INFLUENCE OF TEMPERATURE VARIATIONS ON SOIL BEHAVIOR

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by

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ABSTRACT

Analyses show that the volume changes and pore water pressure variations that develop in saturated soils as a consequence of temperature changes can be expressed in terms of the thermal expansion of the soil components, compressibility of the soil and physico-chemical effects.

Drained triaxial tests on saturated, remolded illite under constant all round confining pressure showed that significant permanent volume decreases may develop during the first cycle of heating. These volume decreases are interpreted as due to irreversible physico-chemical adjustments needed to enable the soil to carry the applied effective stresses. Temperature induced volume changes appear analogous to pressure induced volume changes with temperature increase causing a decrease in sample volume and temperature decrease leading to a volume increase.

The compression index of remolded illite has been found to be independent of temperature; however, the higher the temperature the lower the void ratio at any given consolidation pressure.

The theoretical analysis gives reasonably good estimates of the pore pressure change caused by temperature variations under undrained conditions. The magnitude of the pore pressure change is controlled primarily by the compressibility of the soil sample and the thermal expansion of the pore water, with the greatest pore pressure changes

associated with the smallest compressibilities. A temperature induced pore pressure parameter has been defined as the change in pore pressure per unit change in temperature per unit effective stress. This parameter is in the range of 0.0075 to 0.010 per °F for several clays.

Knowledge of the effects of temperature changes on the volume and effective stress in saturated clays is useful for assessing the effects of temperature variations on properties of soils in the field, the determination of the effect of differences between field and laboratory temperatures on behavior, the assessment of necessary laboratory temperature control during undrained tests, and the study of physico-chemical phenomena in soils.

APPENDIX II

Notations

The following symbols are used in this paper.

C_R = swelling index

e = void ratio

F = change in pore pressure per unit change in temperature per unit effective stress = $\frac{\Delta u}{\Delta T}/\sigma'$.

m_s = compressibility of mineral solids under an all-round pressure

m_s' = compressibility of mineral solids when particles are subjected to concentrated loadings

m_w = compressibility of water

n = porosity

P = pressure

T = temperature

u = pore water pressure

V_m = total volume of soil specimen

V_s = volume of mineral solids

V_w = volume of pore water

α_s = thermal coefficient of cubical expansion of mineral solids

α_{st} = physico-chemical temperature coefficient of soil structure volume change

α_w = thermal coefficient of cubical expansion of soil water

$(\Delta V_{DR})_{\Delta T}$ = volume of water drained due to temperature change

$(\Delta V_m)_{\Delta T}$ = change in volume of soil specimen due to temperature change

$(\Delta V_s)_{\Delta T}$ = change in volume of mineral solids due to temperature change

$(\Delta V_{st})_{\Delta T}$ = change in volume of soil structure due to temperature induced changes in interparticle forces

$(\Delta V_{w})_{\Delta T}$ = change in volume of water due to a temperature change

σ' = effective stress

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INTRODUCTION

A number of studies have established that the engineering properties of soils can be influenced significantly by temperature variations. Finn (1)[3] and Paaswell (2), among others, have investigated the effect of temperature on consolidation. Temperature effects on interparticle forces, pore water pressures, and swelling have been considered by Lambe (3,4,5). Ladd (6), Scott (7), Mitchell (8), Duncan and Campanella (9), and others have investigated the effects of temperature variations on soil strength. Murayama and Shibata (10) and Mitchell and Campanella (11) have illustrated the influence of temperature on soil creep. Pore water pressure variations and volume changes associated with temperature changes have been discussed by Mitchell and Campanella (11), Henkel and Sowa (12) and Duncan and Campanella (9).

Not only do temperature variations have an effect on the engineering properties that may be important in field application, but also it is now generally recognized that close temperature control is needed in many types of soil testing. In addition knowledge of behavior as a function of

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[3] Numbers in parentheses refer to corresponding items in the Appendix.

temperature can provide a basis for the study of mechanisms controlling the flow and deformation properties of soils as shown by Andersland and Akili (13) and Mitchell, Campanella and Singh (14).

That temperature variations may indeed cause significant variations in the volume and effective stress of saturated clays is shown in Figs. 1 and 2. Fig. 1 shows the percentage of original pore water volume drained from a saturated specimen of illite subjected to a temperature increase from 66°F to 140°F followed by cooling to 66°F. In this test the specimen was maintained under a constant all round effective stress, σ_3', of 2.0 kg per sq. cm. The temperature changes were made in increments as defined by the numbered points in Fig. 1. In each case the change in temperature was made rapidly and the sample was allowed to drain freely until equilibrium had been established.

Fig. 2 shows the variation of effective stress, σ_3', with temperature under undrained conditions for the same material subjected to the same temperature variations. It may be seen that a very large pore water pressure developed as a result of heating which dissipated, but not reversibly, during the subsequent cooling.

The purposes of this paper are to further illustrate behavior of the type shown in Figs. 1 and 2, to interpret the causes of this behavior in terms of fundamental soil parameters, and to consider some practical implications.

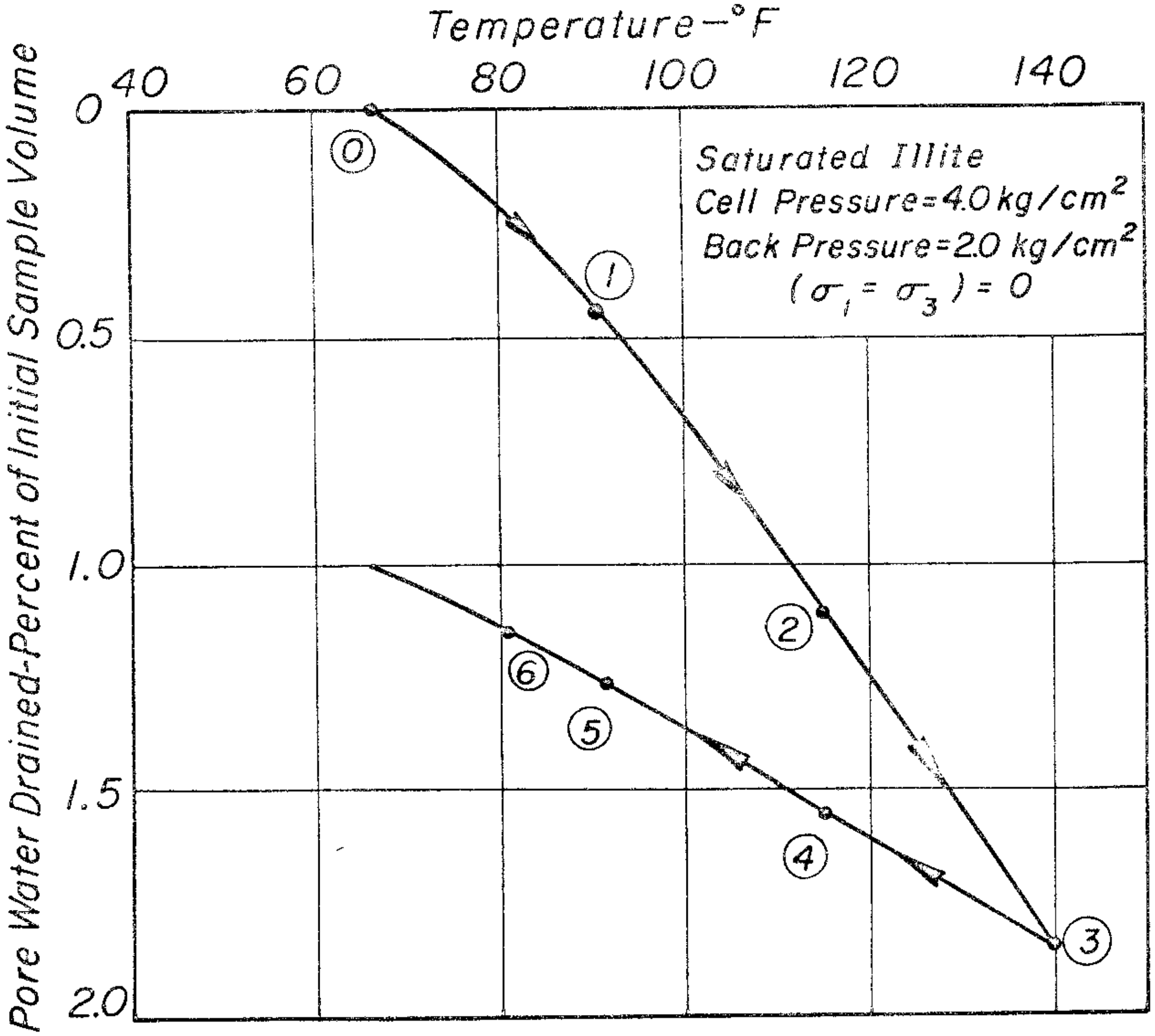

FIG. I EFFECT OF TEMPERATURE VARIATIONS ON VOLUME UNDER DRAINED CONDITIONS

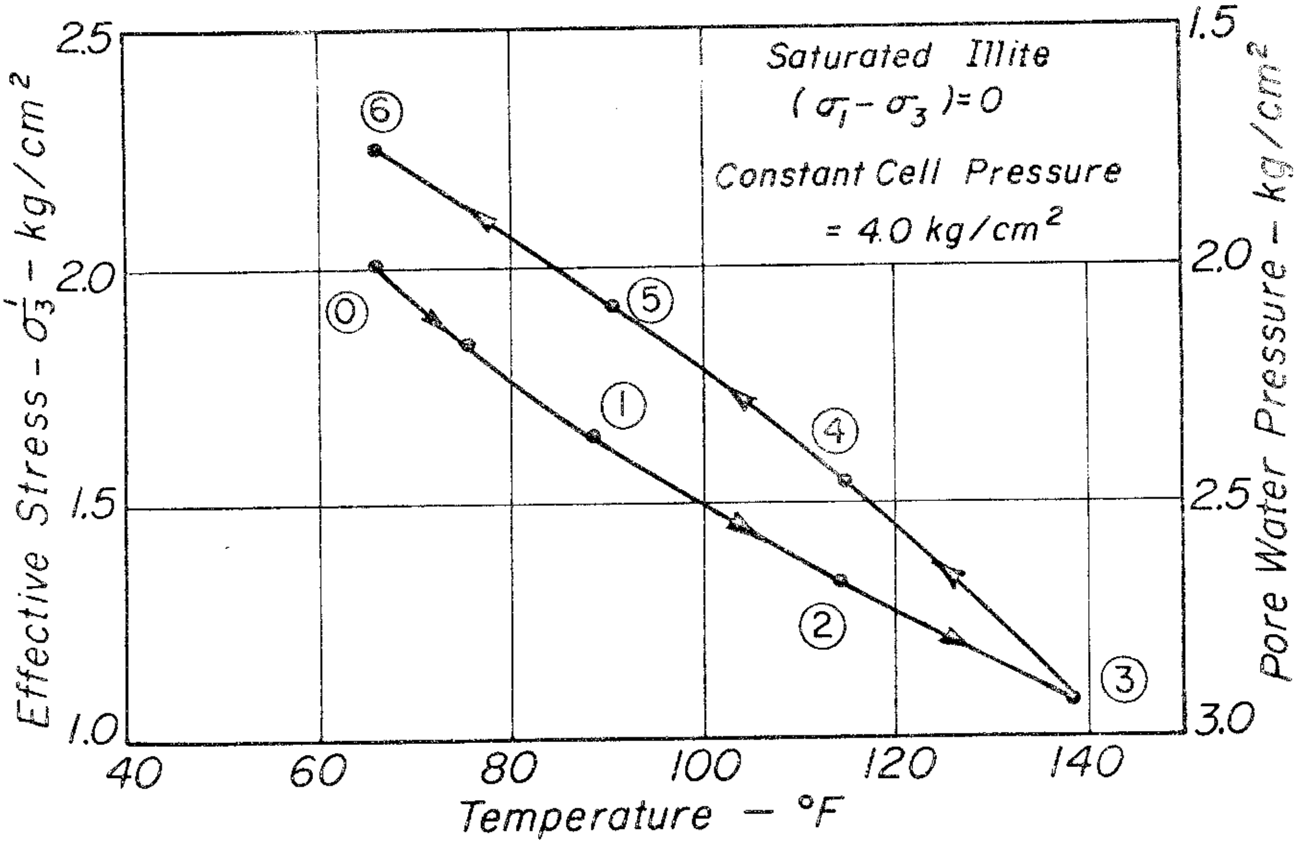

FIG.2 EFFECT OF TEMPERATURE VARIATIONS ON EFFECTIVE STRESS UNDER UNDRAINED CONDITIONS

THEORETICAL ANALYSIS OF TEMPERATURE EFFECTS ON VOLUME CHANGES AND PORE PRESSURES IN SATURATED SOILS

Drained Conditions

Volume changes due to temperature variations under conditions of constant effective stress may be considered in terms of the temperature-dependent properties of a given saturated soil. These volume changes result from the thermal expansion of the mineral solid and pore water, and any changes in the soil structure resulting from temperature changes.

