Behavior of Frozen and Unfrozen Sands in Triaxial Testing

H. YOUSSEF AND A. HANNA

Frozen and unfrozen soils are natural composite materials composed of soil particles and voids that can be partly or totally filled with ice or water. When the temperature of the soils decreases below 0°C, its water phase crystallizes to ice, which changes its mechanical behavior. The purpose of this paper is to present the results of an experimental investigation of the behavior of frozen and unfrozen sands in triaxial testing. The results are presented in the form of a comparative analysis of the relationships among stress, strain, and volume change of these materials tested under the same conditions of confining pressure and strain rate. In the conclusion, emphasis is given to structures that are usually subjected to such changes in behavior because of seasonal temperature changes.

Because of the high viscosity of intergranular ice, the strength of frozen sand is due to its ice cohesion as well as its frictional components. This strength is time dependent; unfrozen sand is a cohesionless material, and because of the low viscosity of the intergranular water, its shear behavior is basically time independent.

Triaxial testing of frozen sands is essentially of one type—closed-system conditions—because the intergranular ice is not free to move out of the samples during testing in shear. However, these samples exhibit volume changes (*1–3*). Triaxial testing of unfrozen sands is mainly of two types (*4, 5*): drained and undrained.

This paper presents the basic difference between the mechanical behavior of unfrozen sand and its state when it is frozen to −5°C during triaxial testing.

TERMINOLOGY

The following terms are used in this paper:

C = cohesion component of shearing resistance of frozen sands,
D_r = relative density,
DU = unfrozen sand sample tested in drained conditions,
e = voids ratio,
F_S = frozen soil sample,
n = porosity,
p = hydrostatic (normal) stress = $\frac{1}{2}(\sigma_1 + \sigma_3)$,
q = shear stress = $\tau = \frac{1}{2}(\sigma_1 - \sigma_3)$,
S_i = degree of saturation of ice,
S = degree of saturation of water,
US = unfrozen sand sample tested in undrained conditions,
U_f = pore-water pressure at failure,
V_s = volume of sand grains in the frozen sample,
V_i = volume of ice in the frozen sample,
W_i = ice content,
W = water content,
ε_{if} = axial strain corresponding to the peak stress,
ε_v = volumetric strain,
ε_1 = axial strain rate,
ϕ = angle of shear resistance,
γ_T = total unit weight,
γ_D = dry unit weight,
τ_{oct} = octahedral shear stress = $(\frac{2}{3})^{\frac{1}{2}}(\sigma_1 - \sigma_3)$, and
σ_{oct} = octahedral normal stress = $\frac{1}{3}(\sigma_1 + 2\sigma_3)$.

Department of Civil Engineering, Concordia University, 1455 de Maisonneuve Boulevard West, Montreal, Quebec H3G 1M8, Canada.

EXPERIMENTAL STUDY

A series of triaxial tests was performed on frozen and unfrozen cylindrical samples of silica sand (average grain size, 0.06 to 0.80 mm) with nominal dimensions of 38.10-mm diameter by 76.20-mm length. The physical properties of the tested samples are reported in Tables 1–3 (*2, 6*), from which it can be seen that the voids ratio for both frozen and unfrozen samples varies in the same range of 0.53 to 0.72. This permits the use of these samples to perform a quantitative comparison between frozen and unfrozen test results.

The testing procedures carried out on unfrozen soils followed the conventional methods described by Bishop and Henkel (*4*) and Bowles (*5*). The procedures followed for sample preparation and testing of frozen sands were essentially the same as those utilized at the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (*7*) and modified by Youssef (*2, 3*); the tests were performed at a temperature of −5°C. The test results are summarized in Table 4. Typical test results for frozen and unfrozen sands are shown in Figures 1–3.

ANALYSIS OF TEST RESULTS

Referring to Table 4 and Figure 1, it can be observed that the short-term strength is influenced to a high degree by

TABLE 1 PHYSICAL PROPERTIES OF FROZEN SANDS (*2*)

