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Measuring Static and Dynamic Properties of Frozen Silty Soils

Michael D. Furnish

Prepared by
Sandia National Laboratories
Albuquerque, New Mexico 87185 and Livermore, California 94550

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Measuring Static and Dynamic Properties of Frozen Silty Soils

Michael D. Furnish
Instrumentation Development Department

Sandia National Laboratories
P.O. Box 5800
Albuquerque NM 87185-1168

Abstract

A mechanical characterization of frozen silty soils has been conducted to support computer modeling of penetrators. The soils were obtained from the Eilson AFB (Alaska) vicinity. Quasi-static testing with a multiaxial system in a cold room and intermediate strain rate testing with a split Hopkinson pressure bar were conducted. Maximum stresses achieved were slightly above 1 GPa, apparently limiting the observed behavior primarily to elastic compression and pore crushing phenomena. Lower temperatures seem to increase the strength of the material markedly, although not by a simple factor. Lower temperatures and higher strain rates increase the apparent Young's and bulk moduli as well (an increase of ~ a factor of two is observed for strain rate increasing from 0.001 s^{-1} to 800 s^{-1}). The strength also depends strongly on strain rate. Increasing the strain rate from 0.001 s^{-1} to 0.07 s^{-1} increases the strength by a factor of five to ten (to values of order 1 GPa). However, only a small increase in strength is seen as strain rate is increased to $\sim 10^2 - 10^3 \text{ s}^{-1}$. The reliability of the strength measurements at strain rates $< 1 \text{ s}^{-1}$ is decreased due to details of the experimental geometry, although general trends are observable. A recipe is provided for a simulant soil based on bentonite, sand, clay-rich soil and water to fit the ~6% air-filled porosity, density and water content of the Alaska soils, based on benchtop mixing and jacketed compression testing of candidate mixes.

Acknowledgments

Several people should be mentioned as key to the completion of this project. Erland Schulson and Eric Gratz of Dartmouth University, who performed the quasistatic studies summarized in Section 3, also suggested various possible ties to ice physics which may prove increasingly useful with further work toward understanding frozen soils. Holly Dockery and Roger Everett worked heroically to obtain the needed core in Alaska for this work (even though on the February day the core was actually acquired Fairbanks was warmer than Albuquerque). Ned Hansen's was an important guiding hand at various stages of the project. Pat Black of the Army Cold Regions Lab (NH) received the cores, provided cold storage and generally kept a helpful interest in the project. Bill Olsson (Sandia) provided and operated the Hopkinson bar for the experiments in Section 4, also generating the interpretive plots of Appendix B. Finally, Paul Yarrington and Wolfgang Wawersik had many helpful suggestions for making this work practical and for possible future work.

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Table of Contents

1.0	Introduction.....	7
2.0	Sample Descriptions and Sources.....	8
3.0	Summary of Quasi-Static Tests	9
3.1	Summary.....	9
3.2	Experimental Method.....	9
3.3	Results.....	10
4.0	Split Hopkinson Pressure Bar Tests.....	16
4.1	Summary of Important Results	16
4.2	Experiments	16
4.3	Results.....	18
	References	20
	Appendix A: Complete Table of Results of Quasi-Static Tests	21
	Appendix B: Split Hopkinson Pressure Bar Plots	23
	Appendix C: Representative Literature About Frozen Soils	32
	Appendix D: Proxy (Artificial) Frozen Soils.....	37
	D.1 Criteria to Match.....	37
	D.2 Materials Readily Available.....	37
	D.3 Detailed Studies of Best Candidates	38
	Distribution	41

Figures

3.1	Schematic geometry of press used for quasi-static tests on frozen soils.	10
3.2	Results for Test 12 ($\sigma_{11}:\sigma_{22}:\sigma_{33} = 1:0.42:0.42$, $T=-10^{\circ}\text{C}$, and $\dot{\epsilon}_{11} = 0.001 \text{ s}^{-1}$	11
3.3	Summary of strength measurements from quasistatic tests on frozen soils	14
4.1	Drawings of 4340 steel jackets used in tests 4 - 7	17
4.2	Figure 4.2. Drawings of 6061-T6 Al jackets used in tests 8 - 9	18
B.1	SHPB Sample #5, uniaxial strain ($\dot{\epsilon}$, σ vs. t).....	23
B.2	SHPB Sample #5, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	23

B.3	SHPB Sample #5 Reload, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)	24
B.4	SHPB Sample #5 Reload, uniaxial strain (σ vs. ϵ - detail)	24
B.5	SHPB Sample #5, Reload, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	25
B.6	SHPB Sample #6, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)	25
B.7	SHPB Sample #6, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	26
B.8	SHPB Sample #6, uniaxial strain (σ vs. ϵ - detail)	26
B.9	SHPB Sample #7, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)	27
B.10	SHPB Sample #7, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	27
B.11	SHPB Sample #8, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)	28
B.12	SHPB Sample #8, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	28
B.13	SHPB Sample #8, raw strain gauge readings	29
B.14	SHPB Sample #8, Hoop strain vs. axial strain	29
B.15	SHPB Sample #9, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)	30
B.16	SHPB Sample #9, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)	30
B.17	SHPB Sample #9, uniaxial strain (hoop strain vs. axial strain)	31
B.18	SHPB Sample #9, Hoop strain vs. axial strain	31

Tables

3.1	Parameters and yield point data for quasi-static triaxial experiments	12
3.2	Pressure vs. deviatoric stress for quasi-static tests at yield point	13
3.3	Apparent Moduli	15
3.4	Yield Strengths	15
4.1	Sample parameters for SHPB tests	16
4.2	Moduli inferred from experiments	19
4.3	Residual strains in fully confined experiments	19
B.1	Results of quasi-static triaxial experiments	21
D.1	Preliminary soil plus water properties	38
D.2	Compaction of samples after cycling to 1.5 kb stress	39
D.3	Recipe for mixing model frozen soil	40

Measuring Static and Dynamic Properties of Frozen Silty Soils

1.0 Introduction

Successful computer modeling of penetration events in frozen soils at impact velocities to a few hundred m/s appears to depend largely on correctly representing the following:

- Material hydrostat
- Material strength

over a strain rate range of $\sim 1 - 10^3$. Various studies have been conducted on artificial frozen soils (generally, combinations of silt or sand and water). Strengths of these mixtures (typically ~ 1 kb - see Appendix D) are probably too high for two reasons:

- Clays and organics are not included, and
- Obtaining a controlled air-filled porosity in an artificial wet soil is difficult.

Therefore, the present study is intended to provide data useful for modeling penetration events in natural frozen soils via testing field core samples from a representative site (a military range near Fairbanks, Alaska).

A summary of previous work is presented in Appendix C.

In the present study, samples are prepared from cores of frozen soil and tested in a Multi-Axis Testing System (MATS) at strain rates of 0.001 s^{-1} to 0.07 s^{-1} , and in a Split Hopkinson Pressure Bar (SHPB) at strain rates of 500 s^{-1} to 1400 s^{-1} . Stress levels range to slightly over 0.1 GPa. Information sought includes compressibility (hydrostat) and strength (yield) properties.

2.0 Sample Descriptions and Sources

All samples were extracted from a set of cores acquired from the Alaska test site in early February, 1997, mostly from near the FTU11-3 site (last few samples from FTU-11-2), in the Yukon Test Range at Eielson Air Force Base, approximately 31 km east of Fairbanks, Alaska. Cores were packaged and frozen solidly, then shipped by express courier in large coolers to the U.S. Army Cold Regions Research and Engineering Laboratory, where they were stored at -20°C .

In general, the soils cored represent unconsolidated, clay/silt (loess) types, nearly water saturated. Although many soil areas included substantial grain size variations (up to cobbles), the test area chosen was much more uniform. The upper 15-20 cm often contained wood debris in various concentrations. Occasional ice lenses were found at or below the purported permafrost horizon (thicknesses to several mm), although detailed cataloguing was done only after the cores were frozen solidly and it was no longer possible to identify the permafrost horizon by identifying partial thawing of the overlying material.