The change in volume of the pore water due to temperature changes can be expressed as

$$(\Delta V_w)_{\Delta T} = \alpha_w V_w \Delta T \quad . \quad . \quad . \quad . \qquad (1)$$

where α_w = thermal coefficient of cubical expansion of soil water,

V_w = volume of pore water, and

ΔT = temperature change.

The change in volume of the mineral solids due to temperature changes can be expressed as

$$(\Delta V_s)_{\Delta T} = \alpha_s V_s \Delta T \quad . \quad . \quad . \quad . \qquad (2)$$

where α_s = thermal coefficient of cubical expansion of mineral solids,

and V_s = volume of mineral solids.

Consider conditions in which a saturated soil specimen is free to drain at constant effective stress. The volume of water drained from the sample due to a temperature change can be expressed as

$$(\Delta V_{DR})_{\Delta T} = (\Delta V_w)_{\Delta T} + (\Delta V_s)_{\Delta T} - (\Delta V_m)_{\Delta T} \quad . \quad . \qquad (3)$$

where $(\Delta V_m)_{\Delta T}$ = change in volume of the soil specimen due to a temperature change.

The negative sign in Eq. 3 is necessary since all volume increases are considered positive. For example, if during a temperature change the increase in the volume of specimen, $(\Delta V_m)_{\Delta T}$, were equal to the sum of the increases of the pore water volume, $(\Delta V_w)_{\Delta T}$, and the solids volume, $(\Delta V_s)_{\Delta T}$, then no water would be drained from the specimen and $(\Delta V_{DR})_{\Delta T}$ would equal zero. Combining Eqs. 1, 2 and 3 gives

$$(\Delta V_{DR})_{\Delta T} = \alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} \quad . \quad . \qquad (4)$$

If the soil grains are in mineral to mineral contact and the temperature is changed, then each particle will undergo the same volumetric strain (assuming the coefficient of thermal expansion to be the same for all the soil minerals present) and the whole soil mass will undergo the same volumetric strain. This volumetric strain will be given by $\alpha_s \Delta T$. In addition a further change in volume may result from a temperature induced change in interparticle forces which requires some reorientation or relative movement of soil grains in order to permit the soil structure to carry the same effective stress. If the volume change of the soil due to this effect is designated by $(\Delta V_{st})_{\Delta T}$, then the total change in volume of the soil, ΔV_m, will be

$$(\Delta V_m)_{\Delta T} = \alpha_s V_m \Delta T + (\Delta V_{st})_{\Delta T} \quad . \quad . \quad . \qquad (5)$$

Undrained Conditions

Pore water pressure changes may develop during temperature changes of a saturated soil under undrained conditions as shown by Fig. 2. The governing criterion for undrained conditions is that the sum of the separate volume changes of the soil constituents (pore water and mineral solids) due to both temperature and pressure changes must equal the sum of the volume changes of the total soil mass due to both temperature and pressure changes. This can be written as

$$(\Delta V_w)_{\Delta T} + (\Delta V_s)_{\Delta T} + (\Delta V_w)_{\Delta P} + (\Delta V_s)_{\Delta P} = (\Delta V_m)_{\Delta T} + (\Delta V_m)_{\Delta P} \quad . \qquad (6)$$

where ΔV_w = volume change of pore water,

ΔV_s = volume change of mineral solids,

ΔV_m = volume change of soil mass

The subscripts ΔT and ΔP represent "due to" temperature and pressure change respectively.

Also,

$$(\Delta V_w)_{\Delta P} = m_w V_w \Delta u \quad . \quad . \quad . \quad . \qquad (7)$$

where m_w = compressibility of water, and

Δu = change in water pressure;

and

$$(\Delta V_s)_{\Delta P} = m_s V_s \Delta u + m_s' V_s \Delta\sigma' \quad . \quad . \quad . \qquad (8)$$

where m_s = compressibility of mineral solid under an all round pressure,

m_s' = compressibility of mineral solids when particles are subjected to concentrated loadings,

$\Delta\sigma'$ = change in effective or intergranular stress,

$m_s V_s \Delta u$ = change in volume of mineral solid due to a change in pore pressure, and

$m_s' V_s \Delta\sigma'$ = change in volume of mineral solid due to a change in intergranular stress*;

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*Changes in intergranular stress will manifest themselves by changes in the forces transmitted at interparticle contacts.

and

$$(\Delta V_m)_{\Delta P} = m_v V_m \Delta\sigma' \quad . \quad . \quad . \quad . \qquad (9)$$

where m_v = compressibility of soil mass or soil structure.

Substituting Eqs. 1, 2, 7, 8, and 9 into Eq. 6 and rearranging terms gives

$$\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = m_v V_m \Delta\sigma' - m_w V_w \Delta u - V_s(m_s \Delta u + m_s' \Delta\sigma') \qquad (10)$$

If the total stress, σ, remains constant during a temperature change, then

$$\Delta\sigma' = -\Delta u \quad . \quad . \quad . \quad . \quad . \qquad (11)$$

Substituting Eq. 11 into Eq. 10 gives

$$\alpha_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = m_v V_m \Delta\sigma' - m_w V_w \Delta u - \Delta u V_s (m_s - m_s')$$

But it is unlikely that m_s and m_s' are significantly different than each other in magnitude, and also m_s and m_s' are much less than m_v and m_w. Thus little error will result from assuming $m_s - m_s' = 0$, and therefore

$$\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = -m_v V_m \Delta u - m_w V_w \Delta u \quad . \qquad (12)$$

It may be noted that the left side of Eq. 12 is equal to $(\Delta V_{DR})_{\Delta T}$, the volume of water drained from the soil due to a temperature change (Eq. 4), and the right side is an equivalent change in volume caused entirely by a change in pore pressure.

Combining Eqs. 12 and 5 gives

$$\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - \alpha_s V_m \Delta T - (\Delta V_{st})_{\Delta T} = -m_v V_m \Delta u - m_w V_w \Delta u \qquad (13)$$

Since

$$V_m = V_w + V_s \qquad (14)$$

Eq. 13 becomes

$$\alpha_w V_w \Delta T - \alpha_s V_w \Delta T - (\Delta V_{st})_{\Delta T} = - m_v V_m \Delta u - m_w V_w \Delta u \quad . \qquad (15)$$

Noting that for a saturated soil the porosity, n, is defined by $\frac{V_w}{V_m}$, equation (15) can be rearranged to give the pore water pressure change accompanying a temperature change under undrained conditions,

$$\Delta u = \frac{n\Delta T(\alpha_s - \alpha_w) + \frac{(\Delta V_{st})_{\Delta T}}{V_m}}{m_v + nm_w} = \frac{n\Delta T(\alpha_s - \alpha_w) + \alpha_{st}\Delta T}{m_v + nm_w} \quad . \qquad (16)$$

where α_{st} = physico-chemical coefficient of structural volume change caused by a change in temperature.

Thus the factors controlling the pore pressure change are the magnitude of the temperature change, the porosity, the difference between the coefficients of thermal expansion for soil grains and water, which should be approximately constant for all soils, the volumetric strain due to physico-chemical effects, and the compressibility of the soil structure. Since for most soils (but not rocks) $m_v >> nm_w$, equation (16) can be approximated by

$$\Delta u = \frac{n\Delta T(\alpha_s - \alpha_w) + \alpha_{st}\Delta T}{m_v} \quad . \quad . \quad . \qquad (16a)$$

When written in this form it may be noted that for a given temperature change the soil compressibility should have a dominating influence. In the application of the above equations consistency in algebraic signs must be observed. Both α_s and α_w are positive and correspond to a volumetric increase with increasing temperature. The compressibilities, m_v and m_w, are negative since an increase in pressure causes a decrease in volume, and α_{st} is positive if an increase in temperature causes an increase in volume of the soil structure. As shown subsequently α_{st} is usually negative.

A series of experiments were carried out to test the relationships developed above and to obtain data on the importance of temperature induced volumetric strains resulting from physico-chemical effects.

TEST EQUIPMENT

Specially designed triaxial equipment, which permits both control of temperature and rapid temperature change under controlled stress conditions, has been described by Mitchell and Campanella (1963). Sample temperature was controlled by means of an air temperature environment chamber surrounding the triaxial chamber. The sample temperature was changed by circulating preconditioned water through the triaxial cell until the sample reached the desired temperature. At that point the water circulation was stopped and the sample temperature maintained constant by means of the outer air conditioned chamber. Temperature equilibrium was established about 20 minutes after the start of the temperature change. This equipment can accurately (±1/2°F) and independently control the temperature of four test specimens over the range of 40 to 140°F. Sample temperature was measured with an iron-constantan thermocouple encased in a 1/16-inch diameter stainless-steel tube inserted about 1/2-inch into the base of the sample.

An electronic pressure transducer (Statham) mounted in the sample base beneath the porous stone provided rapid and reliable pore pressure measurements. Changes in sample length were measured by means of a .0001 inch dial indicator. Test pressures were controlled by means of precision air regulators (Fairchild-Stratos) which had a measured accuracy of ±1/2 inch of water (±.001 kg/cm^2).

A volume change measuring device, as shown in Fig. 3, was entirely enclosed within the air temperature control environment surrounding the triaxial chamber. Pore water volume changes could be measured to the nearest .01 cm^3 using the 2 cm^3 pipette. The 10 cm^3 pipette permitted recharging or discharging the measuring pipette as needed.

The cylindrical test specimens (1.4 inches in diameter by 3.5 inches long), surrounded by a slit filter paper side drain to accelerate drainage and pore pressure equalization, were drained at the base only, and were wrapped in two rubber membranes separated by silicon grease and independently sealed at top and bottom with O-rings.

Calibrations were required for accurate analyses of test data from specimens subjected to changes in temperature as follows:

1. All pipettes were calibrated to indicate true volume at temperatures other than 68°F, the reference temperature for the pipettes.
2. The water volume in the porous stones and transmission lines was determined as a function of temperature.
3. The components of the triaxial apparatus were corrected for change in length with temperature variations.
4. A most important consideration in the present investigation was the effect of temperature on the electrical output of the pressure transducers. Although the slope of the calibration curve of electrical output versus pressure was independent of temperature, the zero point did vary with temperature. This zero shift was not linear but was unique for each transducer. Thus, the zero point for all transducers

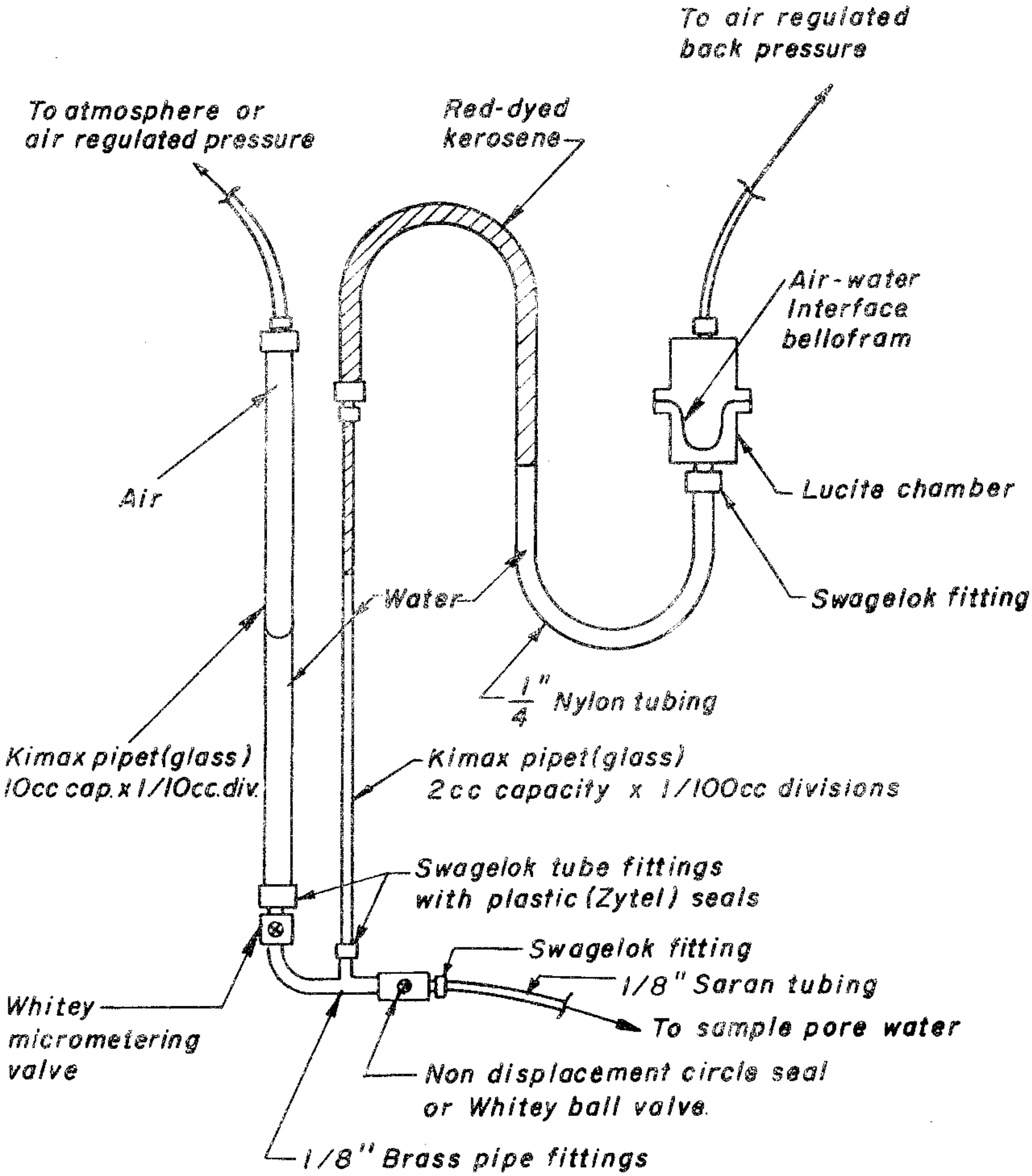

FIG. 3 VOLUME CHANGE MEASURING DEVICE

could be accurately determined at all temperatures. In some transducers, errors as large as 0.10 kg/cm^2 would have resulted if these corrections were not made.