Test Number	Void Ratio, e	Ice Content, W_i %	Degree of Saturation S_i %	Volume of Sand Grains, $V_s \times 10^{-6}$ m^3	Volumetric Ratio of Ice to Sand Grains, V_i/V_s	Total Unit Weight, γ_T kN/m^3	Dry Unit Weight γ_D kN/m^3
FS 1	0.6	21	95	54	0.6	19.6	16.3
FS 2	0.6	20	98	54	0.6	19.9	16.5
FS 3	0.7	22	92	50	0.6	19.0	15.7
FS 4	0.6	21	95	54	0.6	19.5	16.2
FS 5	0.7	28	92	50	0.6	19.3	15.9
FS 6	0.7	22	99	53	0.7	19.6	16.0
FS 7	0.6	19	94	52	0.5	19.9	16.8
FS 8	0.7	20	90	53	0.6	19.3	16.0
FS 9	0.7	20	90	53	0.6	19.3	16.0
FS 10	0.6	21	93	55	0.6	19.5	16.2
FS 11	0.7	23	100	55	0.7	19.7	16.1
FS 12	0.6	21	94	55	0.6	19.5	16.1
FS 13	0.6	20	93	55	0.6	19.6	16.3
FS 14	0.7	21	93	52	0.6	19.3	15.9
FS 15	0.6	19	93	56	0.5	20.0	16.8

TABLE 2 PHYSICAL PROPERTIES OF CONSOLIDATED DRAINED UNFROZEN SANDS (*6*)

Test No.	Dry Unit Weight γ_D kN/M^3	Void Ratio, e	Porosity n %	Relative Density D_r %	Cell Pressure σ_3, kPa	Deviator Stress $(\sigma_1-\sigma_3)_f$ kPa	Volumetric Strain ε_{vf}%	Axial Strain ε_1%	Angle of Shearing Resistance ϕ^omax
DU1	17	0.6	37	78	167	557	1	4	38
DU2	17	0.6	39	68	334	997	1	4	36.
DU3	17	0.6	38	75	434	1234	1	5	34.
DU4	17	0.6	39	71	167	667	1	6	37
DU5	16	0.7	42	45	334	848	0.4	8	34
DU6	18	0.5	35	93	167	512	2	3	39
DU7	18	0.5	35	92	334	1083	1	4	38
DU8	18	0.6	36	88	434	1339	1	4	38
DU9	17	0.6	38	73	334	1001	1	3	37
DU10	18	0.5	35	93	434	1419	1	4	39

TABLE 3 PHYSICAL PROPERTIES OF CONSOLIDATED UNDRAINED UNFROZEN SANDS (*2*)

Test No.	Void Ratio, e	Water Content, W %	Degree of Saturation S %	Volume of Sand Grains, $V_s \times 10^6$ m^3	Vol. Ratio of Water to Sand, V_w/V_s	Total Unit Weight γ_T, kN/m^3	Dry Unit Weight γ_D, kN/m^3
US1	0.7	25	98	67	0.7	19.7	15.80
US2	0.6	24	99	69	0.6	20.0	16.30
US4	0.7	24	99	71	0.6	20.0	16.00
US5	0.6	22	99	70	0.6	20.4	16.80
US6	0.6	21	100	72	0.6	20.6	17.00
US7	0.7	25	100	68	0.7	20.0	16.00
US8	0.6	23	100	69	0.6	20.3	16.50
US9	0.6	23	100	69	0.6	20.3	16.60
US10	0.6	24	99	69	0.6	20.0	16.20
US11	0.7	25	100	68	0.7	19.9	15.90
US12	0.8	25	99	68	0.7	19.8	15.80

TABLE 4 TEST RESULTS: FROZEN SANDS (*2*)

Test Conditions				Maximum Stresses, kPa						
Test No.	Strain Rate, $\varepsilon \times 10^{-5}$ sec^{-1}	Failure Strain, ε_f %	Confining Pressure σ_3 kPa	$(\sigma_1 - \sigma_3)$	σ_1	(σ_1/σ_3)	τ_{oct}	σ_{oct}	p	q
FS1	3.20	4	277	9100	9381	33.35	4290	3315	4831	4550
FS2	3.25	4	138	8041	8182	58.03	3791	2821	4162	4021
FS3	3.40	3	345	5057	5409	15.37	2384	2038	2880	2528
FS4	161.00	2	448	11440	11893	26.02	5391	4269	6175	5718
FS5	170.00	3	448	10200	10656	23.32	4808	3857	5557	5100
FS6	162.00	2	448	10510	10967	24.00	4954	3960	5712	5255
FS7	3.50	4	448	9590	10047	21.99	4520	3653	5252	4795
FS8	3.25	3	448	7180	7637	16.71	3384	2850	4047	3590
FS9	3.25	3	448	7030	7430	16.38	3314	2800	3972	3515
FS10	3.20	3	448	7235	7692	16.83	3410	2869	4074	3617
FS11	3.20	5	448	10543	11000	24.07	4922	3938	5678	5221
FS12	3.20	5	138	8488	8629	61.20	4001	2970	4385	4244
FS13	3.20	4	277	7859	8141	28.87	3705	2901	4211	3930
FS14	3.30	5	553	11190	11749	20.87	5273	4292	6156	5593
FS15	3.25	2	448	5618	6075	13.29	2649	2330	3266	2809