Air-filled porosities were not measured in the field, but (as will be discussed later) are estimated at $\sim 6\%$ from both the SHPB and QS experiments. Densities centered around 1.68 gm/cc . Water comprised about 22 wt% of the samples. These data apply to the cores used for mechanical testing, and may not represent all of the cores taken.

Cores acquired near the FTU11-3 site were comprised of approximately 1 meter of frozen soil overlying partially frozen material. The depth to permafrost was greater than 11 m and was not reached in this coring. These cores were taken in an open area (forest had been cleared some years earlier), and the permafrost horizon (below which the soil was permanently frozen) was significantly deeper than in the FT11-2 and FT11-4 areas, where forest clearing had occurred only recently. The temperature range was narrow: -0.75 to -2°C in the first meter, exclusive of the first 25 cm (the depth zone subject to significant daily temperature fluctuations).

By contrast, cores acquired near the FTU11-2 site showed no thawed soil; the seasonally frozen material contacted the permafrost. The nearby FTU11-4 site showed approximately 1 m of seasonally frozen soil overlying 1 m of partially thawed material, in turn overlying an unknown (more than 4 m) depth of permafrost.

The samples machined easily. For example, preparation of cubical samples for the quasistatic experiments was performed on a standard mill in a cold room, using a carbide-tipped fly-cutter. Negligible plucking was observed. Some slight surface drying occurred during this process, indicated by a color change to a lighter shade of brown. All prepared samples were tightly wrapped in plastic wrap, then placed in Ziploc-type bags.

3.0 Summary of Quasi-Static Tests

The present chapter is a brief summary and interpretation of a work documented fully in Gratz and Schulson (1996), and conducted in support of the present project. For complete details, the reader is referred to that reference.

3.1 Summary

Objectives of this study were as follows:

- Assess failure envelope at 3 temperatures (-1, -5 and -10°C), using a proportional-stress loading (see below).
- Assess hydrostatic compressibility to the highest stress level available,
- Assess brittle/ductile transitions by looking for failure morphology changes and failure stress level changes with strain rate. This is by analogy with earlier studies by Schulson and coworkers on ice.

Principle conclusions of this study were as follows:

- Yield strength decreases slightly with increasing mean stress, decreases strongly with increasing temperature, and increases with increasing strain rate ($\sigma_Y = C\dot{\epsilon}^m$, with $M \approx 0.26$). Including the negative Clapeyron slope of the ice/water transition, the yield strength behavior at a single strain rate and temperature may approximate the von Mises description.

The yield strength from these tests, however, is not quantitatively valid because of problems with unsupported sample edges (see below)

- The apparent Young's modulus increases with increasing strain rate and decreasing temperature (the latter suggesting that thermally activated deformation mechanisms are important in this material). The bulk modulus behaves similarly.
- Irreversible compaction ranged up to 8%, however, was not always easy to measure.
- Deformation after yield is characterized by strain hardening.

3.2 Experimental method

A set of 36 triaxial compression tests were performed at the Ice Research Laboratory, Thayer School of Engineering on cubes of frozen soil cut from the Eielson AFB cores. These tests were conducted using a true triaxial testing system which applied confining loads proportional to a master load. The triaxial testing machinery is housed in a cold room, obviating problems of separately cooling the sample and contacting platens. The geometry is indicated in Figure 3.1.

These tests were performed by straining the soil a fixed amount (usually between $\epsilon_{11} = 0.03$ and $\epsilon_{11} = 0.04$), and then unloading, while monitoring ram thrust required. Variables fixed for a given experiment included strain rate (0.001 s^{-1} to 0.07 s^{-1}), temperatures (-10°C, -5°C and -1°C), and confining ratio (zero to unity). The confining ratio, expressed as $r_1:r_2:r_3$, represents the ratio $\sigma_{11}:\sigma_{22}:\sigma_{33}$.

Figure 3.1. Schematic geometry of press used for quasi-static tests on frozen soils. Rams and plattens along normal to page are omitted for clarity.

3.3 Results

Under all test conditions, ductile behavior was observed. Both the apparent Young's modulus and the bulk modulus varied from around 0.3 GPa to 1.6 GPa and increased with decreasing temperature and increasing strain rate. The yield strength varied from 1.5 MPa to 45 MPa and increased with decreasing temperature, increasing strain rate and increasing confinement. The thermal and rate sensitivities are similar to those for ice. Strain hardening was observed for all tests except for those conducted under uniaxial and near-hydrostatic loading. Reductions in volume of up to 8% were measured just before unloading, followed by volumetric growth (in some cases back to the initial volume) upon post-test storage at -10°C . When account is taken of pressure melting of the ice phase, the materials appears to exhibit von Mises flow behavior.

Most test curves exhibited a compression to yield, followed by a slower buildup of stresses with ram motion, to a preselected strain of 0.03 to 0.04, at which point ram direction was reversed and stresses monitored during decompression. This is illustrated in Figure 3.2.

Experiment parameters, together with stress-strain conditions at the "yield" point, are given in Table 3.1 (a more comprehensive table is contained in Appendix B).

While this configuration affords true triaxial testing (3 independent axes), it has the significant disadvantage that the sample edges are unsupported. Under sample compression, sample leakage into the voids between plattens was observed to form "ears" with volumes of up to several percent of the original sample, which is comparable to the irreversible volume losses noted in these experiments.

To address this difficulty, "bandages" or reinforcements of 2 mil (0.051 mm) thick brass shim stock were applied to the edges (details in Gratz and Schulson, 1996). In addition, the sample size was reduced somewhat (from 53.3 mm edge length to 50.8 mm, where the platten faces were 48.3 mm edge length). The brass edge reinforcements reduced the volume of the "ears," but did not eliminate ear formation entirely.

Figure 3.3 summarizes strength measurements for this material (these are also tabulated in Table 3.2). These are subject to the caveat that the yielding of the unsupported or partially

Figure 3.2. Results for Test 12, in which $\sigma_{11}:\sigma_{22}:\sigma_{33} = 1:0.42:0.42$, $T = -10^\circ\text{C}$, and $\dot{\epsilon}_{11} = 0.001 \text{ s}^{-1}$. This test is unique; no sample leakage between the platens was observed.

Table 3.1. Parameters and yield point data for quasi-static triaxial experiments. Sample intervals extend to ± 3 cm from position noted.