VOLUME CHANGE BEHAVIOR

Rate of Volume Change

In order to investigate the effect of temperature change on the volume of saturated clay, specimens were subjected to temperature variations under conditions of constant isotropic confining and back pressure.

Remolded illite specimens were consolidated initially under an isotropic stress of 2 kg/cm^2 at 65°F for a period of about 10,000 minutes. At the end of this time further volume changes due to secondary compression were negligible. The time required for the completion of primary consolidation was about 700 minutes, and after 10,000 minutes the secondary compression rate, denoted by the rate of volume change, was approximately .001 cm^3/hr. After consolidation the sample temperature was changed daily and readings were taken periodically of the volume of water expelled or absorbed by the sample. Sample height changes and temperature were also recorded.

The results of two such tests are presented here. Fig. 4 shows the result for Sample 1-V-4 in the form of plots of volume* of water expelled or absorbed by the sample against logarithm of time at each temperature. The sample underwent three complete cycles of temperature changes over a

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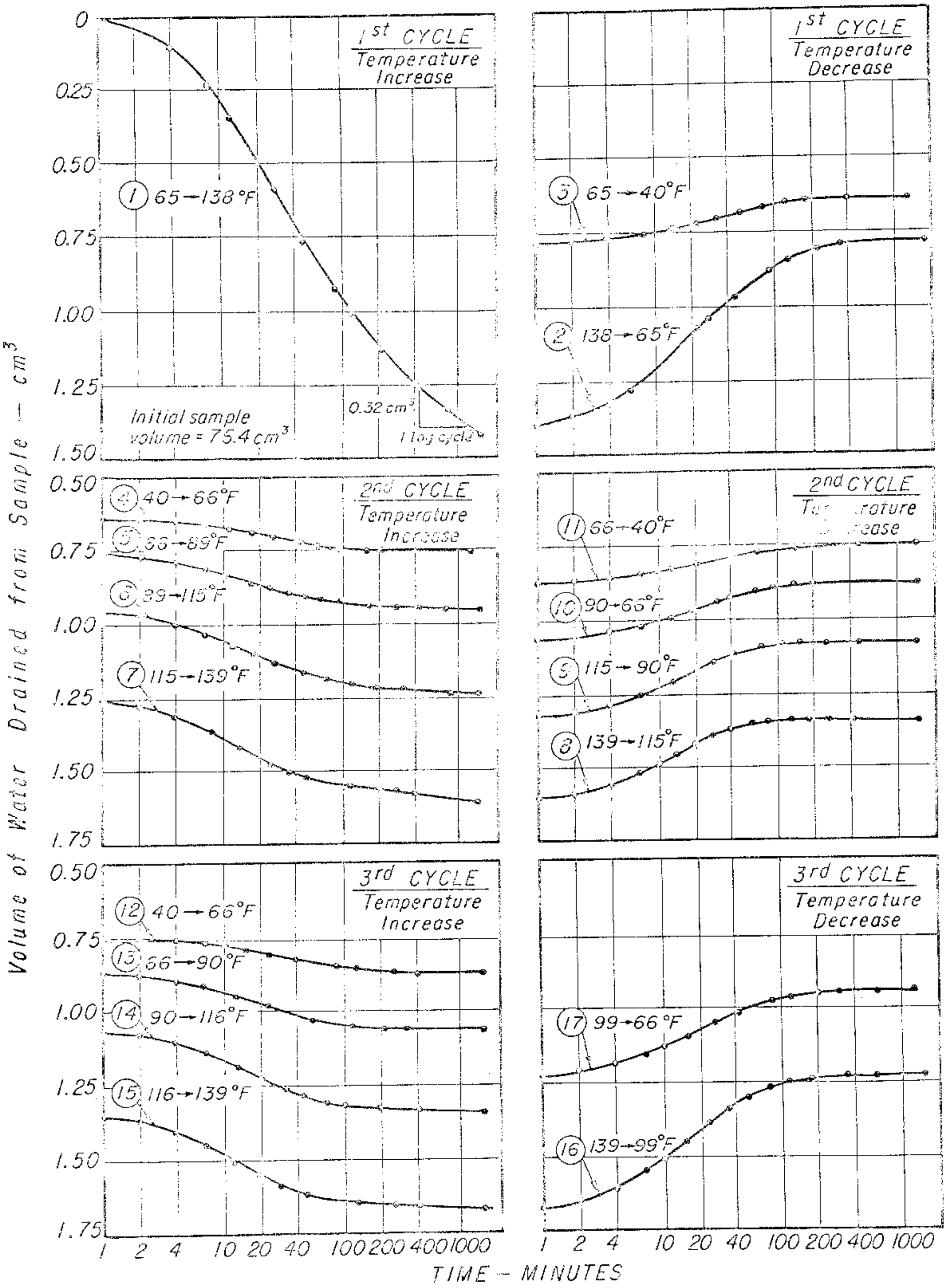
*The plotted volumes include the small volume changes of water in the porous stone and transmission line.



Remolded Illite (Sample I-V-4)

Cell pressure = 4.00 kg/cm² $(\sigma_1 - \sigma_3) = 0$

Back pressure = 2.00 kg/cm²

FIG. 4 RELATIONSHIP BETWEEN VOLUME OF WATER DRAINED FROM SAMPLE AND TIME DURING TEMPERATURE CHANGES AT CONSTANT STRESS range of about 40 to 140°F. The initial temperature increase and decrease were large (65 to 138°F and 138 to 65°F -- see curves 1 and 2) compared to subsequent temperature changes. In each case the ordinate values indicate the amount of water drained from the sample since the start of the first cycle of temperature increase.

The results show that water drains out of the sample during temperature increases and is absorbed by the sample during temperature decreases. It may be noted that the shape of all these curves is similar to normal consolidation curves where volume changes result from changes in effective stress. In consolidation, water drainage takes place in order to dissipate excess pore pressures resulting from stress changes. In clay soils this drainage is usually retarded because of the relatively low permeability of the clay and is often followed by a significant rate of secondary compression. Fig. 2 shows that when sample temperature is increased pore pressures increase and when sample temperature is decreased pore pressures decrease. These temperature-induced excess pore pressures will dissipate if the sample is allowed to drain. Thus, from the standpoint of dissipation of excess pore pressures, the process of consolidation due to temperature changes may be analogous to that due to pressure changes.

The results in Fig. 4 indicate that for the first temperature increase (curve 1) the rate of secondary compression (rate at which water was drained from the sample) was significant near the end of the test period as compared with that prior to the temperature increase. It is evident, however, that this high rate developed only during the initial temperature increase, since the secondary compression rates during subsequent temperature increases were small (curves 4 to 7 and 12 to 15). This behavior is similar to that commonly observed during recompression cycles in the usual type of pressure induced consolidation process. Secondary rebound rates were negligible during all of the three cycles of temperature decreases. The following explanation is offered for the volume change behavior described above.

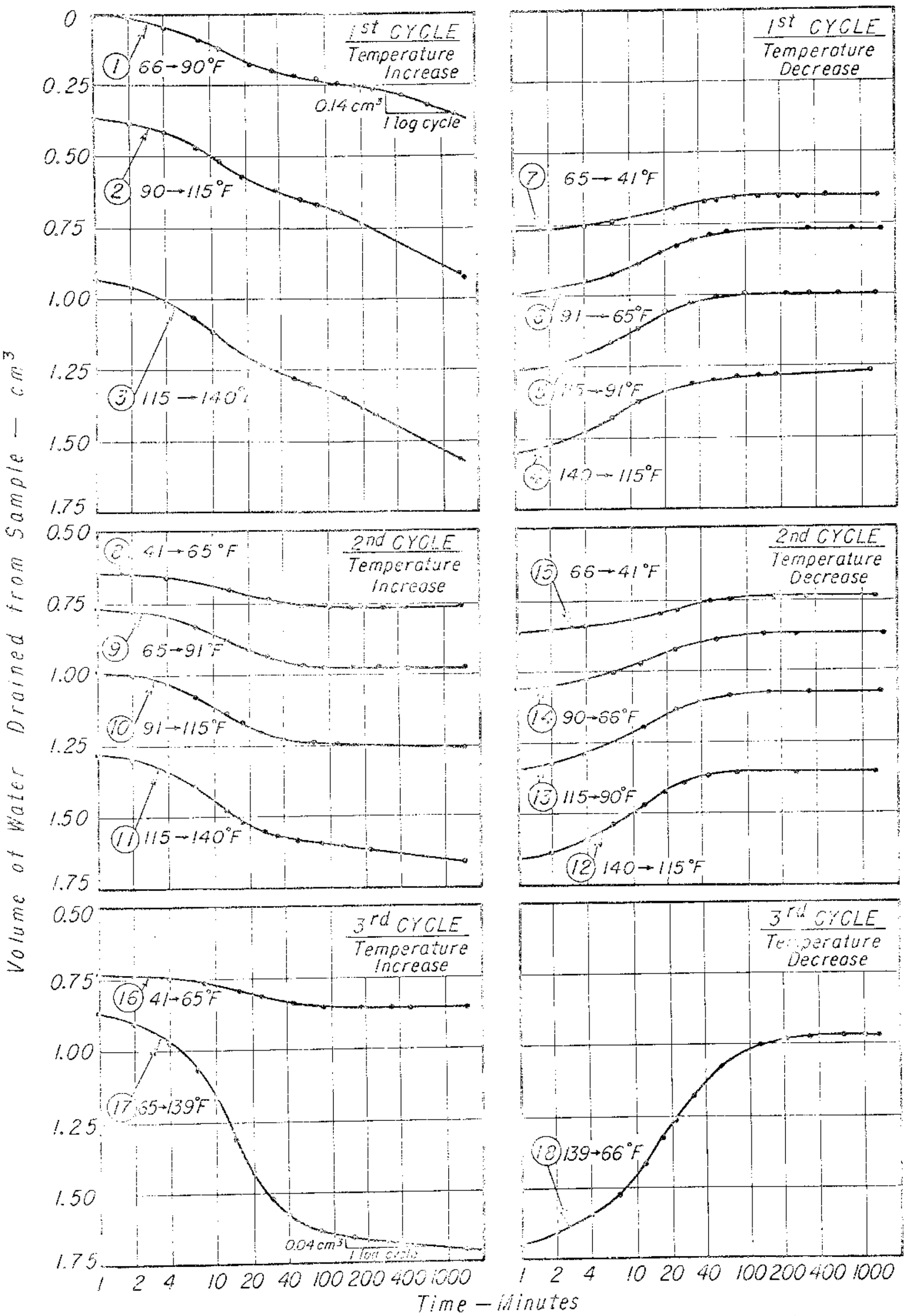
During initial consolidation at constant temperature the void ratio decreases until sufficient shearing resistance is developed through interparticle bonds to resist the interparticle shear forces resulting from applied boundary normal stresses. When the temperature of a normally consolidated specimen is increased two effects occur. If the increase in temperature is rapid a significant positive pore pressure may develop, as shown by Fig. 2, even though the sample is maintained under fully drained conditions. This excess pore pressure results primarily from a greater volumetric expansion of the pore water than of the mineral solids. The lower the permeability of the soil the longer the period required for this pressure to dissipate. Dissipation of this pressure can account for the parts of the curves in Fig. 4 that resemble primary consolidation behavior.

The second effect relates to the influence of increased temperature on the strength of the soil structure. An increase in temperature causes a decrease in the shearing strength of individual interparticle contacts. This decrease in interparticle bond strength may be considered to result from the increase in thermal energy which acts in conjunction with the shear force at interparticle contacts to increase the probability of bond slippage or failure. As a consequence there is a partial collapse of the soil structure, and a decrease in void ratio until a sufficient number of additional bonds are formed to enable the soil to carry the stress at the higher temperature. This effect depends only on properties of the soil structure and its rate is independent of pore water pressure considerations. It is analogous to secondary compression under a stress increase.