the applied strain rate (ε_1) and the level of confining pressure (σ_3). In addition it is a function of its physical properties, mainly the initial voids ratio (e_i) and the degree of saturation. The effect of the applied strain rate on the strength of the frozen sand is noted by comparing Samples FS4 and FS10, both of which have similar physical properties (Table 1) and are subjected to identical testing conditions of confining pressure and temperature (Table 4). Sample FS4 was tested under an applied strain rate of 1.61×10^{-3} sec^{-1}, whereas Sample FS10 was tested at a strain of 3.19×10^{-5} sec^{-1}. The resulting shear stress ratio is $\tau_4/\tau_{10} = 1.58$, which indicates that the higher the strain rate, the higher the peak shear strength of the tested frozen sand. This is due to the high viscosity of the intergranular ice phase.

The variation of the voids ratio influences the shear strength of frozen sands. In general, the smaller the voids ratio, the higher the shear strength, as shown in Tables 1 and 4.

The increase of the confining pressure from 345.31 to 552.30 kPa (Tests FS3 and FS14) causes an increase in the shear strength of 22 percent. This is in agreement with Goughnour and Andersland (*8*) and Chamberlain et al. (*9*), who found that increasing the confining pressure increases the strength of the sand. This is due to the fact that increasing the confining pressure causes the sand particles to be held in more intimate contact, which makes the grain boundary adjustment more difficult and consequently increases the intergranular strength.

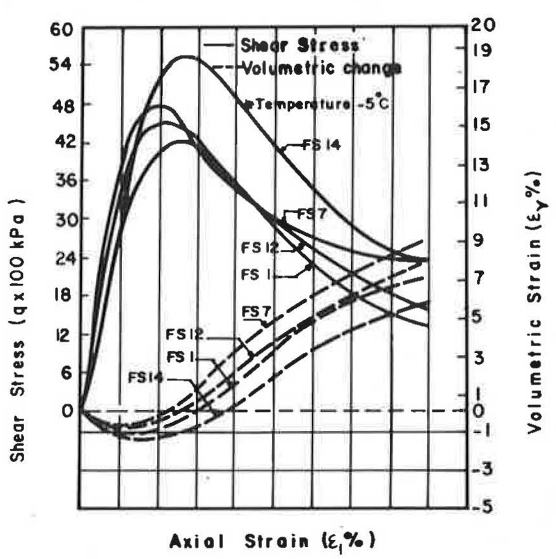

FIGURE 1 Test results for frozen sands.

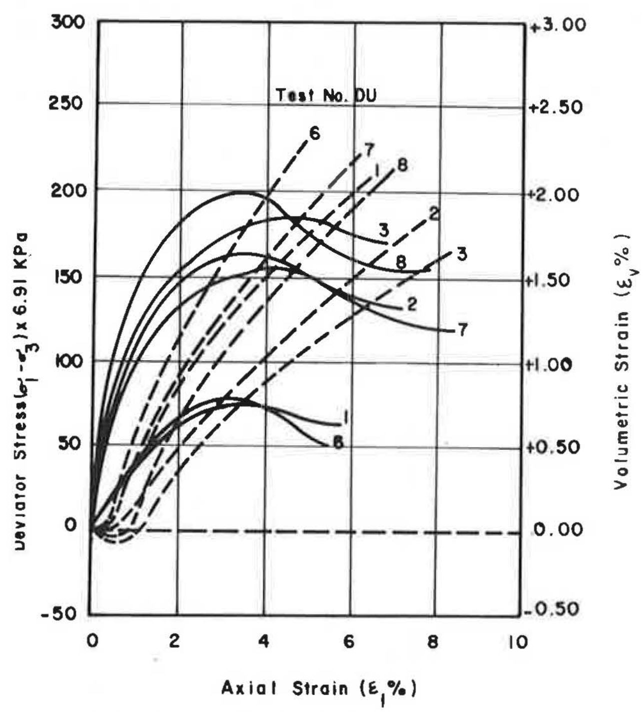

FIGURE 2 Test results for drained unfrozen sands.