Test	T °C	Edge Width (mil)	Strain Rate (s ⁻¹)	r ₁	r ₂	r ₃	σ_1 (GPa)	$\sigma_{\max}$ - $\sigma_{\min}$ (GPa)	ϵ_{11}	ϵ_{22}	ϵ_{33}	Core Desig & Position (cm)
				$(\sigma_{11}:\sigma_{22}:\sigma_{33})$					$\times 10^{-3}$			
1	-10	0	0.001	1	0.99	1.00	19.998	0.0	8	5	8	FTU11-3T2 45
2	-10	0	0.010	1	0.95	0.99	11.759	0.6	5	3	9	FTU11-3 33
3	-10	0	0.050	1	0.88	0.94	37.596	4.8	8	2	6	FTU11-3T2 51
4	-10	0	0.050	1	0.88	0.94	37.596	4.8	11	3	7	FTU11-3T2 39
5	-10	0	0.001	1	1.00	0.99	19.931	0.2	7	7	6	FTU11-3T2 57
6	-10	8	0.001	1	0.99	1.00	55.661	1.0	27	26	24	FTU11-3T2 65
7	-10	0	0.070	1	0.00	0.00	4.999	15.0	8	0	0	FTU11-3T2 71
9	-10	8	0.001	1	0.99	1.00	79.659	1.0	23	18	24	FTU11-3T2 89
11	-10	3	0.002	1	0.45	0.45	5.499	5.2	13	1	-1	FTU11-3T2 27
12	-10	3	0.070	1	0.52	0.46	15.665	14.0	15	-1	1	FTU11-3T2 21
13	-10	0	0.002	1	0.00	0.00	1.567	4.7	8	-	-	FTU11-3T2 15
14	-10	3	0.001	1	0.80	0.80	14.998	3.0	12	4	4	FTU11-3 48
15	-10	3	0.070	1	0.79	0.81	34.330	11.0	17	1	7	FTU11-3 54
16	-10	3	0.001	1	0.90	0.91	15.665	2.0	12	7	10	FTU11-3 60
17	-10	3	0.070	1	0.87	0.90	40.663	10.0	16	1	10	FTU11-3 66
18	-10	8	0.070	1	0.78	0.80	37.663	12.0	26	1	11	FTU11-3 72
19	-10	8	0.070	1	0.83	0.89	35.996	9.0	25	4	15	FTU11-3 78
20	-1	0	0.002	1	0.00	0.00	0.533	1.6	8	-	-	FTU11-3 99
21	-1	3	0.002	1	0.80	0.81	3.400	0.9	8	3	4	FTU11-3 27
22	-1	3	0.015	1	0.44	0.47	1.467	1.3	8	0	-1	FTU11-3 21
23	-1	3	0.056	1	0.00	0.00	1.567	4.7	9	-	-	FTU11-3 15
24	-1	3	0.001	1	1.00	1.00	21.998	0.0	14	16	19	FTU11-3T 65
25	-1	3	0.070	1	0.96	0.97	37.663	2.0	20	9	11	FTU11-3T 71
26	-1	3	0.050	1	0.81	0.81	10.232	2.7	9	1	3	FTU11-3T 77
27	-1	3	0.050	1	0.90	0.90	12.665	2.0	9	3	4	FTU11-3T 83
28	-1	3	0.002	1	0.90	0.90	4.666	0.5	6	4	4	FTU11-3T 89
29	-1	3	0.001	1	1.00	1.00	20.965	0.1	14	15	15	FTU11-3T 95
30	-1	3	0.050	1	0.46	0.45	2.333	2.2	6	0	0	FTU11-3T 30
31	-5	3	0.060	1	0.89	0.89	22.664	4.0	14	6	9	FTU11-3T 24
32	-5	3	0.070	1	0.44	0.45	10.932	10.6	16	2	1	FTU11-3T 18
34	-5	3	0.060	1	0.43	0.43	6.233	5.7	13	1	3	FTU11-2a 55
35	-5	3	0.060	1	0.98	0.99	27.664	2.0	15	10	18	FTU11-2a 49
36	-5	3	0.070	1	0.79	0.80	23.998	6.0	16	4	5	FTU11-2a 43
37	-5	3	0.060	1	0.00	0.00	4.333	13.0	12	-	-	FTU11-2a 37

supported sample edges may affect (or effect) the bulk yield process in these measurements. Several trends may be noted. For all of these plots, the shear stress S may be converted to similar quantities via:

$$S = (2/3)^{1/2} (\sigma_{\max} - \sigma_{\min}) = 2(2/3)^{1/2} \tau_{\max} = (2/3)^{1/2} Y = (2J_2')^{1/2}$$

First, lower temperatures seem to increase the strength of the material (see two upper plots in Fig. 3.3). This strength increase is marked. However, there does not appear to be a simple factor by which the strength is increased with a set drop in temperature.

Second, the strength depends strongly on strain rate. Increasing the strain rate from 0.001 s^{-1} to 0.07 s^{-1} increased the strength by a factor of five to ten. However, plotting these results together with results from split Hopkinson pressure bar (SHPB) tests shows only a small increase in strength as strain rate is increased to $\sim 10^2 - 10^3 \text{ s}^{-1}$.

Table 3.2. Pressure vs. deviatoric stress for quasi-static tests at yield point

Test	T	$\dot{\epsilon}$	$\bar{P}$	$\sigma_{\max} - \sigma_{\min}$	Test	T	$\dot{\epsilon}$	$\bar{P}$	$\sigma_{\max} - \sigma_{\min}$
	°C		MPa	MPa		°C		MPa	MPa
1*	-10	0.001	20.0	0.0	20*	-1	0.002	0.5	1.6
2*	-10	0.010	11.8	0.6	21	-1	0.002	3.4	0.9
3*	-10	0.050	37.6	4.8	22	-1	0.015	1.5	1.3
4*	-10	0.050	37.6	4.8	23	-1	0.056	1.6	4.7
5*	-10	0.001	19.9	0.2	24	-1	0.001	22.0	0.0
6	-10	0.001	55.7	1.0	25	-1	0.070	37.7	2.0
7	-10	0.070	5.0	15.0	26	-1	0.050	10.2	2.7
9	-10	0.001	79.7	1.0	27	-1	0.050	12.7	2.0
11	-10	0.002	5.5	5.2	28	-1	0.002	4.7	0.5
12	-10	0.070	15.7	14.0	29	-1	0.001	21.0	0.1
13*	-10	0.002	1.6	4.7	30	-1	0.050	2.3	2.2
14	-10	0.001	15.0	3.0	31	-5	0.060	22.7	4.0
15	-10	0.070	34.3	11.0	32	-5	0.070	10.9	10.6
16	-10	0.001	15.7	2.0	34	-5	0.060	6.2	5.7
17	-10	0.070	40.7	10.0	35	-5	0.060	27.7	2.0
18	-10	0.070	37.7	12.0	36	-5	0.070	24.0	6.0
19	-10	0.070	36.0	9.0	37	-5	0.060	4.3	13.0

*Unsupported sample corners

These conclusions are reiterated in Tables 3.3 and 3.4, which reproduce the summary interpretations of Gratz and Schulson (1996) regarding loading moduli and yield strength.

Frozen Soil Strength
Fits are to all points shown

Bold symbols: Strain rate = $0.07 /s$ or $0.001 /s$
Fine symbols: Strain rate $\sim 0.05 /s$ or $0.002 /s$

Figure 3.3. Summary of strength measurements from quasistatic tests on frozen soils.

Table 3.3. Apparent Moduli
(adapted from Gratz and Schulson, 1996)

Temperature (°C)	Apparent Young's Modulus ¹ E (GPa)		Apparent Bulk Modulus ² K (GPa)	
	(at $\dot{\epsilon}_{11} = 0.001 \text{ s}^{-1}$)	(at $\dot{\epsilon}_{11} = 0.07 \text{ s}^{-1}$)	(at $\dot{\epsilon}_{11} = 0.001 \text{ s}^{-1}$)	(at $\dot{\epsilon}_{11} = 0.07 \text{ s}^{-1}$)
-10	1.2±0.5	1.5±0.5	0.89±0.37	1.6±0.6
-5	---	1.0±0.1	---	0.69±0.2
-1	0.37±0.12	0.77±0.09	0.34±0.1	0.75±0.2

¹ Young's modulus is calculated as $E = (1-2R\nu) d\sigma_{11}/d\epsilon_{11}$ for data below apparent yield; Poisson's ratio assumed $\nu = 0.33$; R = stress ratio.

² Bulk modulus is calculated as $K = 1/3 \Sigma\sigma_{ii}/\Sigma\epsilon_{ii}$; if $\nu = 1/3$, then $K = E/3(1-2\nu)$.

Table 3.4. Yield Strengths¹
(adapted from Gratz and Schulson, 1996)

Temperature (°C)	Strain rate $\dot{\epsilon}_{11} = 0.001 \text{ s}^{-1}$			Strain rate $\dot{\epsilon}_{11} = 0.07 \text{ s}^{-1}$		
	A	B	r^2	A	B	r^2
-10	5.3	0.82	0.98	16	0.85	0.99
-5	---	---	---	11	0.70	0.88
-1	1.6	0.77	0.99	3.7	0.87	0.97

¹A, B are parameters in the yield surface equation $\sigma_{11} = A + B\sigma_{33}$; r^2 is the statistical correlation factor.

