Temperature Effects on the Unconfined Shear Strength of Saturated, Cohesive Soil

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Temperature-controlled, undrained and unconfined compression tests were conducted on reconstituted clay. The reconstituted clay studied, one-half Champion and one-half Challenger (known by its trade name as C & C), had previously demonstrated a linear relationship between the logarithm of shear strength and moisture content at room temperature. The experiments were aimed toward studying the effect of temperature on this relationship.

The samples were consolidated at room temperature and were failed in undrained, unconfined compression at temperatures of 75, 100, 125, and 150 F. The results indicate that, for a soil of constant moisture content, an increase in temperature causes a reduction in shear strength, and for the same temperature difference, the absolute reduction in the original strength increases as the initial moisture content of the soil decreases. The relation between the logarithm of shear strength and moisture content appears to be linear for constant temperature over the range of temperatures tested.

Two different equations describing the shear strength of the clay as a function of both moisture content and temperature are proposed. The first relates an increase in temperature to an equivalent increase in moisture content by an experimentally determined factor. The second relates the decrease in strength to the ratio of absolute reference and test temperatures and to the densities of the pore fluid corresponding to these temperatures.

•IN 1936 Hogentogler and Willis (4) found that the stability or strength of a compacted cohesive soil decreases with increasing temperature. Their explanation of this phenomenon was based on Winterkorn and Baver's (16) concept, which postulated an increase in free water at the expense of adsorbed water with rising temperatures. In 1958 Lambe (7) believed that an increase in temperature should result in an increase in shear strength.

Leonard (8) believed that an increase in temperature should cause a reduction in shear strength and supported his point of view by citing the work of Hogentogler and Willis (4), Rosenqvist (12), and Trask and Close (15), which show that an increase in temperature tends to cause reduction in shear strength. Ladd (5) conducted cone-penetration tests on Buckshot clay at various temperatures (5, 22 and 50 C) and found that hot samples gave slightly higher strength at a given mositure content and cold samples gave a higher strength at a given consolidation pressure. Semchuk (14) performed undrained triaxial tests on two soils (both consolidated and sheared at test temperatures of 35 and 77 F) and found practically no temperature influence on shear strength for both soils. Seed, Mitchell, and Chan (13) found that pore pressures in undrained triaxial samples vary with temperature and that an increase in temperature causes an increase in pore pressure. This result was varified by Ladd (5) and others.

From undrained triaxial tests conducted on compacted San Francisco Bay mud, Mitchell (10) found that higher temperatures produce lower shear strength and higher pore-pressure buildup during the shear. The data obtained by Duncan and Campanella (3) on soils consolidated at 68 F and sheared in undrained triaxial tests (at 68, 95.8, and 119.8 F) indicated that an increase in temperature causes a reduction in strength,

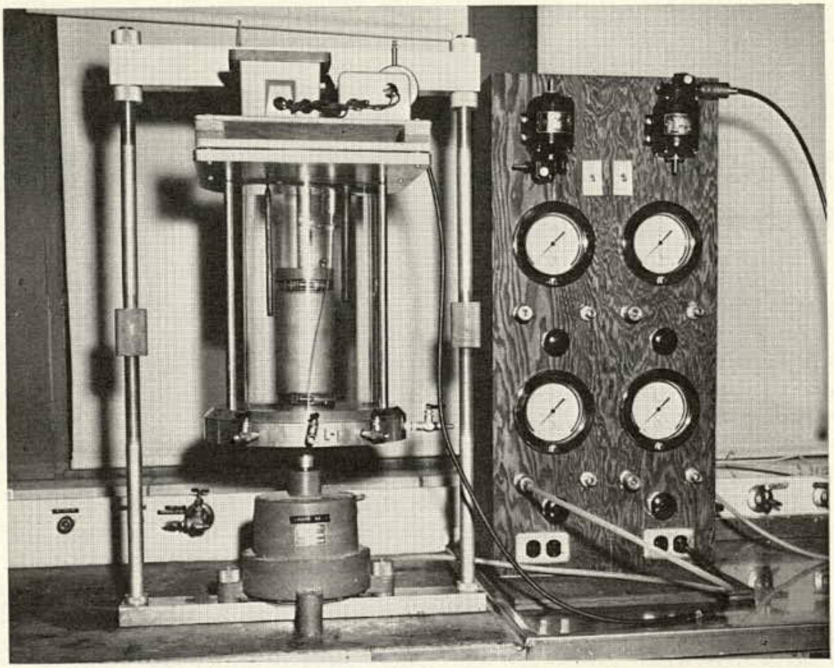

Figure 1. Temperature-controlled triaxial cell.

an increase in initial pore pressure, and a decrease in pore-pressure buildup during the shear testing.