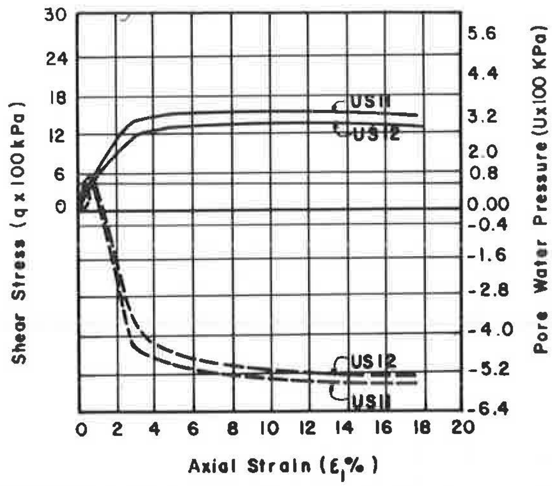

FIGURE 3 Test results for undrained unfrozen sands.

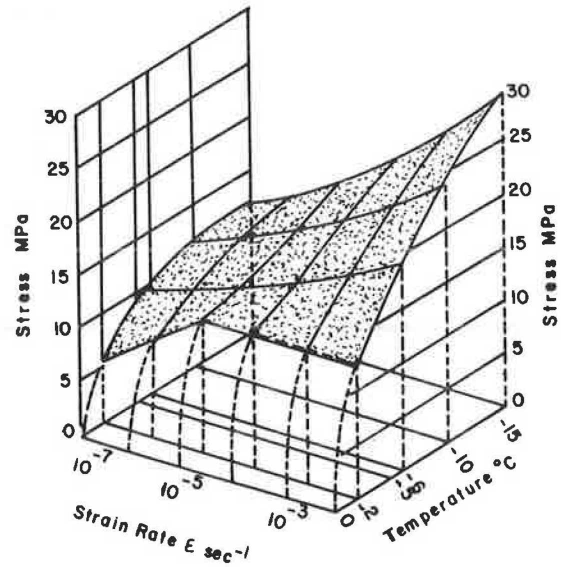

FIGURE 4 Variation in strength of frozen sand with temperature and applied strain rate ε_1 [after Parameswaran (*10*)].

The influence of the degree of saturation (S_i) on the shear strength can also be traced from Table 4. The higher the degree of saturation, the higher the shear strength of the frozen sands. As previously mentioned, the increase in the shear strength because of the increase in the degree of saturation is attributed to the increase in the area of contact between the sand particles and the ice. This in turn causes intensification of the cementation bond.

As can be seen in Figure 1, the volumetric change behavior for all samples was tested under frozen conditions. The volume initially decreases with an increase in the axial strain; it shows a rapid increase up to the failure strain; then it continues to increase with a milder slope to the end of the test (strain level, 20 percent).

The stress-strain behavior shows one peak at a strain level in the range of 3.94 to 5.30 percent, depending on the applied confining pressure, strain rate, and physical properties of the sample (see Table 4 and Figure 1). In general, the applied strain or deformation rate affects the magnitude of the failure strain because of the high viscosity of the intergranular ice in the frozen sample.

Parameswaran (*10*) presented the dependence of the uniaxial strength of frozen soils as a function of temperature and strain rate (Figure 4). As can be seen from this diagram, increasing the confining pressure as well as decreasing the temperature results in increasing the frozen soil strength.

The test results of drained unfrozen sand (Figure 2 and Tables 2 and 5) show that increasing the confining pressure increases the drained shear strength. The failure strain (ε_{1f}) varies from 2.95 to 8 percent depending on the voids ratio after consolidation (e) and the applied confining pressure (σ_3). The maximum deviatoric stress was taken as the failure criterion, which was similar to those for frozen and undrained unfrozen results. The maximum deviatoric stress ($\sigma_1 - \sigma_3$) at failure depends on the voids ratio (e), and the confining pressure (σ_3) varies from 512.08 to 14.53 kPa. The volumetric strain (ε_v) during the shearing stage initially shows compressive behavior at small strain levels up to and close to an axial strain of 1 percent and then starts to increase progressively as the samples dilate, first with

TABLE 5 TEST RESULTS: DRAINED UNFROZEN SANDS (*6*)