4.0 Split Hopkinson Pressure Bar Tests

4.1 Summary of important results:

(1) Compressive moduli (Table 2) are generally ~2 GPa. (2) Elastic moduli are not well constrained. (3) Partial lateral confinement tests only provided lower bounds on the material strength; these are 40 MPa and 21 MPa (compared with values of 13 - 16 MPa for unconfined SHPB tests (samples 1 - 3), 10 GPa (preliminary value from quasistatic work at Dartmouth College (Gratz and Schulson, 1996), using a strain rate of 0.07 s^{-1}), and 3.3 MPa (strain rate of 0.001 s^{-1})². Here, strength is taken as $S = (2/3)^{1/2} (\sigma_{\max} - \sigma_{\min}) = 2(2/3)^{1/2} \tau_{\max} = (2/3)^{1/2} Y = (2J_2')^{1/2}$.

4.2 Experiments

Nine tests were conducted on the frozen soil material acquired from core FTU11-3 (range circa 90 cm depth). The 6" length of core from which these samples were derived was taken approximately 15 feet west of the FTU11-3 site, and is clear of macroscopic organics and has a homogeneous appearance. Tests are shown in Table 1 (where test number and sample number are the same).

Table 4.1. Sample parameters for SHPB tests

(Unconfined)	Length (in)	Diameter (in)	Mass (gm)	ρ (gm/cc)	Peak σ	Strain rate
Sample #1	0.993	0.977	21.21	1.739	20 MPa	530 s^{-1}
Sample #2 (19°F)	1.000	0.975	21.05	1.720	19 MPa	600 s^{-1}
Sample #3	0.998	0.982	21.23	1.714	16 MPa	1050 s^{-1}
(Fully confined)	Length (in)	Diameter (in)	Mass (gm)	ρ (gm/cc)	Peak σ	Strain rate
Sample #4	0.995	0.980	21.32	1.733	41 MPa	1050 s^{-1}
Sample #5	1.003	0.985	21.78	1.739	42 MPa	900 s^{-1}
(Second test on same)	0.912	0.985	21.78	1.912	126 MPa	950 s^{-1}
Sample #6	1.004	0.982	21.65	1.737	60 MPa	1250 s^{-1}
Sample #7	1.000	0.985	21.78	1.744	99 MPa	1400 s^{-1}
(70 MPa Confine)	Length (in)	Diameter (in)	Mass (gm)	ρ (gm/cc)	Peak σ	Strain rate
Sample #8	1.012	0.978	21.93	1.761	65 MPa	1400 s^{-1}
Sample #9	1.011	0.979	22.05	1.768	35 MPa	1400 s^{-1}

Samples 1 - 3 were used in uniaxial failure tests; Olsson (1996) has summarized the results of the corresponding tests by memo. Unconfined peak stresses were 16 - 20 MPa.

Samples 4 - 7 were tested using a heavy confinement jacket of 4340 steel (1" walls) heat treated to RC50 - 55 (giving a tensile yield strength of 230 ksi). The inside finish was specified as number 8 to minimize asperities to provide friction.

The samples were all slightly undersized ($\sim 0.015 - 0.018$ " in radius) relative to the confinement jackets, which were sized to fit around the bars. For Test 4, an attempt was made to fill the gap by using a small amount of water (intended to freeze on contact with the sample). Even using iced water, it proved difficult to do this operation without causing a minor amount of sample melting at the sample surface, leading to uneven surfaces in contact with the pressure bars. This sample was tested at approximately freezing (with results discussed by Olsson - reference on preceding page); all other tests in this series were performed in the range -5 to -10°C .

Prior to Test 5, a small extension piece ("anvil") was removed from the end of the transmitted-wave bar and the associated attachment point machined off. This anvil affected unloading signatures (which were not of interest in Tests 1 - 3).

Test 5 - 7 used the full-confinement steel jackets with 3 layers of 5-mil Teflon tape between the sample and the jacket. The I.D. of the jacket was 1.015 ". 3 layers of tape were used, filling nearly all of the excess diameter. More layers would have been impractical due to the difficulty of forcing the sample into the confinement ring. This tape had two uses: filling space and lubricating the sample-jacket contact. Drawings of the jackets are shown in Figure 4.1.

(Holes are concentric)

Figure 4.1.
Drawings of 4340
steel jackets used in
tests 4 - 7. Jackets
were machined,
heat-treated to
maximum strength,
then the inside was
ground to specs.

In Tests 8 and 9, the sample was confined by a cylinder ($1/8$ " walls) of 6061-T6 aluminum, which was anticipated to yield at a (radial) stress of 70 MPa (using a tensile failure stress of $\sim 46\text{ ksi}$, determined at a strain of 0.7%). Actual hoop strains were $1.6 - 1.8\%$ for the central portions of these jackets after the experiments. However, it was later determined that these nonrecovered strains occurred primarily during the second and third "rings" of the bar; radial stresses in the initial compression-release cycle did not cause the jackets to yield; these stresses are calculated as about 20 MPa (tensile hoop stresses of 83 MPa).

Sample compaction during the initial cycle resulted in different material behavior for subsequent cycles.

Drawings of these jackets are shown as Figure 4.2.

Figure 4.2. Drawings of 6061-T6 Al jackets used in tests 8 - 9. No heat treatment was used.

4.3 Results

Plots of the data are shown in Appendix B; these show the axial stress, strain rate, and strain versus time, as well as stress and strain rate versus axial strain, for tests 5-9. For the partial-confinement tests, plots of raw data to greater time are included to illustrate how the second and subsequent load cycles produced the irrecoverable strain in the aluminum sleeve. The following general comments apply:

1. The maximum stress reached starting with virgin core was approximately 1 kb (test 7). A stress of 1.25 kb was obtained in a reloading of sample 5.
2. The aluminum-jacketed tests (8,9) did not produce the anticipated jacket yielding in the first reverberation of the SHPB; results from the first compression-release cycle may be treated as if an elastic sleeve had been used. Since a deviatoric stress was supported by the sample, it is possible to assign a minimum strength. For sample 8, the maximum axial stress (69 MPa) and confining stress (20 MPa) occur at the same time; a strength of > 40 MPa is thus inferred for a pressure of 36 MPa and strain rate of 1400 s^{-1} . For sample 9, however, the maximum axial stress (33 MPa) coincided with a hoop strain of -0.00045 (confining stress of 7.3 MPa), giving a lower bound for strength of 21 MPa (pressure of 16 MPa; strain rate of 1400 s^{-1}).
3. Several attributes of the $\sigma_{11}(t)$ plots (axial stress vs. time) are observed. For the fully-confined tests (samples 5, 6, 7), an initial peak of amplitude ~ 0.4 kb is followed by a stress decrease to 0.11 - 0.25 kb, an increase to a value depending on the initial piston velocity, then at least one further oscillation. The aluminum confinement tests gave a lower magnitude for the initial stress peak (0.24 - 0.30 k). The difference may be due to elastic yielding in the aluminum confinement. Similarly, the reloading of sample 5 (under the same bar conditions as the first impact) showed peak stresses roughly double those in the original loading of sample 5 (consistent with the pre-compacted starting condition of this sample).

4. Elastic moduli inferred from the initial stress/strain loading are erratic and probably unreliable; with the data processing method used, they depend strongly on the exact arrival times picked from the waveforms. Values for these tests are shown in Table 4.2. The great variability of values for similar tests (e.g. tests 8 and 9) especially points out the erratic nature and apparent unreliability of these values. The quasistatic tests on this material (temperature of -10°C and strain rates of 0.001 and 0.07 s^{-1}) suggest that values of 1 - 2 GPa are plausible.
5. Compressive moduli obtained from the second peak, shown in Table 2, are far more consistent, and may be usable in modeling. These also are of similar magnitude to the values from the quasistatic results.
6. The strain in the fully confined samples (Tests 4 - 7) inferred by post-test measurement of sample dimensions is consistently about 20 - 30% higher than that calculated from the strain gage data. This is probably due to subsequent reverberations of the pressure pulse in the SHPB. The pertinent values are shown in Table 4.3.