When the temperature is decreased reverse phenomena may be anticipated. Differences between the volumetric shrinkage of the soil grains and water gives rise to a tension in the pore water, which in turn causes the sample to absorb water as shown by the temperature decrease curves in Fig. 4. A secondary volume change effect is not observed in this case, however, because the temperature decrease causes a strengthening of the soil structure and no further structural adjustment is needed to carry the effective stresses.

On subsequent temperature increase cycles the secondary effect is negligible since the structure has been previously strengthened so that it can carry the effective stress at the higher temperature.

From the foregoing discussion it would be reasonable to conclude that the larger the initial temperature increase the greater the rate of temperature induced secondary compression. Upon subsequent cooling followed by reheating, secondary compression rates should be small provided the temperature increase does not exceed the initial temperature increase. To test these conclusions a second sample of remolded illite, essentially identical to the first, was tested. In the second test the sample temperature was changed in increments of about 25°F over the range of 40° to 140° for the first two cycles of temperature changes. In the third cycle the temperature change was large, 65 to 139°F and 139 to 66°F. The results are shown in Fig. 5. It may be noted that the secondary compression rate for the large initial increase in temperature shown as curve 1 in Fig. 4 (0.32 cm^3/log cycle of time) was



Remolded Illite (Sample I-V-6)

Cell pressure = 4.00 kg/cm² $(\sigma_1 - \sigma_3) = 0$
Back pressure = 2.00 kg/cm²

FIG. 5 RELATIONSHIP BETWEEN VOLUME OF WATER DRAINED FROM SAMPLE AND TIME DURING TEMPERATURE CHANGES AT CONSTANT STRESS very much greater than the secondary compression rate for the relatively smaller initial increase in temperature shown in Fig. 5 (0.14 cm^3/log cycle of time). The secondary compression rate for the large temperature increase during the third cycle, curve 17 in Fig. 5, was quite small, however, amounting to 0.04 cm^3/log cycle of time, thus supporting the previously advanced concepts.

Except for the initial temperature increases, the results in Fig. 5 are essentially the same as those in Fig. 4. Curves 1, 2 and 3 in Fig. 5 have the same characteristic shape as Type II consolidation curves reported by Leonards and Girault (15); whereas, Curve 1 in Fig. 4 is characteristic of Type I Curves. Typical Type I and Type II Curves for undisturbed Mexico City clay are shown in Fig 6. According to Leonards and Girault Type I Curves result under relatively high load-increment ratios, of the order of 1.0. Type II Curves are observed when the load-increment ratio is small or the load increment straddles the preconsolidation pressure. It would appear that the similarity between temperature induced and pressure-induced consolidation extends to the shape of the volume change versus logarithm of time, since Curve 1 in Fig. 4 pertains to a large temperature-increment (73°F) and Curves 1, 2, and 3 in Fig. 5 are for small temperature increments (25°F).

Amount of Volume Change

In order to analyze quantitatively the data in Figs. 4 and 5 the results have been summarized and plotted in Figs. 7 and 8, respectively. The lower plot in Figs. 7 and 8 shows cumulative volume of water drained from the sample as a function of sample temperature, and the upper plot shows the corresponding sample height change. The values of water volume