Test Conditions*				Maximum Stresses, kPa						
Test No.	Axial Strain ε_1%	Vol. Strain ε_v%	Confining Pressure σ_3 kPa	$(\sigma_1 - \sigma_3)$	σ_1	(σ_1/σ_3)	τ_{oct}	σ_{oct}	p	q
DU1	4	1	167	557	724	4.34	262	353	445	279
DU2	4	1	334	997	1030	3.10	469	566	682	498
DU3	5	1	434	1234	1668	3.85	581	845	1051	617
DU4	5	1	167	667	834	5.00	314	389	500	334
DU5	8	0.40	334	848	1181	3.54	399	616	607	424
DU6	3	2	167	512	679	4.10	241	338	423	256
DU7	4	1	334	1083	1417	4.25	510	695	875	541
DU8	4	1	434	1339	1773	4.10	631	880	1103	670
DU9	3	1	334	1001	1334	4.00	471	667	834	500
DU10	4	1	434	1419	1852	4.27	668	907	1143	709

* Strain Rate [ε] = 0.25 x 10^{-5} sec^{-1}

TABLE 6 TEST RESULTS: UNDRAINED UNFROZEN SANDS (*2*)

Test	Test Conditions			Maximum Total Stresses [KPa]						
Number	ε_1%	σ_3	U_b	$[\sigma_1 - \sigma_2]$	σ_1	p	q	σ_{oct}	σ^{oct}	σ_1/σ_3
US 1	13	277	504	1984	*2765 **2260	1773 1268	992	1442	935	3.54
US2	8	277	804	2669	3750 2946	2415 1346	1334	1971	1259	3.47
US3	7	448	602	3038	4089 3487	2570 1968	1519	2064	1432	3.89
US4	7	448	602	3006	4056 3453	2553 1951	1503	2051	1417	3.86
US5	4	448	601	3304	4354 3752	2702 2100	1652	2151	1558	4.15
US6	21	448	906	2960	4314 3408	2834 1929	1480	2341	1395	3.19
US7	19	139	600	1915	2654 2053	1696 1096	958	1377	885	3.59
US8	14	138	600	2088	2826 2226	1783 1182	1044	1435	984	3.83
US9	20	277	600	2503	3380 2779	2128 1528	1251	1711	1179	3.85
US10	7	552	501	3062	4115 3614	2584 2084	1530	2073	1443	3.90
US11	8	552	501	2668	3722 3222	2388 1887	1334	1943	1258	3.53

* values do include U_b
** values do not include U_b

a relatively steep slope up to the strain level (ε_{1f}) (corresponding to the sample shear strength) followed by a milder slope of increase to the end of the test. The values of (ε_v) at failure vary from 0.40 to 1.50 percent, depending on (e_1) and (σ_3). In general, the volumetric strain at failure decreases with an increase in the confining pressure because of the decrease in the interlocking of the sand particles and also with the increase in porosity for the same confining pressure. The denser the sample, the higher the dilatancy (interlocking among particles) observed.

As can be seen from the typical results of tests on undrained unfrozen sands (Table 6 and Figure 3), the shear stress increases up to a strain level of 6.67 percent (Test US10) and then decreases slightly. The residual stress at a strain level of 20 percent is more than 90 percent of the maximum strength. The pore-water pressure initially displays a small increase at a small strain level of less than 1 percent and then decreases as the sand particle skeleton tends to dilate. This is typical behavior for dilatant soils. The current experimental results (*2*) support the results obtained by Atkinson and Bransby (*11*) on medium dense (e = 0.75) brasted sand tested at a confining pressure (σ_3) of 73 kPa (0.744 kg/cm^2). It should be mentioned that the particle size and shape (round or angular), as well as the arrangement of the sand particles inside the sample (sand structure), also affect stresses and pore-water pressures.

COMPARATIVE STUDY BETWEEN FROZEN AND UNFROZEN SAND

The changing ground temperature in seasonally frozen geographical areas changes the mechanical behavior of the soil. It is important to know the effect of freezing and thawing of the water in the ground on the shear stress-strain volume change behavior of the soil. This mechanical behavior of the soil provides the basis for design and construction of structures built on seasonally frozen ground.

This section presents a comparative study of frozen and unfrozen sand. Figure 5 shows the shear stress and strain curves for both frozen and water-saturated sands (samples FS8, US6, FS9, and US4). In general, the shear strength of frozen sand is much higher than that of unfrozen sand. Freezing the water-saturated sand, even at a temperature of −5°C, will result in an increase of the shear strength by a factor of more than 2.5 and increase its modulus of