A more detailed discussion of the first 4 tests was presented by Olsson (1996).

Table 4.2. Moduli inferred from experiments

Test (Sample) #	Elastic Modulus (First Peak) (GPa)	Compressive Modulus (Second Peak) (GPa)
----- (Fully confined) -----		
5	3.6	0.8
5 (reload)	15	3.1
6	18	1.9
7	Large	1.8
----- (70 MPa confined) -----		
8	60	1.5
9	2	0.6

Table 4.3. Residual strains in fully confined experiments

Test/Sample #	Initial Density (gm/cc)	Post-test Density (gm/cc)	Strain following First Cycle	Residual Strain
Sample #4	1.733	~1.906	(Not recorded)	~-0.091
Sample #5	1.739	1.912	-0.059	-0.090
(Second test on same)	1.912	1.927	-0.059	-0.008
Sample #6	1.737	1.955	-0.081	-0.111
Sample #7	1.744	1.953	-0.085	-0.107

References

Gratz, Eric T. and Erland M. Schulson, The deformation of cubes of frozen soil loaded under triaxial compression, Ice Research Laboratory (Thayer School of Engineering, Dartmouth College) Final Report, Project AN-2014, 1996.

Olsson, W. A., memo to M. Furnish titled "Data from Kolsky-bar tests on frozen soil," dated 6/17/96.

Appendix A

Complete Table of Results of Quasi-Static Tests

Table B.1 is a more exhaustive tabulation of results from the quasi-static tests than is Table 3.1. Of special interest are the inferred Young's modulus and bulk modulus from the pre-yield portion of each loading curve. Also, the apparent stress-strain points for each of the curves at the turnaround point is given, although the reader is cautioned that these are almost certainly unphysically soft because the extruded material ("ears") was not included in the final cube.

Table B.1. Results of quasi-static triaxial experiments

[Following page]

Test #	T °C	Reinf. width	s ⁻¹	R ₁	R ₂	R ₃	E (GPa)	K (GPa)	σ_{11} ("Yield")	σ_{22} (MPa)	σ_{33} (MPa)	ϵ_{11}	ϵ_{22} (Break)	ϵ_{33}	σ_{11} (Turnaround)	σ_{22} (MPa)	σ_{33} (MPa)	ϵ_{11} (Turnaround)	ϵ_{22}	ϵ_{33}
1	-10	0	0.001	1	0.99	1.00	1.50	1.30	20.0	20.0	20.0	0.008	0.005	0.008	54.7	54.3	54.9	0.040	0.031	0.030
2	-10	0	0.010	1	0.95	0.99	0.90	1.20	12.0	11.4	11.9	0.005	0.003	0.009	41.0	39.0	40.4	0.039	0.025	0.041
3	-10	0	0.050	1	0.88	0.94	1.90	2.60	40.0	35.2	37.6	0.008	0.002	0.006	55.6	55.6	52.3	0.023	0.005	0.016
4	-10	0	0.050	1	0.88	0.94	1.70	2.20	40.0	35.2	37.6	0.011	0.003	0.007	54.3	54.3	51.2	0.022	0.004	0.014
5	-10	0	0.001	1	1.00	0.99	1.80	1.30	20.0	20.0	19.8	0.007	0.007	0.006	67.9	67.6	67.1	0.039	0.041	0.030
6	-10	8	0.001	1	0.99	1.00	0.93	0.72	56.0	56.0	55.0	0.027	0.026	0.024	57.3	56.6	57.3	0.041	0.034	0.038
7	-10	0	0.070	1	0.00	0.00	2.50	0.00	15.0	0.0	0.0	0.008	0.000	0.000	16.5	0.0	0.0	0.120	0.000	0.000
8	-10	3	0.070	1	0.91	0.97	1.10	1.60	0.0	0.0	0.0	0.000	0.000	0.000	76.3	69.7	73.6	0.031	0.006	0.025
9	-10	8	0.001	1	0.99	1.00	2.10	1.20	80.0	79.0	80.0	0.023	0.018	0.024	74.2	74.1	74.2	0.025	0.019	0.027
10	-10	8	0.080	1	0.94	0.99	1.40	2.00	0.0	0.0	0.0	0.000	0.000	0.000	77.9	73.9	77.3	0.025	0.005	0.020
11	-10	3	0.002	1	0.45	0.45	0.53	0.42	8.9	3.7	3.9	0.013	0.001	-0.001	11.9	5.0	5.0	0.034	-0.005	-0.010
12	-10	3	0.070	1	0.52	0.46	1.40	1.10	25.0	11.0	11.0	0.015	-0.001	0.001	36.6	19.1	16.9	0.039	-0.006	-0.011
13	-10	0	0.002	1	0.00	0.00	0.87	0.00	4.7	0.0	0.0	0.008	0.000	0.000	6.3	0.0	0.0	0.159	0.000	0.000
14	-10	3	0.001	1	0.80	0.80	0.85	0.71	17.0	14.0	14.0	0.012	0.004	0.004	30.2	24.1	24.3	0.033	0.002	0.003
15	-10	3	0.070	1	0.79	0.81	1.40	1.40	41.0	30.0	32.0	0.017	0.001	0.007	60.5	47.9	49.1	0.038	0.000	0.007
16	-10	3	0.001	1	0.90	0.91	0.93	0.58	17.0	15.0	15.0	0.012	0.007	0.010	47.6	42.7	43.1	0.031	0.008	0.012
17	-10	3	0.070	1	0.87	0.90	1.90	1.50	46.0	36.0	40.0	0.016	0.001	0.010	72.1	65.6	65.0	0.038	0.002	0.014
18	-10	8	0.070	1	0.78	0.80	1.00	1.00	45.0	33.0	35.0	0.026	0.001	0.011	54.9	42.7	43.8	0.038	0.000	0.011
19	-10	8	0.070	1	0.83	0.89	0.87	0.84	41.0	32.0	35.0	0.025	0.004	0.015	53.6	44.5	47.5	0.039	0.005	0.019
20	-1	-	0.002	1	0.00	0.00	0.31	0.00	1.6	0.0	0.0	0.008	0.000	0.000	1.9	0.0	0.0	0.036	0.000	0.000
21	-1	3	0.002	1	0.80	0.81	0.27	0.23	4.0	3.1	3.1	0.008	0.003	0.004	9.5	7.6	7.7	0.035	0.001	0.001
22	-1	3	0.015	1	0.44	0.47	0.22	0.22	2.3	1.0	1.1	0.008	0.000	-0.001	3.2	1.4	1.5	0.035	-0.008	-0.013
23	-1	3	0.056	1	0.00	0.00	0.72	0.00	4.7	0.0	0.0	0.009	0.000	0.000	5.4	0.0	0.0	0.029	0.000	0.000
24	-1	3	0.001	1	1.00	1.00	0.51	0.44	22.0	22.0	22.0	0.014	0.016	0.019	22.7	22.7	22.8	0.034	0.031	0.035
25	-1	3	0.070	1	0.96	0.97	0.76	0.95	39.0	37.0	37.0	0.020	0.009	0.011	28.6	27.7	28.3	0.029	0.025	0.019
26	-1	3	0.050	1	0.81	0.81	0.79	0.80	12.0	9.3	9.4	0.009	0.001	0.003	18.7	15.1	15.2	0.028	0.001	0.002
27	-1	3	0.050	1	0.90	0.90	0.91	0.83	14.0	12.0	12.0	0.009	0.003	0.004	27.0	24.2	24.3	0.028	0.006	0.006
28	-1	3	0.002	1	0.90	0.90	0.45	0.34	5.0	4.5	4.5	0.006	0.004	0.004	14.8	13.3	13.3	0.034	0.004	0.005
29	-1	3	0.001	1	1.00	1.00	0.51	0.47	21.0	20.9	21.0	0.014	0.015	0.015	19.8	19.7	19.8	0.035	0.032	0.024
30	-1	3	0.050	1	0.46	0.45	0.66	0.41	3.8	1.6	1.6	0.006	0.000	0.000	5.9	2.7	2.7	0.029	-0.003	-0.001
31	-5	3	0.060	1	0.89	0.89	0.91	0.79	25.0	22.0	21.0	0.014	0.006	0.009	37.3	33.0	33.0	0.028	0.010	0.014
32	-5	3	0.070	1	0.44	0.45	0.94	0.61	18.0	7.4	7.4	0.016	0.002	0.001	22.6	10.0	10.3	0.044	-0.006	-0.011
34	-5	3	0.060	1	0.43	0.43	0.91	0.38	10.0	4.3	4.4	0.013	0.001	0.003	13.4	5.7	5.7	0.029	-0.001	0.003
35	-5	3	0.060	1	0.98	0.99	1.00	0.65	29.0	27.0	27.0	0.015	0.010	0.018	50.9	49.9	50.4	0.027	0.018	0.029