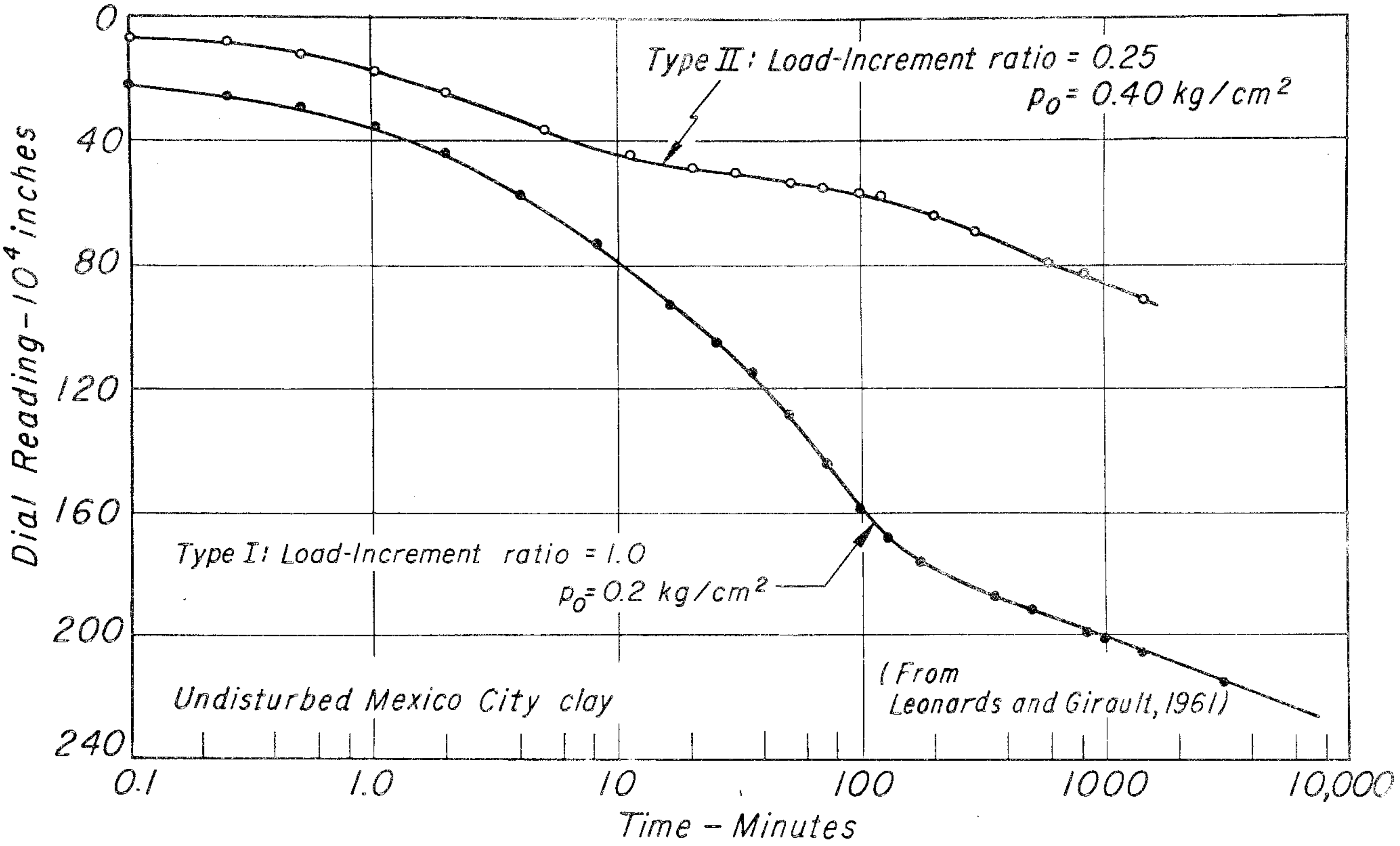


FIG. 6 EFFECT OF LOAD-INCREMENT RATIO ON CONSOLIDATION TIME CURVES

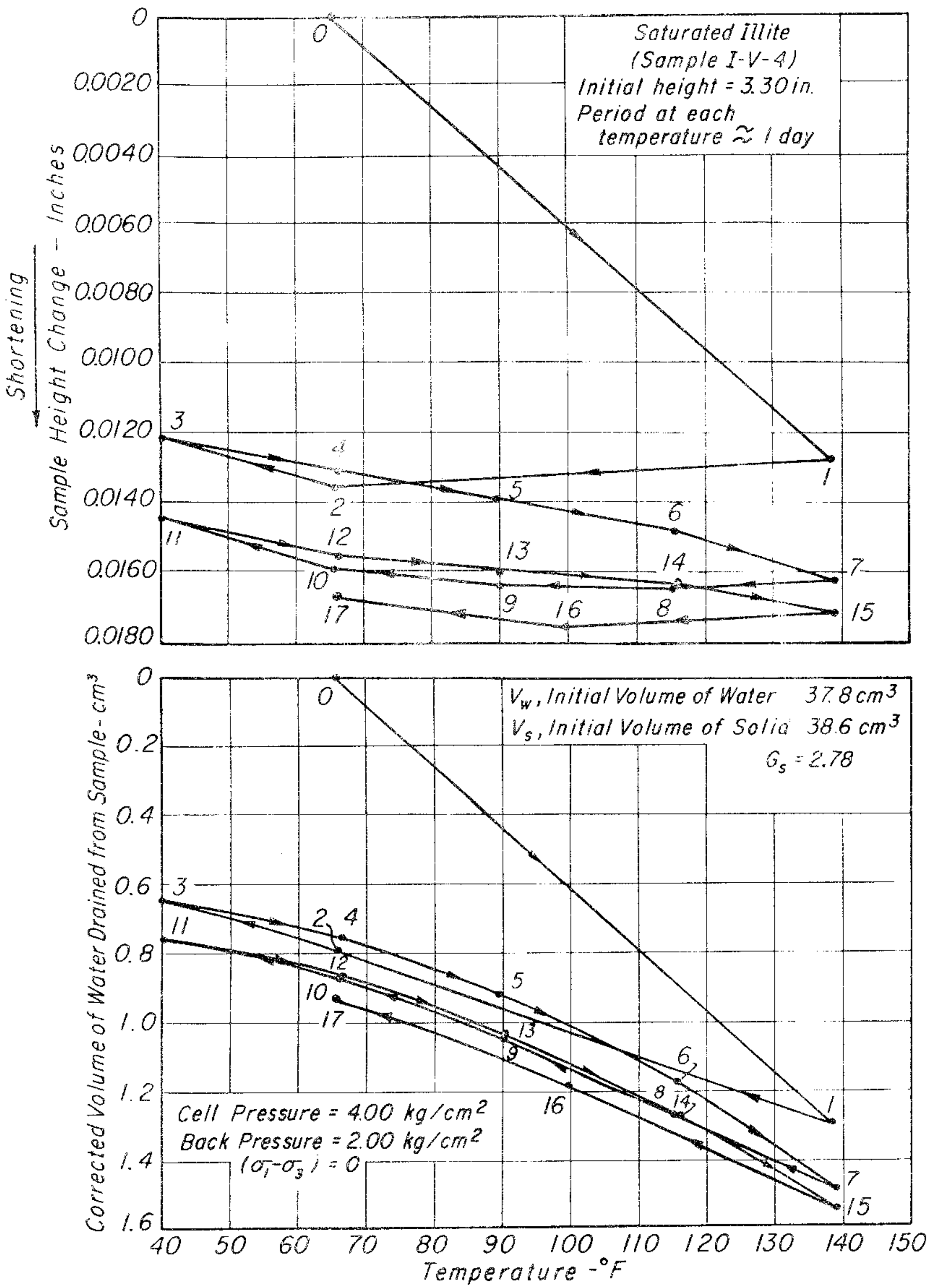


FIG. 7 EFFECT OF TEMPERATURE CHANGE ON SAMPLE HEIGHT AND VOLUME OF WATER DRAINED FROM A SATURATED ILLITE SPECIMEN

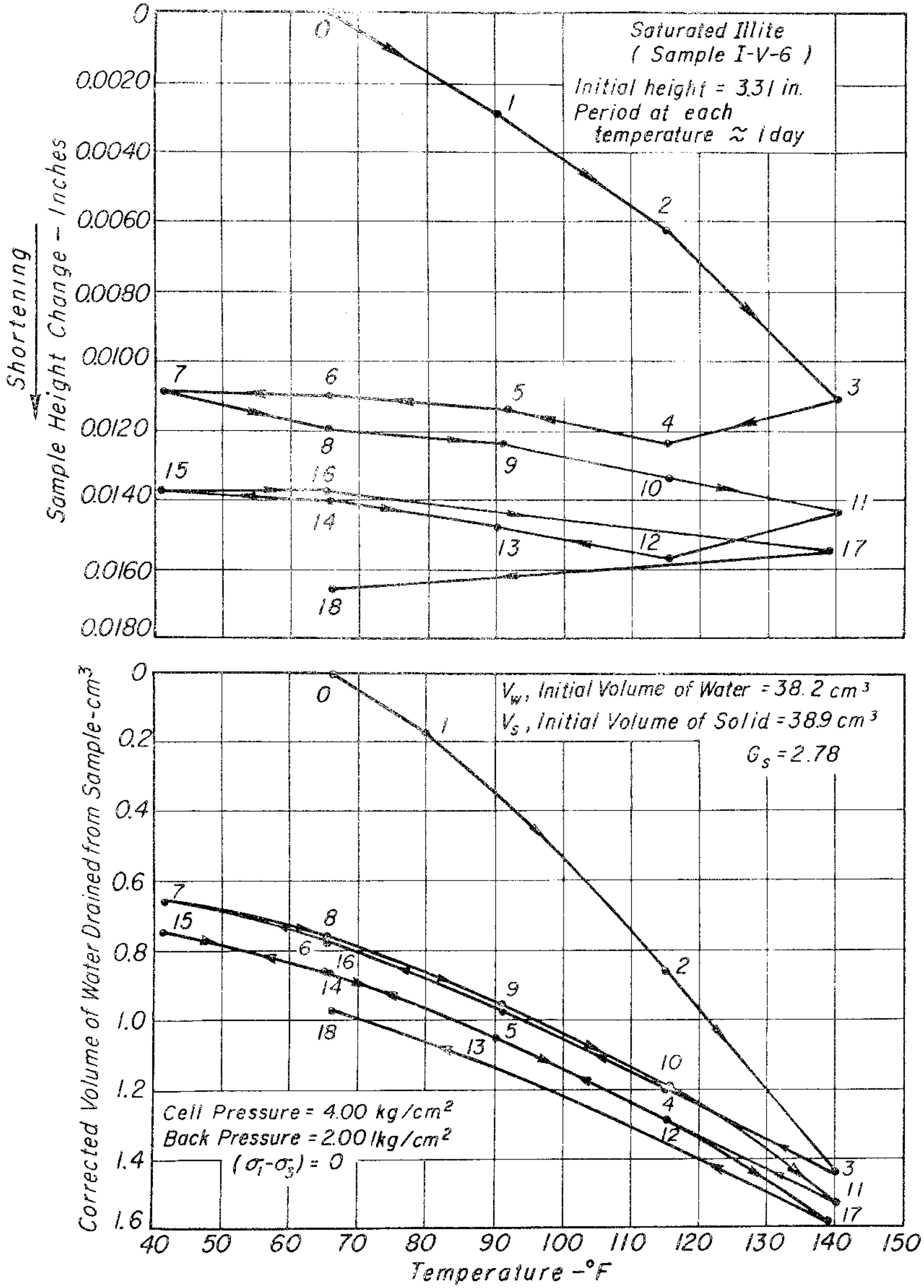


FIG. 8 EFFECT OF TEMPERATURE CHANGE ON SAMPLE HEIGHT AND VOLUME OF WATER DRAINED FROM A SATURATED ILLITE SPECIMEN

have been corrected for the thermal expansion and contraction of the water in the porous stone base and in the transmission line leading to the drainage pipette. Figs. 7 and 8 clearly show that the volume of water drained from the sample during initial temperature increases was very much larger than the volume of water absorbed into the sample when the sample temperature was returned to its initial value. In addition, both Figs. 7 and 8 suggest that after the initial temperature increases, the volume of water drained and absorbed by the sample was essentially reversible except at the higher temperatures.

The volume change of the sample due to a temperature change may be evaluated with the aid of Eq. 4. The thermal coefficient of cubical expansion of water is not constant but can be determined for a given change, $\alpha_w \Delta T$, (Handbook of Physics and Chemistry (16)), assuming clay water has the same volume change properties as pure water. The thermal coefficient for the mineral illite is not available; however, it can be approximated from measured thermal coefficient values reported for other minerals. The Handbook of Physics and Chemistry (40th Edition, pgs. 2239-2246) gives a range from $0.15 - 0.52 \times 10^{-4}$ per °C for substances such as SiO_2, Al_2O_3, Quartz, Slate, Marble, and Brick Fire Clay. A representative value for α_s would appear to be 0.35×10^{-4} per °C.

Consider the volume changes for Sample I-V-4. The values of $(\Delta V_{DR})_{\Delta T}$ for Sample I-V-4 can be obtained from the lower plot in Fig. 7 which also indicates the values of V_w and V_s. If temperature changes are referenced to the initial sample temperature, then the sample volume changes due to temperature changes, $(\Delta V_m)_{\Delta T}$, can be evaluated for each of the points shown in Fig. 7 by substituting the appropriate values into Eq. 4. For example, the sample volume change corresponding to point 1 in Fig. 7 is determined as follows.

From Point 0 to Point 1: $T_o = 65.2°F(18.4°C)$, $T_1 = 138.3°F(59.0°C)$

$$T_1 - T_o = \Delta T = 40.6°C$$

$$(\Delta V_m)_{\Delta T} = (\alpha_w \Delta T)V_w \qquad + \alpha_s V_s \Delta T \qquad - (\Delta V_{DR})_{\Delta T}$$

$$(\Delta V_m)_{\Delta T} = (.0165 - .0014)37.8 + .000035(38.6)40.6 - (1.300 - 0)$$

$$(\Delta V_m)_{\Delta T} = -0.675 \text{ cm}^3 \qquad \text{and} \qquad \frac{(\Delta V_m)_{\Delta T}}{V_m} = -0.88\%$$

The negative value for $(\Delta V_m)_{\Delta T}$ indicates a volume decrease. Computations proceed in the same manner for the other points in Fig. 7. The results of the computations are shown in the lower plot in Fig. 9. The upper plot in Fig. 9 shows the corresponding observations of height change against temperature. For comparison purposes the percent unit height change and percent unit volume change are indicated on the right side of the upper and lower plots, respectively.

The results in Fig. 9 show that the pattern of height change with temperature is exactly the same as the pattern of volume change with temperature. An analysis of the volume changes of Sample I-V-6 shows similar behavior and equally good agreement between height and volume changes.

The results in Fig. 9 indicate that a permanent volume decrease of about 1% occurred as a result of the initial temperature increase and decrease. Subsequent permanent volume decreases with temperature variations were small (about 0.1%) and occurred only at the high temperatures.

If temperature changes are again referenced to the initial sample temperature, then $(\Delta V_{st})_{\Delta T}$ can be evaluated for each of the points shown in Fig. 9 by substituting the appropriate values into Eq. 5. Values of

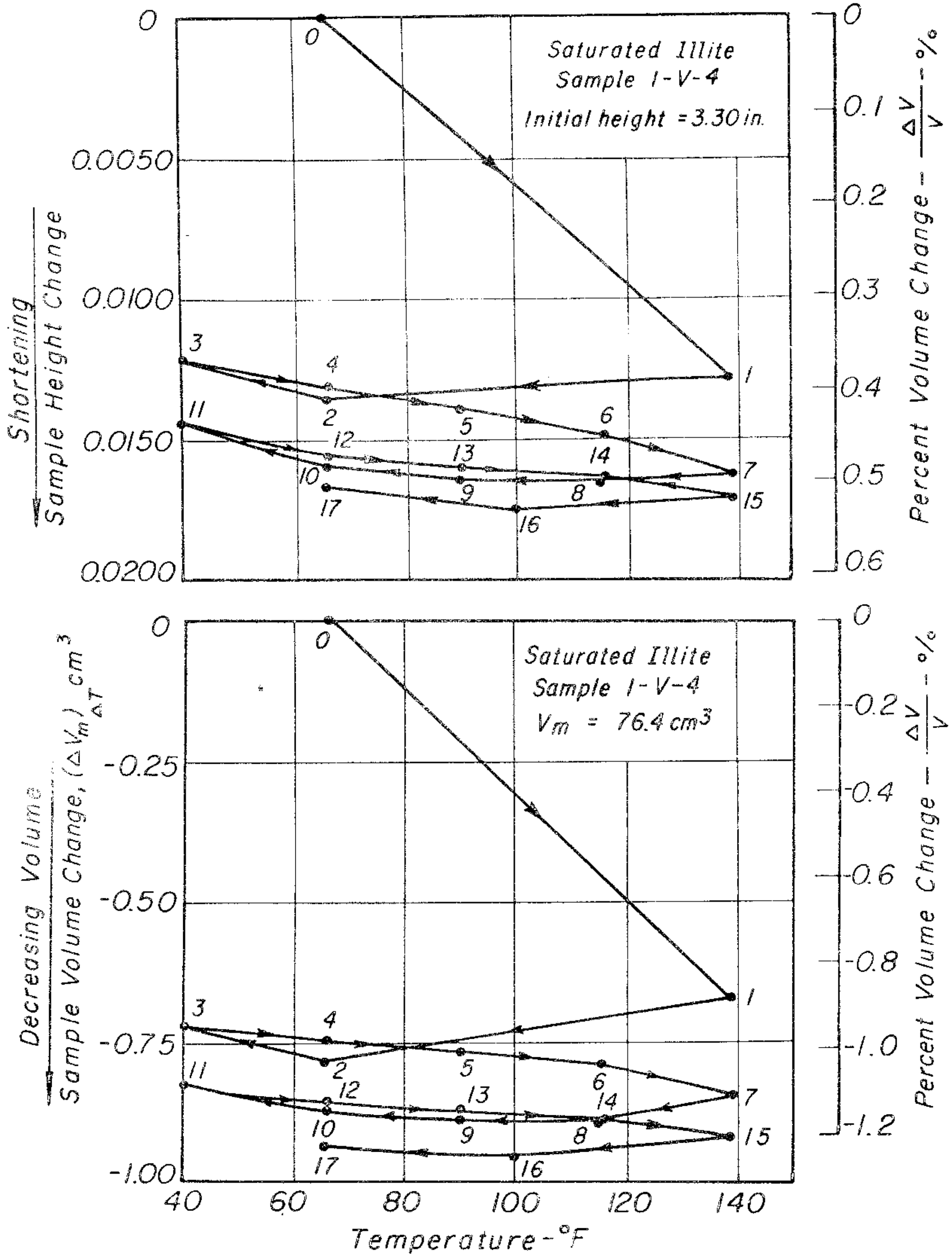


FIG. 9 EFFECT OF TEMPERATURE VARIATIONS ON HEIGHT AND VOLUME CHANGE

$(\Delta V_m)_{\Delta T}$ may be obtained from the lower plot in Fig. 9. The results of the computations for Sample I-V-4 are shown in the lower plot in Fig. 10 as volume change of clay structure against temperature. The upper plot in Fig. 10 shows the results of similar computations for Sample I-V-6. The percent unit volume change of clay structure is shown to the right of the plots in Fig. 10.

The results in Fig. 10 show that the decrease in volume of the clay structure was large only during initial temperature increases. Also, for the initial temperature increase from about 65 to 140°F the decrease in Unit volume of Sample I-V-6 (1.2%) was larger than the decrease in unit volume of Sample I-V-4 (1.0%). This behavior was most likely due to time effects, since Sample I-V-6 was initially increased in three temperature increments which took three days while Sample I-V-4 was subjected to only one temperature increase which took one day. Further permanent volume decreases in subsequent heating cycles were small (about .1% in each cycle), but occurred only at the high temperature.

This temperature history effect on volume change is similar in form to the stress history effect on volume changes during consolidation. The similarity between volume change-time behavior during temperature variations and volume change-time behavior during stress variations was previously noted. Thus, a very good analogy may exist between volume changes caused by temperature changes and those caused by stress changes. This implies that subjecting a soil to a temperature increase cycle may be equivalent to overconsolidation. The permanent volume decrease of 1% indicated for the initial temperature cycle (65 to 140 to 65°F) corresponds to a void ratio decrease of about 0.02. The results of isotropic consolidation tests on illite showed that a void ratio decrease of .02 resulted