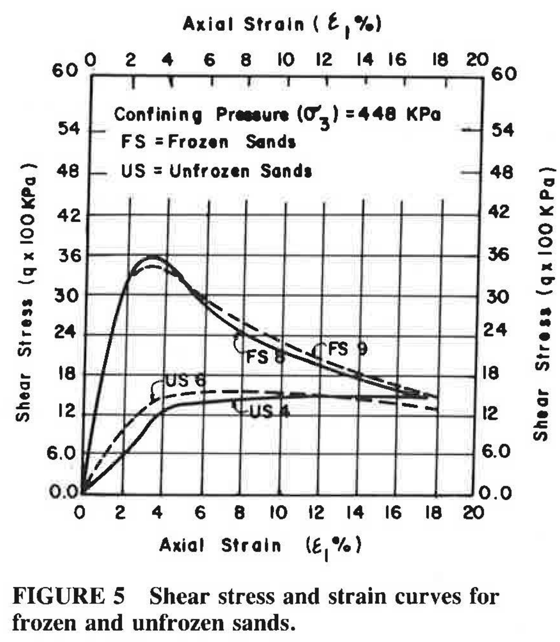

FIGURE 5 Shear stress and strain curves for frozen and unfrozen sands.

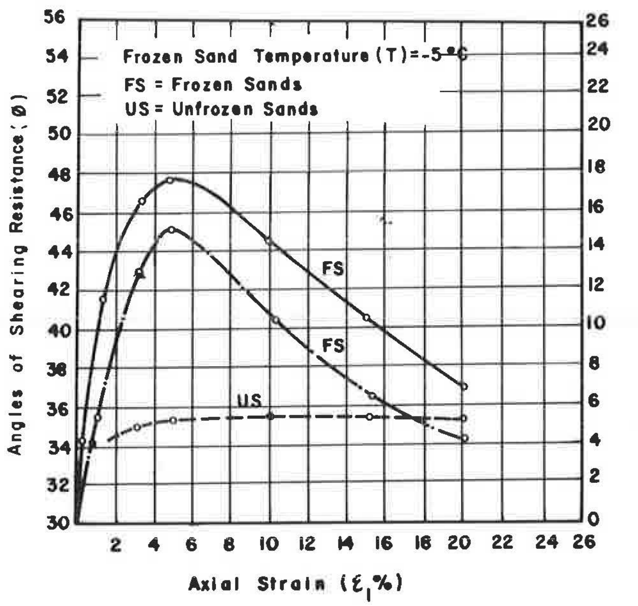

FIGURE 6 Shear strength components of sands versus strain.

elasticity by a factor of about 4. Transformation of the water to its solid state (ice) increases the brittleness of the sand samples.

The residual strength (at 20 percent strain or higher) of frozen sand approaches that of unfrozen soil. This indicates that at higher strain levels (longer time duration of loading), the contribution of the ice matrix to cohesion and friction will decrease to a negligible value. The higher strength of the frozen samples is due to the contribution of the intergranular ice shear strength. Because of the high viscosity of the ice inclusion, the strength of the frozen sample can be made higher by increasing the strain rate during testing.

The variation in the frictional and cohesion components of the shear strength is shown in Figure 6 as a function of strain. The apparent average cohesion of frozen sand (ice cohesion) increases rapidly to 15.30 kg/cm^2 (1.50 MPa) at an average strain level of 4.60 percent and then decreases rapidly to 4.38 kg/cm^2 (0.43 MPa) at a strain level of 20 percent. Because unfrozen sand samples are cohesionless materials, the contribution of the cohesion in the strength of frozen sands is solely due to the intergranular ice, which acts as a cohesive bond.

The variation of the effective angle of friction (ϕ) for frozen and unfrozen sand samples is also shown in Figure 6 for comparison purposes. The friction angle of frozen sand increases to 47.70 degrees and then decreases to 36.93 degrees at a strain level of 20 percent. It could be noted that at higher strain levels, the angle ϕ approaches that for unfrozen sand (ϕ = 35.43 degrees), and the value of cohesion approaches zero. This indicates that at higher strain levels (i.e., with the passing of time) the contribution of the ice matrix to the shear strength for both friction and cohesion components appears to dissipate, and the shear strength of the frozen sand tends to approach the values for unfrozen undrained tests.

Figure 7 shows typical volume change behavior during triaxial testing on frozen and unfrozen sands. It can be seen that the apparent volume change behavior of both frozen and unfrozen sands is similar; it first starts to decrease and then increases progressively until the end of the test. However, the mechanisms of deformation are different for frozen and unfrozen sands.