Appendix B

Split Hopkinson Pressure Bar Plots

In this section, plots of data derived from the SHPB tests are presented. For interpretation, please see Section 4.

Figure B.1. SHPB Sample #5, uniaxial strain ($\dot{\epsilon}$, σ vs. t)

Figure B.2. SHPB Sample #5, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.3. SHPB Sample #5 Reload, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)

Figure B.4. SHPB Sample #5 Reload, uniaxial strain (σ vs. ϵ - detail)

Figure B.5. SHPB Sample #5, Reload, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.6. SHPB Sample #6, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)

Figure B.7. SHPB Sample #6, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.8. SHPB Sample #6, uniaxial strain (σ vs. ϵ - detail)

Figure B.9. SHPB Sample #7, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)

Figure B.10. SHPB Sample #7, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.11. SHPB Sample #8, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)

Figure B.12. SHPB Sample #8, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.13. SHPB Sample #8, raw strain gauge readings

Figure B.14. SHPB Sample #8, Hoop strain vs. axial strain

Figure B.15. SHPB Sample #9, uniaxial strain ($\dot{\epsilon}$, σ vs. ϵ)

Figure B.16. SHPB Sample #9, uniaxial strain (ϵ , $\dot{\epsilon}$, σ vs. t)

Figure B.17. SHPB Sample #9, uniaxial strain (hoop strain vs. axial strain)

Figure B.18. SHPB Sample #9, Hoop strain vs. axial strain

Appendix C

Representative Literature About Frozen Soils

The following represents a cross-section of the available unrestricted literature pertinent to the mechanical properties of frozen soils under penetration conditions.

Quasi-static characterizations

1. Chamberlain, E., *Mechanical properties of frozen ground under high pressure*, pp. 295 - 305 in Permafrost; North American Contribution, Second International Conference; Physics, Physical Chemistry, and Mechanics of Frozen Ground and Ice, 1973. Note set of ice phase transitions traversed over 0 - 30 kb on Hugoniot (I/water/V/VI/VIII). Isothermal compression used to 25 kb. Triax: shear stress shows 3 regions. I - increasing confining stress increases shear stress more in sand than in till (interparticle lock-ups. II - decreasing shear stress due to ice melting ($\bar{\sigma} = 800 \text{ kg/cm}^2 = 0.8 \text{ kb}$ up to 1.13 kb). III - Gradual increase in shear stress. Expts. at -10°C .
2. Ohrai, T. and S. Takahashi, *Uniaxial compressive strength of ice segregated from soil*, pp. 945 - 950 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Segregated ice prepared from clay samples (Tokyo); distinct axial structure. Load in direction of growth: Uniax compressive strength up as strength down. (18.6 MPa to 24.5 MPa - -10°C to -30°C). Load in perpendicular direction: 4.5 MPa @ -20°C . (~that of commercial ice).
3. Jones, S. K. and V. R. Parameswaran, *Deformation behavior of frozen sand-ice materials under triaxial compression*, pp. 560 - 565 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Plots of peak strength vs. hydrostatic pressure for various amounts of sand in ice (pure ice to saturated sand). Peaks determined by plotting stress/strain at a given hydrostatic pressure P. Peak strength increases until 25 - 40 MPa, then drops toward or past unconfined values. Values of 8 - 36 MPa are maxima.
4. Baker, T. H. W. and G. H. Johnston, *Unconfined compression tests on natural anisotropic frozen soils from Thompson, Manitoba*, pp. 40 -45 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Laminated frozen soils (layers of dk. brown clay and lt. brown silt), with ice lenses between layers. Stronger parallel to layers. ~15% unfrozen content at -1°C . Clay content ~80 - 90%. Yields ~0.75 MPa (axial stress; $1.7 \times 10^{-4} \text{ s}^{-1}$); 1.5 MPa (axial stress; $1.7 \times 10^{-2} \text{ s}^{-1}$).

Ultrasonic, oscillatory characterizations

1. Ping, H., Z. Yuanlin, Z. Jiaya, S. Zhongyan and Y. Qihaao, *Dynamic elastic modulus and strength of saturated frozen silt*, pp. 250 - 254 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Dynamic elastic modulus decreases with increasing strain or temperature; increases with increasing frequency.

Sample values: $E = 2000$ at 2% strain, -10°C (the most important variable) and 1 - 5 Hz. Same at -2°C : $E = 900$ MPa. Dynamic strength (-10°C) drops from 12 MPa (failure time of 6 s) to 4 MPa (failure time > 1 hr).

2. Fukuda, M. and S. L. Huang, Effects of total water content on dynamic properties of frozen soils, SPE 22140, Society of Petroleum Engineers. Issue is actually unfrozen water content. Strengths of frozen fine-grained soils are not as sensitive to confining stresses as are those of sands because of the durable water films in the soils. Ultrasonic velocities are strongly temperature dependent. Adsorption of water may be an important phenomenon in locally depressing freezing temperatures.
3. Vinson, T. S., Parameter effects on dynamic properties of frozen soils, J. Geotech. Eng'rg Div, GT10, pp. 1289 - 1306, October, 1978. Table 5 (p. 1303) is a good summary of which field and material parameters are important under various circumstances. "Dynamic stress-strain properties" refer to mechanical measurables, and are contrasted with "Energy absorbing properties" (phase lag, attenuation and damping coefficients, Q , ...) Items of special interest: Confining pressure may be important for coarse-grained systems and unimportant for fine-grained systems; temperature is critical for fine-grained systems, but not for coarse systems. Also, density (void ratio) is important for stress-strain properties; less so for energy absorption properties. {Many plots are presented.} Note that "Discussion" (Y. S. Chae; GT7; pp. 907 - 8) brings out viscoelasticity description as a framework which could be used to model this.
4. Kaplar, C. W., Laboratory determination of dynamic moduli of frozen soils and ice, US Army Cold Regions Research and Engineering Laboratory Research Report 163, 1969. 12 materials - 8 natural soils with various grain sizes, 2 blended soils, a peat and lake ice; temperatures $-23 - 0^{\circ}\text{C}$. E computed from flexural vibration lower than E from longitudinal. Values for coarse granular soils had E up to $4\times$ times the values for clays. Poisson's ratio variations did not seem systematic. Typical values: $E = 2$ to 4 psi (20°F)
5. Vinson, T. S., C. R. Wilson and P. Bolander, Dynamic properties of naturally frozen silt, pp. 1315 - 1320 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Dynamic moduli (Young's, shear) and damping ratio determined. Confining pressure and water content had little effect. Moduli decrease with strain amplitude and increase with frequency and [decreasing] temperature. Damping ratio acts in opposite way. Geometry of soil structures didn't have strong effect.
6. Rong, F., Z. Jinsheng and H. Zhongjie, Ultrasonic velocity in frozen soil, pp. 311 - 315 in Permafrost, Fourth International Conference, Proceedings, National Academy Press, 1983. Frozen medium sand and clay samples; varying water content and temperature, with ultrasonics (dilatational and shear waves). Coarse samples: velocity increases with water content. Fine samples: velocity decreases, then increases. Aside: It's not clear to me how they're varying water content. Also, their clay results are extremely noisy.
7. Vinson, T. S., C. R. Wilson and P. Bolander, Dynamic properties of naturally frozen silt, pp. 1315 - 1320, Proceedings of the Permafrost Fourth International Conference, Pewe, T. L.; Brown, J. (chairperson; chairperson), 1983. Concentrates on cyclical