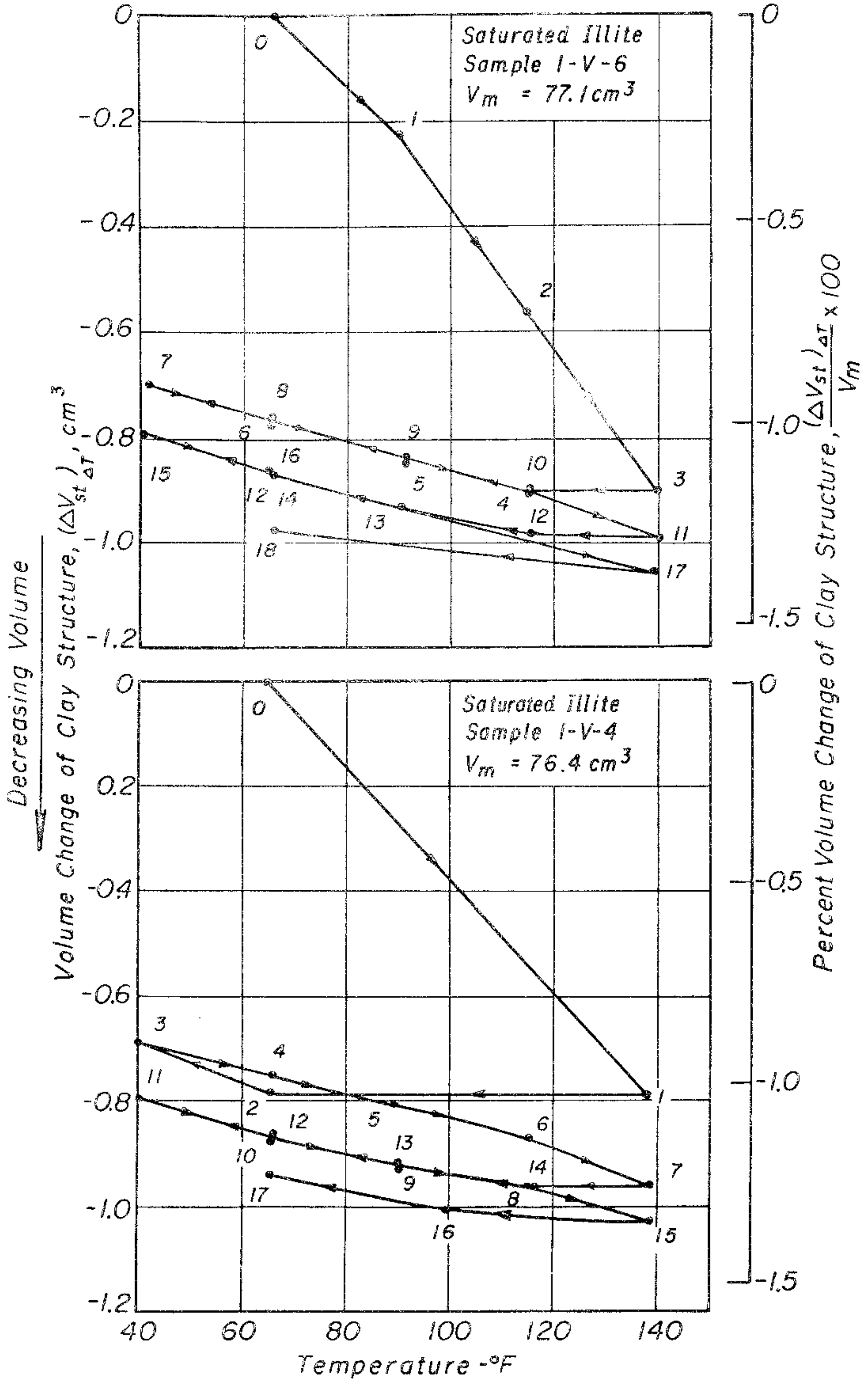


FIG. 10 EFFECT OF TEMPERATURE VARIATIONS ON VOLUME CHANGE OF CLAY STRUCTURE DUE TO PHYSICO-CHEMICAL PHENOMENA.

when the effective stress was increased from 2.00 to 2.25 $\mathrm{kg/cm^2}$ during virgin compression.

The data in Fig. 10 indicate that after initial temperature changes the volume change of the clay structure was reversible except in the higher temperature range. These deviations; e.g., in the upper plot from points 3 to 4, 11 to 12, and 17 to 18, are as yet unexplained and require further study. The deviations become less pronounced with increasing numbers of temperature cycles, however.

Coefficient of Thermal Expansion for the Soil Structure

The reversible changes in clay structure volume with temperature variations indicate a swelling of the clay structure during temperature decrease and a compression during temperature increase. This is just the reverse of what would be anticipated if the soil structure simply expanded and contracted in response to the volumetric strains of the soil grains alone. Temperature dependent physico-chemical phenomena must be superimposed on the normal thermal expansions and contractions in such a manner that the physico-chemical effects dominate. Possible effects are greater water adsorption force fields and higher osmotic pressures at lower temperatures, both of which could account for swelling on cooling. No specific interpretation is possible on the basis of the data available, however.

The reversible pattern of changes in clay structure volume shown in Fig. 10 was essentially linear with temperature throughout the temperature range of about 40 to 115°F and was repeatable during successive temperature cycles after the first heating. The slope of these reversible curves with respect to unit volume change may be used to determine the physico-chemical coefficient, α_{st}, defined previously as

$$\alpha_{st} = \frac{\Delta V_{st}/V_m}{\Delta T} \qquad . \qquad . \qquad . \qquad . \qquad (17)$$

For the two samples shown in Fig. 10, α_{st} has a value of about -0.5×10^{4} per °C. The negative sign indicates compression during temperature increase. This value is relatively small and its effect on volume changes is also small. For example, in the case of saturated illite under drained conditions an increase in temperature from 70 to 110°F would cause a unit volume decrease of about 0.1% which would be reversible during an equivalent temperature decrease.

Influence of Temperature on Consolidation

In order to investigate further the effect of temperature on volume change of saturated clay, triaxial consolidation tests were performed on three remolded illite specimens, each at a different temperature for the entire duration of the test. The samples were consolidated initially to an effective stress of 2 kg/cm^2 at temperatures of 76.5, 100, and 124.5°F, and maintained under this stress for 5800 minutes to insure that additional secondary compression effects due to the initial stress would be negligible. At the end of this period the consolidation pressure was increased at intervals of 24 hours using load increment ratios of 0.20 to 0.27. The results of these consolidation tests are shown in Fig. 11. The plotted void ratios are absolute values for the given test temperature and correspond to the total volume change for a given stress change (24 hour reading).

Fig. 11 shows that the higher the temperature the lower the void ratio after the initial stress application of 2 kg/cm^2. Since the void ratio of all samples was essentially the same prior to the initial stress application, the successively lower void ratios were, therefore, due to

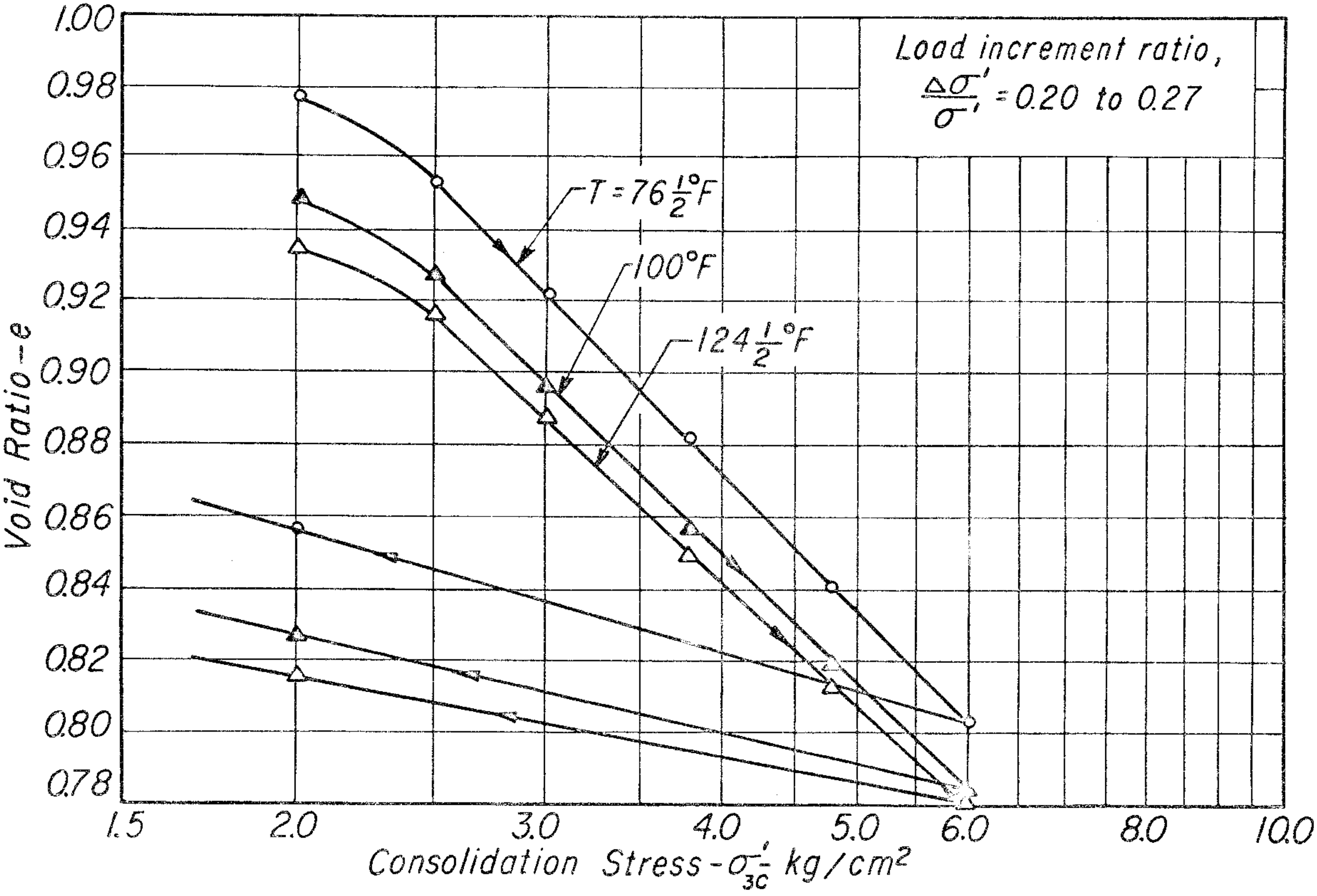


FIG.11 EFFECT OF TEMPERATURE ON ISOTROPIC CONSOLIDATION BEHAVIOR OF SATURATED ILLITE

the higher initial temperatures. This order of void ratios would be expected because the weaker soil structures at the higher temperatures would have to densify more in order to carry the effective stresses.

Fig. 11 also shows that all curves were linear during virgin consolidation except for the pressure increment from 2 to 2.5 kg/cm^2. This "bend" in the curve is interpreted to have been due to the small stress increment ratios, which created an apparent "quasi-preconsolidation pressure" resulting from the relatively long period of initial stress application, Leonards and Ramiah, (17).

Since all consolidation curves were essentially parallel, it may be concluded that the compressibility of remolded illite at a given stress is, for the most part, unaffected by temperature. Similar observations were previously reported by Finn (1), whose results indicated that the compressibility was not affected by temperature in the range 40 to 80°F during one-dimensional consolidation of remolded clay.

It appears, then, that the predominant effect of temperature on consolidation was to cause successively lower void ratios with increasing temperature for a given initial consolidation stress. Once equilibrium had been established between the soil structure, temperature, and this stress, the different temperatures had no significant additional effect on the amount of consolidation.

PORE PRESSURE BEHAVIOR

After cycling the temperature of the two saturated illite specimens under fully drained conditions at constant isotropic stress (Figs. 4 and 5), the drainage valves were closed. The temperature of both samples was then rapidly changed in 25°F intervals from 65°F to a maximum of 140°F and a minimum of 40°F for two complete cycles. The samples were maintained at each temperature for 100 minutes, which experience has shown to be ample time for the test specimen to attain equilibrium. Fig. 12 shows the results in terms of pore pressure versus temperature.

Essentially identical behavior was observed for each sample, and after the initial temperature increase the relationship between pore pressure and temperature was repeatable and formed a closed hysteresis loop. It may be seen that relatively small changes in temperature caused substantial changes in pore pressure, with temperature increases causing pore pressure increases and temperature decreases causing pore pressure decreases.

The previous analysis of temperature induced pore pressures led to equations (12), (13), (15), (16) and (16a), any one of which could be used for prediction of pore water pressure changes accompanying temperature changes under undrained conditions. While equation (16a) is perhaps the most useful for general application if α_{st} has been determined, equation (13) will be used here. The reason for this is that for the test data to be analyzed the volume of water in the porous stone and in the pore pressure measuring system must be accounted for as well as the pore water in the sample itself.

The value of m_v to be used in Eq. 13 is related to the consolidation and rebound characteristics of the soil. When the temperature of a sample is increased, pore pressures increase and effective stresses decrease which would cause rebound if drainage were permitted. When the sample temperature is decreased, pore pressures decreased and effective stresses increase. Since the previous temperature history resulted in permanent volume decreases at the higher temperature, sample behavior during the undrained portion of the tests should be more closely related to recompression than to virgin compression. On this basis a representative value for m_v may be determined from the rebound curve, since it is known that the slope along rebound curves is approximately the same as the slope along recompression curves at a given value of effective stress. Since rebound curves are usually approximately linear when void ratio is plotted against logarithm of effective stress, the following relationship exists.

$$(m_v)_R = \frac{\Delta V_m/V_m}{\Delta\sigma'} = \frac{0.435}{(1+e_o)} \frac{C_R}{\sigma'} \tag{18}$$

where C_R = slope of the linear rebound curve (swelling index) and is equal to the change in void ratio for one log cycle change in stress,

e_o = initial void ratio, and

σ' = effective stress at which $(m_v)_R$ is evaluated.

Consider, as an example, the pore pressure change resulting from a temperature increase of 70 to 110°F (21.1 to 43.4°C) applied to Sample I-V-4. An approximate value of C_R for remolded illite can be determined from the consolidation test results shown in Fig. 11. The average value of C_R for these three tests was -.089 for stress in units of kg/cm^2. The value of $(\Delta V_{st})_{\Delta T}$ as determined from the last cycle of temperature increase from 70 to 110°F for the drained test in Fig. 10 was -.08 cm^3, and the compressibility of water is -4.83×10^{-5} per kg/cm^2. All other physical factors for the specimen are the same as given previously except V_w, which now must equal the volume of pore water (36.8 cm^3) plus the volume of water in the porous stone and in the transmission line up to the drainage valve (7.9 cm^3), or a total of 44.7 cm^3.