Very little work has been reported on triaxial testing of frozen soils with volume change (*7*) because of the assumption that frozen soils are tested in closed systems (similar to the undrained tests of unfrozen sands). Furthermore, because of the high viscosity of ice, it is not free to move in or out of the samples during shearing. Goughnour and Andersland (*8*), O'Connor (*12*), and Lade et al. (*1*) have presented data on volume change measurements of frozen soils; however, the mechanism controlling the behavior of this composite material was not explained. On the basis of results of compression and triaxial tests on frozen samples tested with and without a rubber membrane (*2*, *3*), it was concluded that the initial volume decrease is due to the compressibility of both the frozen sample and the air bubbles entrapped between the rubber membrane and the sample (which are very difficult to avoid in testing of frozen soil). In addition, the volume increase is due to initiation and progress of cracks in the frozen soil.

APPLICATIONS

The objectives of this paper are to present the results of an experimental investigation of frozen and unfrozen sands using triaxial equipment and to report on the changes in the mechanical behavior of unfrozen soils caused by freezing. The results of this study demonstrate the advantages of freezing soils for construction purposes. As can be seen from Figure 5, freezing the ground, even at a temperature of −5°C, results in an increase of the shear strength by a factor of more than 2.50. The study also shows (Figure 4) that the strength of frozen sand increases because of the decrease in temperature. In essence, artificially freezing the ground to a very low temperature (−196°C can be achieved by utilizing liquid nitrogen) will sharply increase the strength of the soil to the strength of rock or concrete. Thus, one of the advantages of ground freezing for construction purposes is the ability to use the ground itself (outside the cold regions) as a temporary construction material, for example, in building retaining walls, deep mine shafts, and tunnels. This will simplify the site work, cut construction costs, and reduce construction time.

CONCLUSIONS

This paper describes the changes in shear stress and strain and the volume change of sand in its thawed and frozen states. Emphasis is given to the mechanical behavior of particulate and composite materials during shearing in triaxial apparatus. It was possible to show from the experimental results that freezing water-saturated sand to −5°C

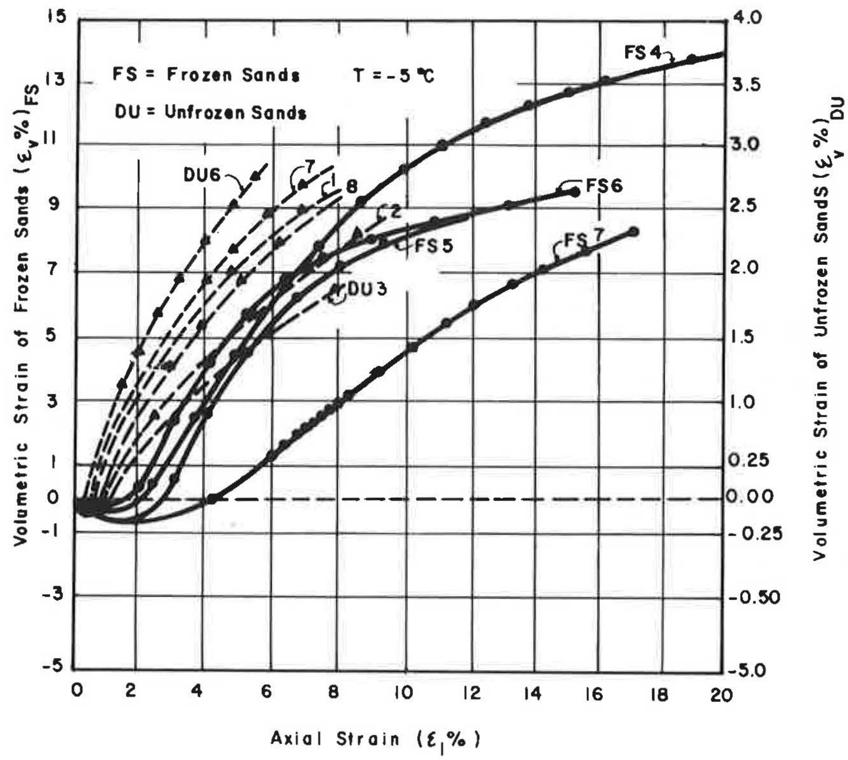

FIGURE 7 Comparison of volume change between frozen and unfrozen sands.

increases its shear stress and modulus of elasticity by factors of 2.50 and 4.00, respectively, compared with unfrozen samples. This increase is time (strain-rate) dependent: the faster the strain rate, the higher the shear strength. This is due mainly to the high viscosity of ice. In addition, as shown by the shear stress and strain curves, freezing the sand causes more brittle behavior. Water-saturated sands compress when subjected to isotropic consolidation, whereas this effect is almost negligible for a sand-and-ice system, especially when subjected to a relatively low confining pressure.