loading. Dynamic Young's, shear moduli and damping ratio measured and dependencies on P , T , ω , ε characterized. By far the strongest dependences are on temperature ($-10 - 0^\circ\text{C}$).

Shock wave testing

1. D. B. Larson, G. D. Bearson and J. R. Taylor, *Shock-wave studies of ice and two frozen soils*, pp. 318 - 325 in Permafrost; North American Contribution, Second International Conference; Physics, Physical Chemistry, and Mechanics of Frozen Ground and Ice, 1973. 0 - 1.5 GPa range. West Lebanon glacial till, Ottawa banded sand, directionally frozen (grain contact maintained). Frozen soils loading wave complicated, including roles of Ice I - liquid transition (which is not completed in these experiments). Frozen soil attenuation larger than ice attenuation - continued transformation in initial unloading in soil increases hysteresis. Samples started at $-10 \pm 2^\circ\text{C}$ - note importance of initial temperature.
2. Nakano, Y. and N. Froula, *Sound and shock transmission in frozen soils*, pp. 359-369 in Permafrost; North American Contribution, Second International Conference; Physics, Physical Chemistry, and Mechanics of Frozen Ground and Ice, 1973. *Ultrasonics* - (has good technique comments) Very granular soils don't show hysteresis in T-cycling; loamy or clayey soils do. Also, shear wave transmission expts show that matrix plays a dominant part in shear transmission. **Gas gun impacts** - In saturated Ottawa sand, freezing lowers shock impedance (0 - 25 kb); on 23% sat., it raises it (above a few kb). Precursor amplitudes $\sim 1.4 - 1.6$ kb.
3. Lyakhov, G. M. and G. B. Frash, *Blast waves in frozen soils*, J. Appl. Mech. and Techn. Phys., 24. 811-815, 1983. Spherical blast waves. Temperature and unfrozen characteristics most important. Stress and wave velocities higher, but peak amplitudes shorter in frozen soils than for unfrozen.
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5. Larson, D. B., *Explosive energy coupling in ice and frozen soils*, LLNL report UCRL-78962, 1976. Similar to Larson et al (above), but addition of 50% porous material with tremendous uncoupling (loss of shock energy).
6. Vovk, A. A. et al., *Mechanical properties of frozen soils under dynamic loads*, Soil Mechanics and Foundation Engineering, 17(2), 64-69, 1980. Pile-driver, free-falling weight, with rubber pads to vary loading rate. Clay soil + water, mixed. Loading rate is pertinent in this (CE) region; 4.4x strain rate gives 40 - 50% increase in deformation modulus. Comments on temperature effect: At temperatures within a few degrees of

freezing, plasticity reduces with consolidation and wave speed increases. At lower temperatures, brittleness gives microcrack formation with consolidation, and velocity drops {I'm not sure I buy this completely}.

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Penetration tests

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2. Aitken, G. W., G. W. Swinzow and D. R. Farrell, Projectile and fragment penetration in snow and frozen soil (CRRELL report, 1976). Hanover silt penetration experiments (5.56 mm steel cubes) showed factor of 2 - 3x increase in penetration with thawing of Hanover silt (actually, 3 cm difference). Includes empirical equations for penetration depth. Unconfined compression tests showed yield strength, elastic and plastic moduli (E, Y). Typical values: -25 °C, -10 °C, -3 °C → $Y_0 = 43 \text{ MPa}$, 23 MPa, 15 MPa. $E = 2.9 \text{ GPa}$, 1.5 GPa, 0.9 GPa (respectively). Also Alaska field tests to assess options for hardening snow trenches.
3. Thompson, L. J., G. H. Ferguson III, J. D. Murff and A. Cetiner, The effect of soil parameters on earth penetration of projectiles, Texas A&M Report, Project 596-1, 1969. Much historical background for EPWs. 8 sets of data, including acceleration curves. Variables affecting acceleration were instantaneous velocity, depth, bulk density and saturation (of soil).
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5. Koshelev, E. A., Penetration of an axisymmetric conical hammer into frozen soil,

Soviet Mining Science, 23, 227-232, 1988. - Pointed axisymmetric hammer into frozen soil. Plastic state; internal Mises-Pleicher friction (internal friction, soil cohesion enter). Shear strength is 2.3 - 3.3 MPa (frozen sand) and ~3 MPa (loam)

6. Conlon, C. M., Remotely monitored battlefield sensor system. TDV (Terminal Delivery Vehicle) penetration tests into frozen sand, Avco Corp. report for Picatinny Arsenal, AVSD-0293-73-RR, 1973. TDV tests (200, 400, 600 fps) and simulations (an AVCO code) were compared and found to be in good agreement.

Penetration calculations

1. Bjork, R. L. (Pacifica Technology), Calculations of earth penetrators impacting soils, DNA report DNA 4189F, 1975. Rigid EPW's (5' long, 6.5" dia., ~conical) in 2 soils at 500, 3500 fps. For first of 5 modelings, friction opposes progress ($|t| = \text{Min} \{0.6\sigma_n, 0.6Y(\sigma_n/\sqrt{3})\}$). Very coarse cells. Soil seemed to behave as elastic-ideally plastic with a nonassociative flow rule (not with associative flow rule). Loading, unloading moduli used in model (which is quite hysteretic, good to ~1 ksi), as well as $\sigma_1 - \sigma_3$. Tests presumably used normal incidence.
2. Wagner, M. H., C. C. Fulton and K. N. Kreyenhagen, Parametric study of the effects of target properties, projectile design and impact conditions on earth penetration processes, California Research and Technology Report, DNA4160T, 1976. Finite-difference code modeling of penetrations events. Most important soil parameters were local shear strength and frictional characteristics; constitutive properties of target were generally less important.
3. Conlon, Charles M. Jr., TDV penetration tests into frozen sand, Picatinny Arsenal report AVSD0293-73-RR, 1973. Experimental and modeling. Experimental variables: Velocity (200, 400, 600 fps). Analytical added calculations for 0, 15, 45 degrees off normal impact, slight variations in angle of attack. Scaled TDV is 3: dia., 18" long. Frozen sand, density ~1.872 g/cc.

General References

1. Recent developments in mechanical testing (ASTM STP 608)
2. Soil mechanics, R. F. Craig, Van Nostrand Reinhold, 1987.
3. Geotechnical engineering for cold regions, O. B. Andersland and D. M. Anderson, McGraw-Hill, 1978.
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Appendix D

Proxy (Artificial) Frozen Soils

The purpose of the present section is to provide an evaluation of possible materials to use in scale penetrator tests, and provide a suggested recipe for modeling the Eielson AFB frozen soils.

D.1 Criteria to Match

From the preceding sections' characterization of the frozen soil samples from the Yukon Test Range at Eielson Air Force Base area, it is apparent that a model soil should have the following attributes:

- Air-filled porosity ~5 - 6%
- Density ~1.6 - 1.8 gm/cc
- A silty texture (not dominately clay; not a coarse sand)
- Approximately 18 - 25 wt% water.