Combining Eqs. 13 and 18, and substituting the appropriate values for the initial temperature increase from 70 to 110°F for Sample I-V-4 gives

$$\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - \alpha_s V_m \Delta T - (\Delta V_{st})_{\Delta T} = - \frac{.435 C_R}{1 + e_o} V_m \frac{\Delta u}{\sigma'} - m_w V_w \Delta u \qquad (19)$$

$$.323 + .030 - .060 + .080 = .78\ \Delta u + .002\ \Delta u$$

$$\Delta u = +\ 0.48\ \mathrm{kg/cm^2}$$

Fig. 12 shows that the experimental change in Δu for Sample I-V-4 over the range from 70° to 110°F during the first cycle of temperature increase was about + 0.54 $\mathrm{kg/cm^2}$. Reasonably good agreement has been obtained using equation (19) for calculation of pore pressures for all points in Fig. 12. The major difficulty associated with precise calculation of temperature induced pore pressures is the determination of the appropriate value of C_R. Small changes in C_R give large changes in calculated pore pressure and the precise value of C_R corresponding to any given value of effective stress is not always known. Thus theoretical predictions of pore pressure changes can be expected to be only approximately correct.

It should be noted also that as σ' becomes very small C_R becomes very large. Thus theory predicts that at very small values of effective stress temperature induced pore pressures become small. This prediction was corroborated by experiment where it was found that as pore pressures approached the confining cell pressure, temperature changes had little effect on pore pressures.

The change in volume of the soil structure, $(\Delta V_{st})_{\Delta T}$, may also be an important factor contributing to pore pressure changes. Any tendencies for large decreases in soil structure volume, as was observed for the

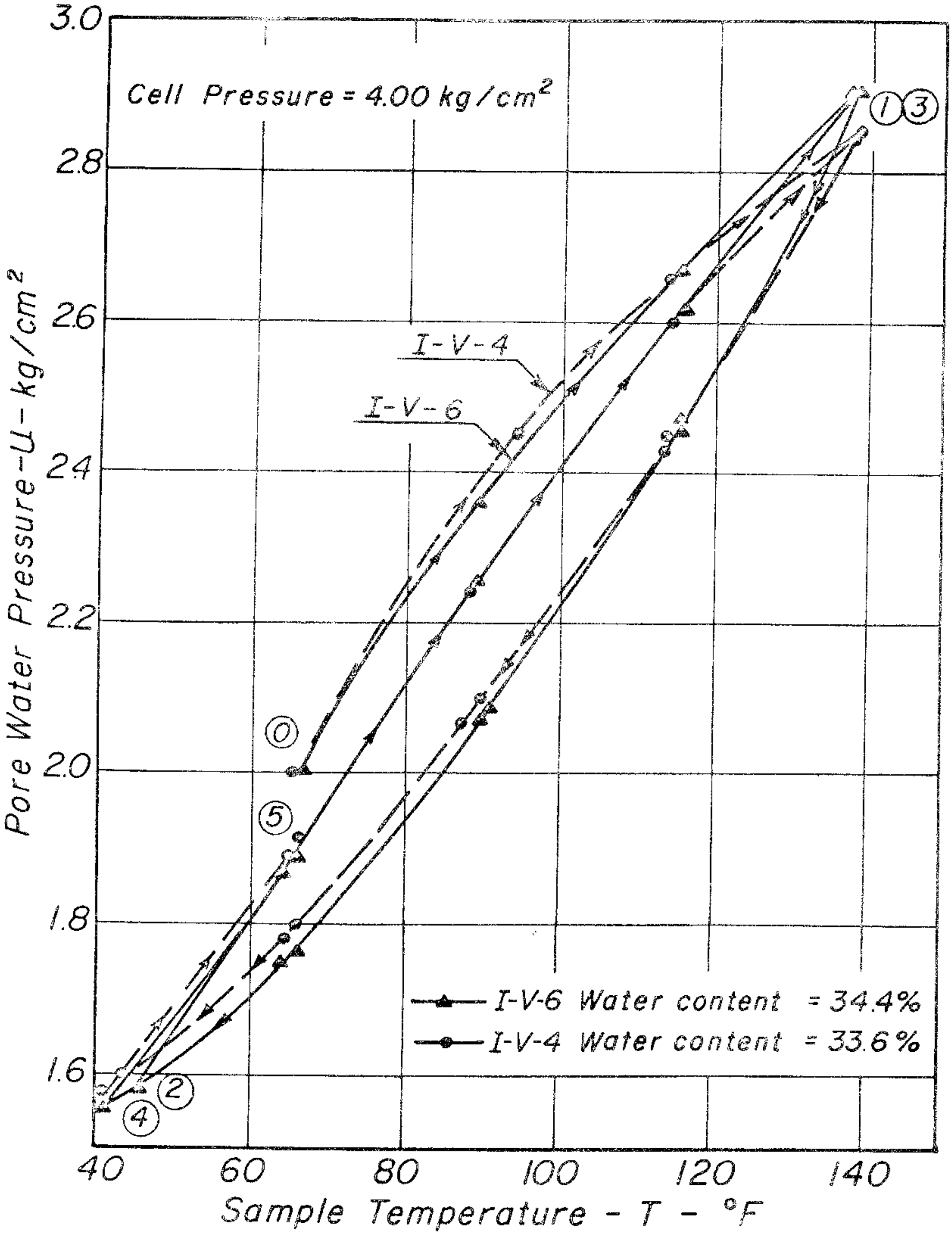


FIG. 12 EFFECT OF TEMPERATURE ON PORE WATER PRESSURE IN SATURATED ILLITE SPECIMENS TESTED UNDER UNDRAINED CONDITIONS

initial temperature increases during drained conditions, would result in correspondingly large increases in pore pressure. Also, since these decreases in sample volume are likely to be, for the most part, permanent, pore pressure increases would also be, for the most part, permanent as long as drainage is prevented.

PORE PRESSURE PARAMETER

From the foregoing analysis a pore pressure-temperature parameter, F, may be defined as the change in pore pressure per unit change in temperature per unit effective stress, or alternately, the change in unit effective stress per unit change in temperature. This is expressed as

$$F = \frac{\Delta u}{\Delta T}/\sigma' = \frac{-\Delta\sigma'}{\sigma'}/\Delta T = \frac{e_o\left[(\alpha_s - \alpha_w) + \frac{\alpha_{st}}{n}\right]}{.435\ C_R} \quad . \quad . \qquad (20)$$

where σ' = effective stress at the time of the temperature change.

The parameter F depends mainly on soil compressibility and the change in volume of the pore water due to a given temperature change. The use of the parameter F may provide a practical tool for comparison of temperature induced pore pressures for different soils under different test conditions.

Recent publications (Ladd (6) ; Mitchell and Campanella (11); and Henkel and Sowa (12)) have included temperature induced pore pressure data. These data have been summarized in Table 1 together with those of the writers. The values given for σ' and Δu are average values for the indicated temperature changes.

The influence of effective stress on the change in pore pressure with temperature change is indicated by the results for both the Vicksburg Buckshot clay and saturated porous stone specimens. The higher the

TABLE 1

Temperature Induced Pore Pressure Changes--Undrained Triaxial Test Conditions at Constant Cell Pressure and Zero Deviator Stress.

Soil Type	σ' kg/cm^2	Δu kg/cm^2	ΔT °F	F $\frac{\Delta u}{\Delta T}/\sigma'$ $°F^{-1}$	Reference
Illite (Grundite)	2.0	+0.58	70 to 110	.0073	Writers
S.F. Bay Mud	1.5	+0.50	70 to 110	.0083	Writers
Weald Clay	7.1	+0.51	77 to 84.2	.0099	Henkel and Sowa, 1963
Kaolinite	2.0	+0.78	70 to 110	.0097	Mitchell and Campanella, 1963
Vicksburg Buckshot Clay	1.0 6.5	+0.28 +1.90	68 to 96.8 68 to 96.8	.0097 .0101	Ladd, 1961, - Fig. VIII-6
Saturated Porous Stone (Sandstone)	2.5 5.8	+1.9 +5.2	41.5 to 59 41.5 to 59	.044 .051	Writers

effective stress the higher was the change in pore pressure for the same change in temperature. This is as predicted by Eq. 19. It may be noted, however, that for a given soil the parameter F was relatively unaffected by different values of effective stress, suggesting that F may be a useful indicator of temperature induced pore pressures.

Table 1 also shows that the F parameter for the porous stone was about seven times greater than the F parameter for the illite clay. This is to be expected according to equation 16(a) since the compressibility of the porous stone is considerably less than the compressibility of illite.

The data in Table 1 indicate that the parameter F may be approximately the same for different clays, as it only varied from about .0075 to .0100 per °F. From a practical standpoint, knowledge of F values allows the determination of the temperature control required for undrained test conditions in order to insure accurate and reliable pore pressure measurements. For example, if it were desired to keep pore pressure fluctuations due to temperature changes to within ±.05 kg/cm^2 for one of the clays in Table 1, the required temperature control would be about ±1°F for a sample at an effective stress of 5 kg/cm^2.

HYSTERESIS EFFECT

The data in Fig. 12 show the formation of a closed loop pore pressure-temperature hysteresis effect with continuous temperature cycling. The portion of the hysteresis loop along which temperatures were decreasing had lower pore pressures at comparable temperatures than the portion of the loop along which temperatures were increasing. The observed repeatable behavior precludes the possibility of leakage being present at either the membrane seals or at the drainage valve.

Similar closed loop hysteresis behavior was reported by Mitchell and Campanella (11) for saturated kaolinite and is shown in Fig. 13. It may be noted, however, that for the undrained test on kaolinite the sample temperature was initially decreased as opposed to the tests on illite (Fig. 12) in which the sample temperature was initially increased. The observed pore pressure-temperature hysteresis behavior was similar in each case.

In a discussion to Mitchell and Campanella (11), Henkel and Sowa (12) also presented data showing the effect of temperature changes on pore pressures for an undrained sample maintained under constant isotropic stress. Henkel and Sowa reported that, unlike the results in Figs. 12 and 13, the pore pressure-temperature relationship did not form a closed hysteresis loop for Weald clay, but instead, a residual pore pressure was built up as shown in Fig. 14. Henkel and Sowa (12) presented evidence to show that membrane leakage was negligible and could not be the cause of the residual pore pressure increase.

The residual pore pressure build-up shown in Fig. 14 is predicted by Eqs. 16a and 19 if the factor $(\Delta V_{st})_{\Delta T}$ is significant and reflects a tendency for the volume of the soil mass to decrease with increase in temperature. As previously discussed, small but significant permanent increase in the magnitude of $(\Delta V_{st})_{\Delta T}$ can result from irreversible physico-chemical phenomena at elevated temperatures, thus causing residual pore pressure increases.

In order to pursue this hypothesis further, an undrained test was performed in which temperature-pore pressure behavior was observed for a specimen of undisturbed San Francisco Bay Mud which was not subjected to previous temperature changes. Bay mud and remolded illite have similar