During the shearing stage under triaxial stress conditions, unfrozen (and relatively dense) sands subjected to low confining pressures show initial compression at lower strain levels followed by a volume increase because of dilatancy (interlocking among particles).

Under the same stress conditions, frozen sands exhibit apparent similar volume change behavior during shearing. An initial decrease in volume is observed, followed by progressive volume increases until the end of the test.

From the analysis of these test results, it was possible to illustrate the influence of the voids ratio and the applied confining pressures on the magnitude of the volume increase of water-saturated sands. For the same voids ratio, increasing the confining pressure decreases the dilatancy, and for the same confining pressure, increasing the porosity also decreases the dilatancy. It was possible to separate the ice cohesion component of strength and the frictional component for frozen sands and to explain their development during deformation as a function of strain. Although unfrozen sand is a cohesionless particulate material, the freezing process (even to −5°C) causes it to become a cohesive frictional composite material (sand-ice system). This change is attributed to the cohesion of the ice matrix to the sand particle skeleton.

It is of interest that although the volumetric strain behavior of the thawed and frozen sands is apparently similar, the mechanisms controlling the behavior, especially during the volume increase phase, are different. The results of tests on frozen sand samples showed that the volumetric increase is mainly due to initiation and progressive development of cracks (void gaps) rather than to interlocking among the sand particles. This change is also due to the cohesion and high viscosity of the pore-ice matrix.

In practice, in seasonal frozen (geographical) areas above the frost line, building foundations will be subjected to thaw settlements, which result from changes in the soil behavior from winter to summer, that is, from the frozen to the unfrozen state. Therefore, it is recommended that foundations be constructed below the frost line. In the case of highways, coarse material that is not frost susceptible (i.e., gravels) should be used for the subgrade in order to provide sufficient drainage. In addition, outside the cold regions, the advantages obtained in the soil behavior by artificially freezing the soil (i.e., increasing its shear strength) enable utilization of the ground as a construction material, for example, for deep mine shafts and tunnels.

ACKNOWLEDGMENT

The financial support of the National Science and Engineering Research Council of Canada is gratefully acknowledged.

REFERENCES

1. P. V. Lade, H. L. Jessberger, and N. Dickman. Stress-Strain and Volumetric Behaviour of Frozen Soils. Presented at Second International Symposium on Ground Freezing, Trondheim, Norway, 1980.
2. H. Youssef. Indirect Determination of Intergranular Stresses in Frozen Soils. Ph.D. thesis. Department of Civil Engineering, Ecole Polytechnique, University of Montreal, Quebec, Canada, 1984.
3. H. Youssef. Development of a New Triaxial Cell with Self-Cooling System (TCWSCS) for Testing Ice and Frozen Soils. Presented at Fourth International Symposium on Ground Freezing, Sapporo, Japan, 1985.
4. A. W. Bishop and D. Henkel. *The Measurements of Soil Properties in the Triaxial Test,* 4th ed. Edward Arnold Ltd., London, 1978.
5. J. E. Bowles. *Engineering Properties of Soils and Their Measurements.* McGraw-Hill, New York, 1978.
6. N. F. Massoud. Shear Strength Characteristics of Sands. M.Sc. thesis. Department of Civil Engineering, Concordia University, Montreal, Quebec, Canada, 1981.
7. F. Sayles. *Triaxial Constant Strain Rates and Triaxial Creep Tests on Frozen Ottawa Sand.* Technical Report 253. U.S. Army Engineer Cold Regions Research and Engineering Laboratory, Hanover, N.H., 1974.
8. R. Goughnour and O. Andersland. Mechanical Properties of a Sand-Ice System. *Journal of the Soil Mechanics and Foundation Division,* ASCE, No. SM4, July 1968.
9. E. Chamberlain et al. The Mechanical Behaviour of Frozen Earth Materials Under High Pressure Triaxial Test Conditions. *Géotechnique,* Vol. 22, 1972.
10. V. Parameswaran. Deformation Behaviour and Strength of Frozen Sands. *Canadian Geotechnical Journal,* Vol. 17, 1980.
11. H. J. Atkinson and L. P. Bransby. *The Mechanics of Soils: An Introduction to Critical State Soil Mechanics.* McGraw-Hill, London, 1978.
12. M. J. O'Connor. Triaxial and Plane Strain Experiments on a Frozen Silt. Ph.D. thesis. Queen's University, Kingston, Ontario, Canada, 1975.

Publication of this paper sponsored by Committee on Frost Action.