D.2 Materials Readily Available

Four materials were obtained for this exercise. Note that the last two sources are no longer available.

- Wyoming bentonite (an expansive clay)
- Western Mobile engineered fill. Silt, aggregate passing 3/8" screen. Low clay content. Taken from Placitas site (approx. 30 km north of Albuquerque).
- I40 Sand/coarse silt. Taken from under edge of I40 operating roadway ~1/4 mile west of Rio Grande Blvd., Albuquerque. The lead engineer on the I40 widening project felt this material was very similar to material throughout the "I40 Upgrade" project except in clay lenses. Clay lens material was also collected, but was not used due to difficult of making measurable shapes. Using material such as this would require an "as received" moisture content measurement.
- 962 fill. Taken from piles of fill to north of Bldg. 962, Sandia Laboratories, dumped there approximately mid-January, 1997. Material is a mixture of clay (dirt clods) and silt, with some rocky content as well.

A preliminary look at the density properties of these materials is summarized in Table 5.1. The water content was chosen rather arbitrarily for these samples, but certain information was available from these tests:

- Mixing pure bentonite and water produced far too low a density. Had less water been used, it would have been nearly impossible to uniformly distribute the water.
- The Western Mobile engineered fill was too dense; adding more water to give 22 wt% porosity was not expected to lower the density to the range required. As well, the

aggregate component was expected to pose problems.

- The I40 Sandy and the Building 962 Fill appeared to be in the desired density range.
- The engineered fill and the I40 sand both appear to have “dial-a-saturation” properties, i.e. we can control air-filled porosity via the amount of water added. The samples prepared above appear to have comparable “wetness” to the frozen soils from the Eilson site.

For these reasons, the I40 sandy and the Building 962 Fill were chosen for study in mixture with water and bentonite.

Table D.1. Preliminary soil plus water properties

Sample	Dry Wt (gm) (200 cc samples)	ρ_{Dry} (gm/cc)	Vol. water added (cc) to 100 cc samples	Frozen Vol. (cc)	Weight after frozen vol. meas.	ρ_{Frozen} (gm/cc)
WM-1 ^a	254.9 gm	1.2745	11.5	61.2	137 gm	2.24
I40 Sandy ^b	237.5 gm	1.1875	10.0	67.25	127 gm	1.89
962 fill ^c	259.8 gm	1.299	10.0	70.78	136 gm	1.92
Bentonite ^d	160.5 gm	0.8025	30.0 (to 40 cc sample)	54.5	60.6	1.11

a. Western Mobile engineered fill.

b. I40 Sand/coarse silt.

c. Taken from piles of fill to north of Bldg. 962.

d. Quoted density is for sample with “poorly mixed” bottom section removed. I feel that this is maybe 10% low due to void issues. To reach desired texture throughout sample, additional water would probably be required (~35 - 40 cc).

D.3 Detailed Studies of Best Candidates

Samples were assembled as follows:

- 10, 30, 50, 70 wt% bentonite in dry mix with I40 sandy silt, then added water to 22 wt% of total
- 10, 30 wt% bentonite in dry mix with 962 fill, then added water to 22 wt% of total

Densities of these were as follows:

10 wt% bentonite + I40 sandy silt 1.63 gm/cc (some voids)
 30 wt% bentonite + I40 sandy silt 1.49 gm/cc
 50 wt% bentonite + I40 sandy silt 1.41 gm/cc
 10 wt% bentonite + 962 fill 1.76 gm/cc (underestimate; poor sample)
 30 wt% bentonite + 962 fill 1.44 gm/cc

These samples were prepared in styrofoam cups, in which adequate mixing and packing of several of these samples was difficult due to the weak cup bottoms and walls. Hence these density measurements are probably somewhat low.

Following these measurements, five samples were mixed and tamped into heavy steel sleeves (1" inside diameter, 2" long, 1" walls of heat-treated 4340 steel (Figure 4.1). The exposed surfaces were pressed flat with custom-machined Teflon[®] pistons.

These samples were then frozen overnight in sealed plastic bags (to retain moisture). After freezing, the exposed surfaces were found to have bulged slightly due to water expansion. Sample dimensions (including the bulge) were carefully measured.

To estimate the air-filled porosity of the samples, they were compressed in an MTS loading frame to ~ 1.5 kb (using steel piston extenders just under 1" diameter.), with force vs. displacement curves monitored. A constant piston motion was used. A small amount of sample extruded during this process, all of which was collected and weighed. The (essentially perfectly cylindrical) samples were removed from the steel sleeves, and weights and dimensions were measured. Post-test densities were calculated. Comparing with pre-test densities (calculated from the initial volume and the total initial mass = final mass + extruded mass) gave the pore volume loss during the compression cycle.

Results of this testing are summarized in Table 2. All samples were initially 22.5 to 26.7 mm long, and were compressed to 20.9 to 22.6 mm during the experiment. The fraction of the total sample extruded is shown for reference.

Table D.2. Compaction of samples after cycling to 1.5 kb stress

Name ^a	ρ_{Initial} (gm/cc)	ρ_{Final} (gm/cc)	Pore Loss (Fraction of V_{Initial})	Fraction of Material Extruded ^b
I40S-1	1.5871	1.8950	16.2%	8.9%
I40S-2	1.7525	1.9665	10.9%	16.9%
962S-1	1.6449	1.8073	9.0%	9.2%
962S-1	1.6877	1.8260	7.6%	8.6%
I40C-1 ^c	1.5400	1.8855	22.4%	33.7%

a. I40S is I40 Sandy; 962S is Building 962 silt; I40C is I40 clay (no bentonite added)

b. Not necessary for interpretation of pore loss or density data

c. Sample probably not completely frozen when tested (1 hour cooling time)

It is inferred from these results that the best match for the Wainwright soils among the samples tested is probably the Building 962 soils, combined with bentonite to make a dry

mixture 10 wt% bentonite, then mixed with water to give an ultimate water fraction of 22 wt%. This match is based on the density and air-filled porosity values mentioned in Section 1 above.

D.4 Recipes for mixing this material

The Building 962 silt has an as-dumped density of roughly 1.36 gm/cc (if many clay chunks are included, it increases to ~ 1.43 gm/cc; with no clay chunks included, it decreases to ~1.29 gm/cc). Dry bentonite has a density of ~0.88 gm/cc.

Three versions of the recipe are shown in Table 5.3, depending on the quantity of final frozen soils required. Note that this material is quite viscous (“sand castle test”), and will probably require tamping to eliminate large void spaces. Water is not expected to segregate.

Table D.3. Recipe for mixing model frozen soil

Yards of frozen soil desired	50# bags of bentonite	# of dirt (ft ³ , yd ³)	# water (gallons, ft ³)
0.22	1 (50#)	450# (5.3 ft ³ , 0.20 yd ³)	141# (16.9 gallons, 2.26 ft ³)
1.0	4.5 (223#)	2,004# (23.6 ft ³ , 0.87 yd ³)	628# (75.2 gallons, 10.0 ft ³)
3.2	14.3 (714#)	6413# (74.4 ft ³ , 2.80 yd ³)	2010# (240 gallons; 32.2 ft ³)

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External:

Michael Lilly (3)
Arctic Region Supercomputing Center
University of Alaska Fairbanks
Fairbanks, Alaska 99775

U.S.A.C.E. Cold Regions Research and Engineering Laboratory
72 Lyme Road
Hanover, NH 03755
Attn:
Pat Black
George Blaisdell
Piyush Dutta
Dan Lawson

Thayer School of Engineering
Dartmouth College
Cummings Hall 8000
Hanover, NH 03755
Attn:
Dr. Erland Schulson
Dr. Eric Gratz