In view of the importance of temperature effects on the engineering properties of soil and because of the apparent lack of complete agreement among previous researchers, it was decided to further enrich the existing body of knowledge on this subject by conducting additional studies at the University of Washington.

TABLE 1

COMPRESSIVE STRENGTH OF C & C CLAY

Test Designation	Temperature (deg F)	Failure Moisture Content (percent)	Compressive Strength (psi)
75-1	75	40.8	12.4
75-2	75	40.3	15.1
75-3	75	38.6	19.5
75-4	75	37.7	26.4
75-5	75	37.4	28.4
75-6	75	34.7	50.2
75-7	75	34.4	52.7
100-1	100	39.2	15.3
100-2	100	38.7	17.2
100-3	100	37.5	23.2
100-4	100	37.1	27.4
100-5	100	35.5	35.0
100-6	100	34.9	42.2
125-1, 2[1]	125	41.5	9.1
125-3	125	38.6	13.9
125-4, 5[1]	125	36.9	25.7
125-6, 7[1]	125	34.65	41.4
150-1, 2[1]	150	40.35	9.85
150-3	150	38.0	18.0
150-4	150	37.1	19.8
150-5	150	35.0	31.9
150-6	150	33.5	46.0

[1]Average of two tests.

SOIL TYPE AND EXPERIMENTAL PROCEDURE

The clay tested was a powdered dry commercial kaolinite clay produced by the Spinks Clay Company (Paris, Tenn.) and known by the trade name C & C. It was selected because previous test data (11) showed a linear relationship between the logarithm of strength and the moisture content at room temperature. Fifty pounds of the dry clay (85 percent finer than 0.005 mm) was mixed with distilled, de-aired water to a moisture content of 47 percent and stored in a plastic container in a humid room for approximately one year before use. The physical properties of the sample, determined just before starting the experiments, are as follows: liquid limit, 68

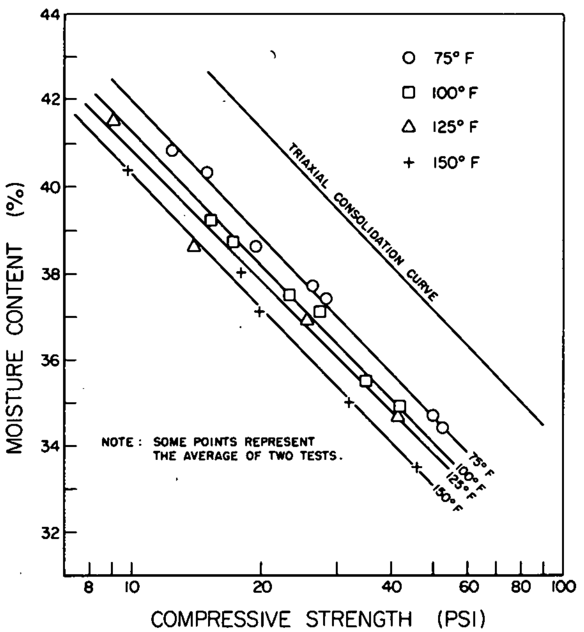

Figure 2. Failure envelopes for C & C clay as a function of moisture content and temperature.

percent; plastic limit, 35 percent; shrinkage limit, 22 percent; specific gravity, 2.58.

The clay was molded and trimmed into test cylinders 2.8 in. in diameter and 6.5 in. in height, then enclosed in a single layer of slit filter paper and two rubber membranes. The samples were placed in triaxial cells and consolidated for 7 days at room temperature (approximately 75 F) under hydrostatic stresses ranging from 20 to 98 psi. At the end of the consolidation period, the samples were taken out of the triaxial cells and their diameter, height, and moisture content were determined. The samples were then placed in heat-controlled triaxial cells over a plastic-covered pedestal to prevent any "communication" between the sample and the drainage outlets; thus no pore pressures were measured. Each cell was subsequently filled with glycerine, which was continuously circulated by propellers to insure a uniform temperature within the cell. Each cell was heated by a 500-watt electric-resistance heater element submerged in the cell fluid. A thermostat kept the temperature range to ±2 F. Several checks proved the effectiveness of the propellers. Figure 1 shows one of the temperature-controlled triaxial cell assemblies during the shearing operation. Each experiment was conducted at a constant cell temperature of 75, 100, 125, and 150 F. The cell was maintained at the test temperature for at least 8 hours prior to shear to insure a uniform temperature throughout the soil. At the end of this period, the change in the height of the sample was measured and the change in the volume of the sample was calculated. The calculation was based on the assumption that the temperature expansion of the pore water is the same as that of ordinary water and that the temperature expansion of the individual soil particles is negligible. With this information, the average cross-sectional area of the sample was estimated.

