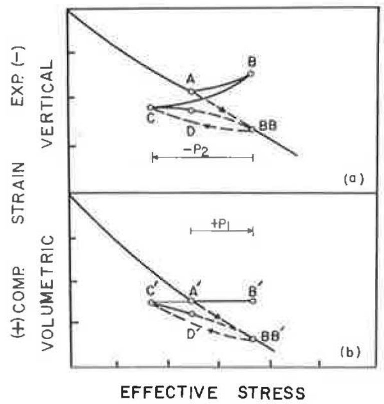


Figure 5. Development of vertical and volumetric strain induced by temperature change during freeze-thaw.

vented and the temperature is lowered. During this period, thermal contraction of the pore fluid, mineral solids, and soil mass may all contribute to observed strain. However, because drainage is prevented and contraction of the pore fluid is greater than the soil skeleton, a negative pore pressure is induced to offset the tendency toward volume reduction. The negative pore pressure causes an increment of effective stress $+p_1$, and, if drainage were allowed, a volumetric strain B' - BB' would occur. Since drainage is prevented, there is no net volumetric strain; however, there is a vertical strain proportional to the increase in pore pressure (9). This is shown in Figure 5(a) as the vertical difference between A and B. The magnitude of the vertical strain is a function of the swell (expansion) characteristic of the soil and should be less than the vertical strain that would occur as a result of a virgin compression for the same increment of effective stress ($+p_1$).

The same phenomena occur during undrained thaw where temperature increases from just above freezing to laboratory temperature. In this case, the induced pore pressure will be positive, and the effective stress will decrease by some amount $-p_2$. In a drained sample, this would cause volumetric rebound from BB' to C' with the end result that the volumetric strain would remain unchanged during the undrained freeze-thaw cycle. Because of the different compressibility characteristics of a soil during rebound and recompression, p_1 and p_2 are not equal and, at the end of undrained thaw, there is an excess pore pressure equal to $(p_2 - p_1)$. If it is allowed to dissipate, this pore pressure difference causes a volumetric strain C' to D'. The net result is a volumetric strain that is related to both the magnitude of excess pore pressure developed during the freeze-thaw cycle and the recompression characteristics of the soil.

Vertical strain also responds to a temperature increase by development of a compressive strain that is proportional to the induced pore pressure change. This strain--B to C in Figure 5(a)--results in a net thaw strain equal to A-C at the end of the undrained thaw phase. Additional vertical strain between C-D also occurs as the excess pore pressure developed during undrained thaw is allowed to dissipate. Thus, vertical strain is developed in two components: a constant volume adjustment of vertical and horizontal strain caused by temperature-induced pore pressure change and the vertical strain associated with dissipation of excess pore pressure developed during the undrained freeze-thaw cycle.

SUMMARY

Results from the test program show that thaw strain is dependent on applied stress ratio, confining pressure, and stress history. The following general observations can be made concerning the development of thaw strain:

1. For the same confining stress, an anisotropically loaded sample will develop more thaw strain than an isotropically loaded sample.
2. As the level of confining stress increases, thaw strain decreases for both isotropic and anisotropic loadings.
3. Increasing the number of cycles of freezing and thawing increases the total thaw strain. However, the increment of thaw strain for successive cycles decreases.
4. Recompression (thaw consolidation) after undrained thaw is also a function of stress ratio. Isotropically loaded samples generally exhibited the most recompression. A related observation is that isotropic samples also have the largest temperature-induced pore pressure at the end of the thaw phase.

ACKNOWLEDGMENTS

The work conducted as part of this study was performed at Michigan Technological University and was supported by a National Science Foundation grant. The laboratory tests were conducted by Mark Grandstrand and George Ehlert. Graphical presentations were completed with the assistance of William Welch and Kenneth Wlodarczak. Their assistance in completing this paper is gratefully acknowledged.

REFERENCES

1. R. G. Campanella and J. K. Mitchell. Influence of Temperature Variations on Soil Behavior. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM3, May 1968, pp. 709-733.
2. R. T. Plum and M. I. Esrig. Some Temperature Effects of Soil Compressibility and Pore Water Pressure. HRB, Special Rept. 103, 1969, pp. 231-242.
3. N. R. Morgenstern and L. B. Smith. Thaw-Consolidation Tests on Remodeled Clays. Canadian Geotechnical Journal, Vol. 10, No. 1, 1973, pp. 25-40.
4. N. A. Tsytovich. Thaw Settling of Frozen Ground. In The Mechanics of Frozen Ground, McGraw-Hill, 1975.
5. U. Luscher and S. S. Afifi. Thaw Consolidation of Alaskan Silts and Granular Soils. Proc., 2nd International Conference on Permafrost, National Science Foundation, North American Contribution, 1973, pp. 325-334.
6. J. F. Nixon and N. R. Morgenstern. The Residual Stress in Thawing Soils. Canadian Geotechnical Journal, Vol. 10, No. 4, 1973, pp. 571-580.
7. G. R. Martin, L. W. D. Finn, and H. B. Seed. Fundamentals of Liquefaction Under Cyclic Loading. Journal of Geotechnical Engineering Division, ASCE, Vol. 101, No. GT5, May 1975, pp. 423-438.
8. T. L. Holzer, K. Hoeg, and K. Arulanandan. Excess Pore Pressures During Undrained Clay Creep. Canadian Geotechnical Journal, Vol. 10, No. 1, Feb. 1973, pp. 12-24.
9. L. K. Walker. Undrained Creep in a Sensitive Clay. Geotechnique, Vol. 19, No. 4, 1969, pp. 515-529.

Publication of this paper sponsored by Committee on Frost Action.