Following the 8-hour heating period, the soil was loaded in unconfined compression in a stress-controlled loader at the rate of 5.07 lb per 2 minutes (approximately 0.8 psi per 2 minutes) until failure. No drainage was permitted and no lateral stresses were applied during the 8-hour heating and during the actual shear. At the end of failure, the samples were taken out for moisture content determination. When the moisture content of the sample, just before heating, exceeded that of the sample at failure by more than 0.1 percent the test data were not included in the calculation. Likewise, the test data were rejected when the moisture content of the soil in the immediate vicinity of the failure plane differed from that of other parts of the sample by 0.2 percent.

The degree of saturation was calculated from

$$S \text{ percent} = \frac{W_w}{\gamma_w\left(V - \dfrac{W_s}{\gamma_s}\right)} \times 100 \quad (1)$$

where S is the degree of saturation; W_w and W_s are the weight of the water and solids, respectively; V is the total volume of the soil-water system; and γ_w and γ_s are the density of water and solids. (Since the measurement of volume is subject to some error, a sample was not rejected as being unsaturated unless the calculated degree of saturation was less than 98 percent.)

For each test, the true compressive stress was plotted against the Henkey strain, e^H (also referred to as the true strain), defined as

$$e^H = - \int_{L_i}^{L_f} \frac{dL}{L} = - \ln (1 - e^c) \tag{2}$$

where e^c is the Cauchy or engineering strain. (The compressive strength at failure was defined as the peak compressive stress.)

EXPERIMENTAL RESULTS

The results obtained from this experimental series are given in Table 1. In Figure 2, a plot of the moisture content of the soil and the logarithm of compressive strength at each test temperature is shown. The lines in Figure 2 are obtained by a least squares fit (at each test temperature) of the unconfined compressive strength as a function of percent moisture content. It is apparent from this figure that an increase in temperature causes a decrease in the compressive strength of the soil. This result is only logical because the adsorbed water around the individual clay particles assumes a less rigid state as the temperature rises (16), thereby increasing pore pressures and reducing the effective stresses and, therefore, the shear strength of the soil. The increase in entropy of the soil-water system and the increase in Brownian movements (within the liquid occupying the pore interstices) with rising temperatures also contribute to the reduction of strength by preventing the system from assuming the state of least potential energy, where it can reach a stable condition that is most favorable to improved bond-formation among the particles.

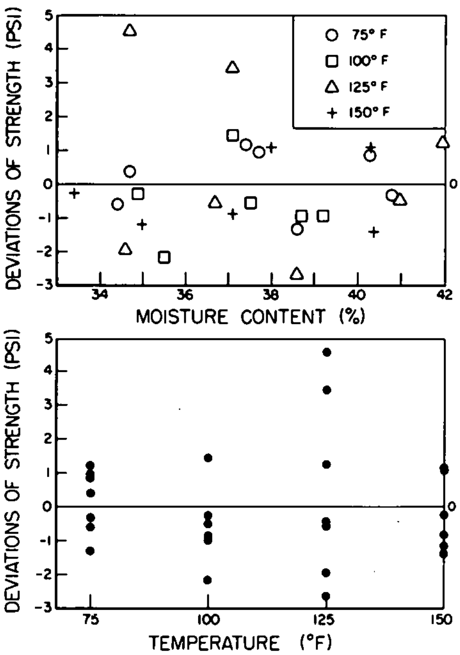

Figure 3. Statistical deviations in strength, or the differences between experimentally predicted (Eq. 4) values of the compressive strength.

Figure 2 further indicates that the soil samples of lower moisture content undergo greater reduction in strength (for the same rise in temperature) than those of higher moisture content, since an increase of 75 F in temperature at w = 41 percent decreases the compressive failure stress by only about 4 psi, while at w = 34 percent the soil loses about 17 psi in strength.

The samples that failed at 75 F were considered to be "reference" samples since this was also the consolidation temperature. As previously mentioned, at the reference temperature there is a linear relationship between the logarithm of the compressive strength and the moisture content. This can be expressed as

$$\sigma_o = \exp (A - Bw) \tag{3}$$

where σ_0 is the compressive strength at the reference temperature, w is the moisture content in percent, and A and B are material constants that must be determined experimentally.

Two different equations describing the compressive strength of C & C clay as a function of both moisture content and temperature have been derived from the data. The first relates an increase in temperature to an equivalent increase in moisture content:

$$\sigma_T = \exp\,(A - Bw - C\Delta T) = \sigma_0 \exp\,(-C\Delta T) \qquad (4)$$

where σ_T is the compressive strength at reference temperature, w is the moisture content, ΔT is the difference between the test and reference (or consolidation) temperature, and A and B are the same material constants as in Eq. 3. The soil parameter C is characteristic of temperature-induced changes in the clay and must be determined by testing at a temperature other than the reference temperature.

Equation 4 indicates that the strength of the soil decreases with both increasing moisture content and temperature, and that the increase in temperature is analogous to an increase in moisture content. Consequently, $C\Delta T/B$ might be regarded as being equal to w', an equivalent moisture content.

A least squares fit of Eq. 4 to the data is shown in Figure 4 by solid lines. The statistical deviations in strength with respect to moisture content and temperature changes are shown in Figure 3. It is seen from Figure 3 that the deviations are relatively random and evenly scattered about line 0-0, thus indicating that the parameter C appears to be independent of both moisture content and temperature (2).

The second proposed equation, shown below, relates the change in compressive strength to the ratio of the reference and test temperatures and to the ratio of the densities of the pore fluid at these respective temperatures. This equation eliminates the need for experimental determination of the constant C:

$$\sigma_T = \frac{T_0}{T} \exp\left[A - B\left(\frac{\gamma_{wo}}{\gamma_w}\right) w\right] \qquad (5)$$

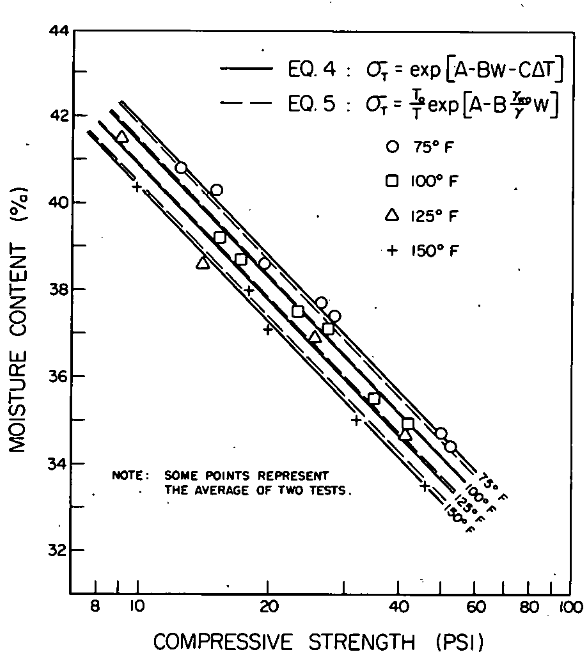

Figure 4. Least squares fit of Eq. 4 and Eq. 5 to the data.

where γ_{wo} and γ_w are the water densities at reference and test temperatures, T_0 and T, respectively.

The least squares fit of Eq. 5 is shown in Figure 3 by dashed lines and can be compared with the least squares fit of Eq. 4, shown by solid lines. Over the temperature and moisture content range considered, Eqs. 4 and 5 predict nearly the same values. These equations become the same as Eq. 3 when the test temperature is the same as the reference (consolidation) temperature.

Further research is being conducted at the University of Washington to determine if the foregoing relationships are applicable to other soils, and if so, how well the parameters discussed can be related to some easily determinable soil properties such as LL, PL, or PI.

CONCLUSIONS

The following conclusions can be made about C & C clay consolidated at 75 F and sheared at 75, 100, 125, and 150 F in unconfined and undrained compression:

1. An increase in temperature causes a decrease in compressive strength, as indicated in Figure 2. Moreover, an increase in temperature is analogous to an increase in moisture content, as revealed by Eq. 4.
2. A linear relationship exists between the logarithm of compressive strength and the moisture content at all test temperature levels. The failure lines at constant temperatures appear to be almost parallel to the consolidation curve, as shown in Figure 2.
3. Soil samples of lower moisture contents undergo greater absolute reduction in strength with rising temperatures than those of higher moisture content levels. This is in agreement with Campanella and Mitchell's recent finding in which they state that, at small values of effective stress, temperature-induced pore pressures decrease (1).

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