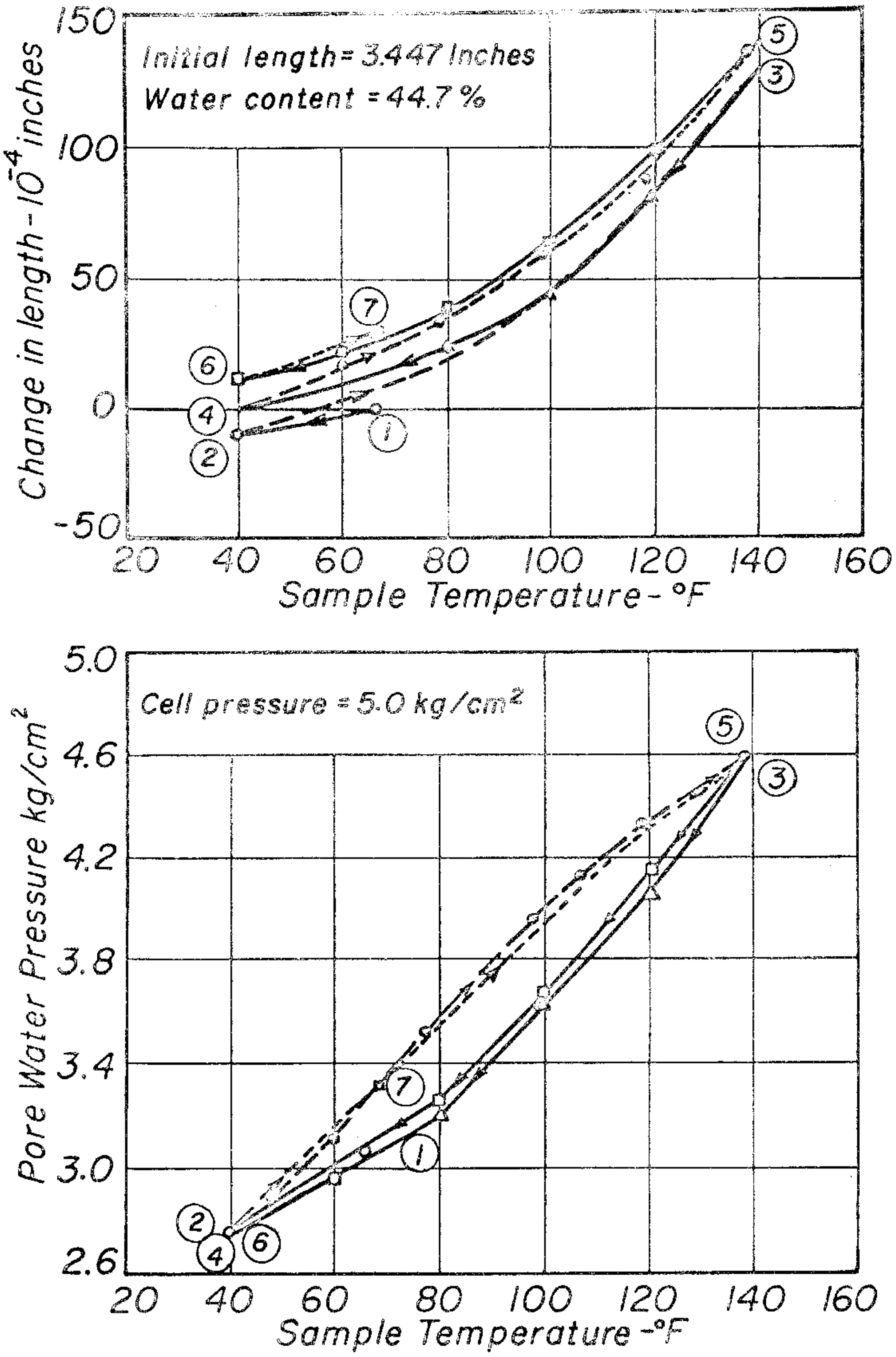
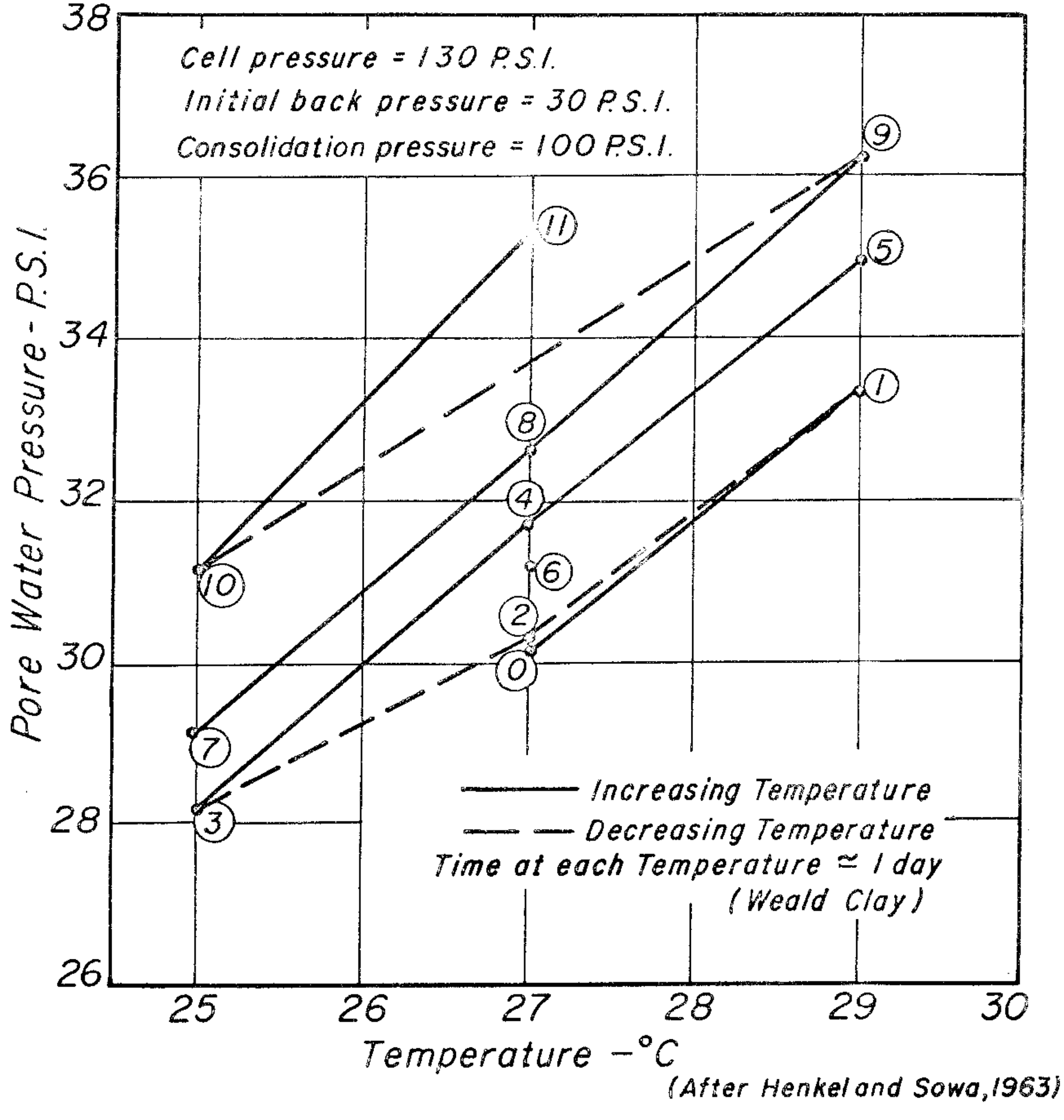


FIG. 13 THE EFFECT OF TEMPERATURE ON SAMPLE LENGTH AND PORE WATER PRESSURES IN SATURATED KAOLINITE TESTED UNDER UNDRAINED CONDITIONS.



(After Henkel and Sowa, 1963)

FIG.14 EFFECT OF TEMPERATURE ON PORE PRESSURE UNDER UNDRAINED CONDITIONS AND HYDROSTATIC STRESS secondary compression rates after consolidation. The bay mud was initially consolidated to 2.00 kg/cm^2 after which pore water drainage was stopped and pore pressures allowed to equalize. The sample temperature was then changed in 20°F increments and maintained at each temperature for 120 minutes. The relationship between pore pressure and sample temperature is shown in Fig. 15. It is immediately evident that a residual pore pressure was developed after temperature cycling. However, upon returning the temperature to its initial value the net rise in pore pressure became considerably less as temperature cycling progressed. Furthermore, the data suggest that a closed hysteresis loop would have developed with continued temperature cycling and the dissipation of secondary compression tendencies at high temperatures. At the end of the test the temperature was maintained constant for 5 days during which the pore pressure increased only .03 kg/cm^2, an amount easily attributed to residual secondary compression effects.

For the test results reported in Fig. 12, however, the illite specimens were subjected initially to extensive temperature changes under drained conditions. Calculations showed that for these samples the value $(\Delta V_{st})_{\Delta T}$ was insignificant for the subsequent temperature changes under undrained conditions. Thus, no residual pore pressure build-up was observed (Fig. 12). In addition the test results shown in Fig. 13 indicated no residual pore pressure build-up, but in this case the kaolinite tested showed essentially no secondary compression effects after consolidation suggesting that this clay was physico-chemically inactive. Hence, the value $(\Delta V_{st})_{\Delta T}$ would be insignificant for this soil. It appears then, that open loop hysteresis effects are evidenced only in

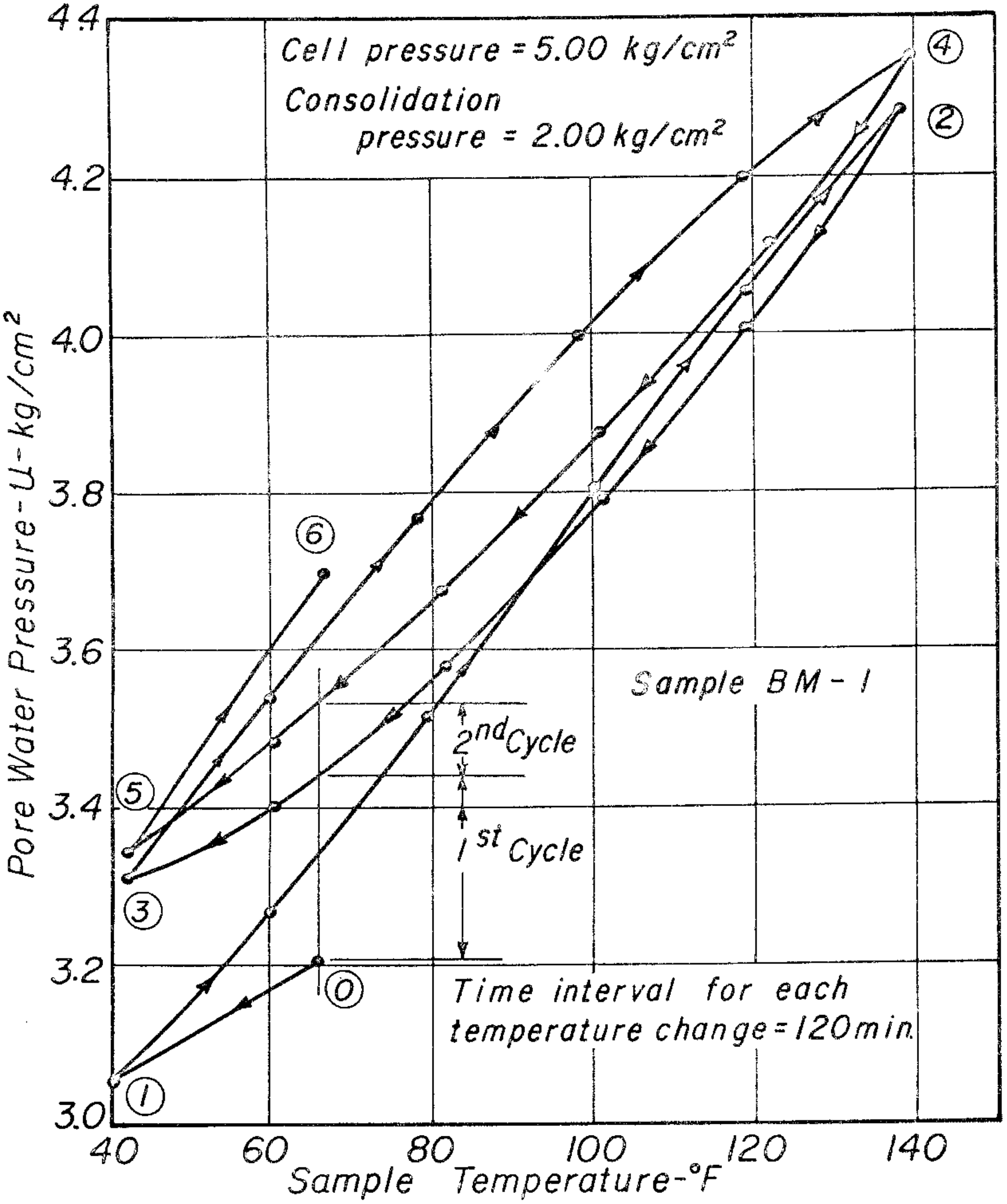


FIG.15 EFFECT OF TEMPERATURE ON PORE PRESSURE IN SATURATED SAN FRANCISCO BAY MUD TESTED UNDER UNDRAINED CONDITIONS

those cases when temperature induced secondary compression effects are significant.

Hysteresis behavior was observed regardless of the time at which a sample was maintained at a given temperature; thus, it cannot be attributed to nonequilibrium conditions. It may be noted also that for the undrained test on Kaolinite (Fig. 13) the plot of change in sample length versus temperature was essentially reversible and showed no hysteresis behavior. This was true also for the illite and bay mud specimens reported herein.

As shown in the previous computation of pore pressure changes accompanying temperature changes, the major factors responsible for pore pressure changes are the soil compressibility and the changes in pore water volume. Changes in pore water volume with temperature should be reversible and not show any hysteresis effects unless there are unusual physico-chemical influences on water density. Previous discussion has indicated that when temperature changes cause effective stress changes the sample is either rebounding or recompressing with respect to virgin compression. These considerations may be used to developing an interpretation of the hysteresis effect.

Eq. 4 indicates that in undrained tests the volume change of the soil mass due to a temperature change is equal to the sum of the expansion or contraction of pore water and mineral solids, since $(\Delta V_{DR})_{\Delta T}$ equals zero. If temperature changes are referenced to the initial sample temperature, then the sample volume change can be evaluated for each of the points in Fig. 12 for Sample I-V-4. The effective stress, σ_3', can also be determined from the observed pore pressures shown in Fig. 12 since the cell pressure was constant. The results are shown in Fig. 16

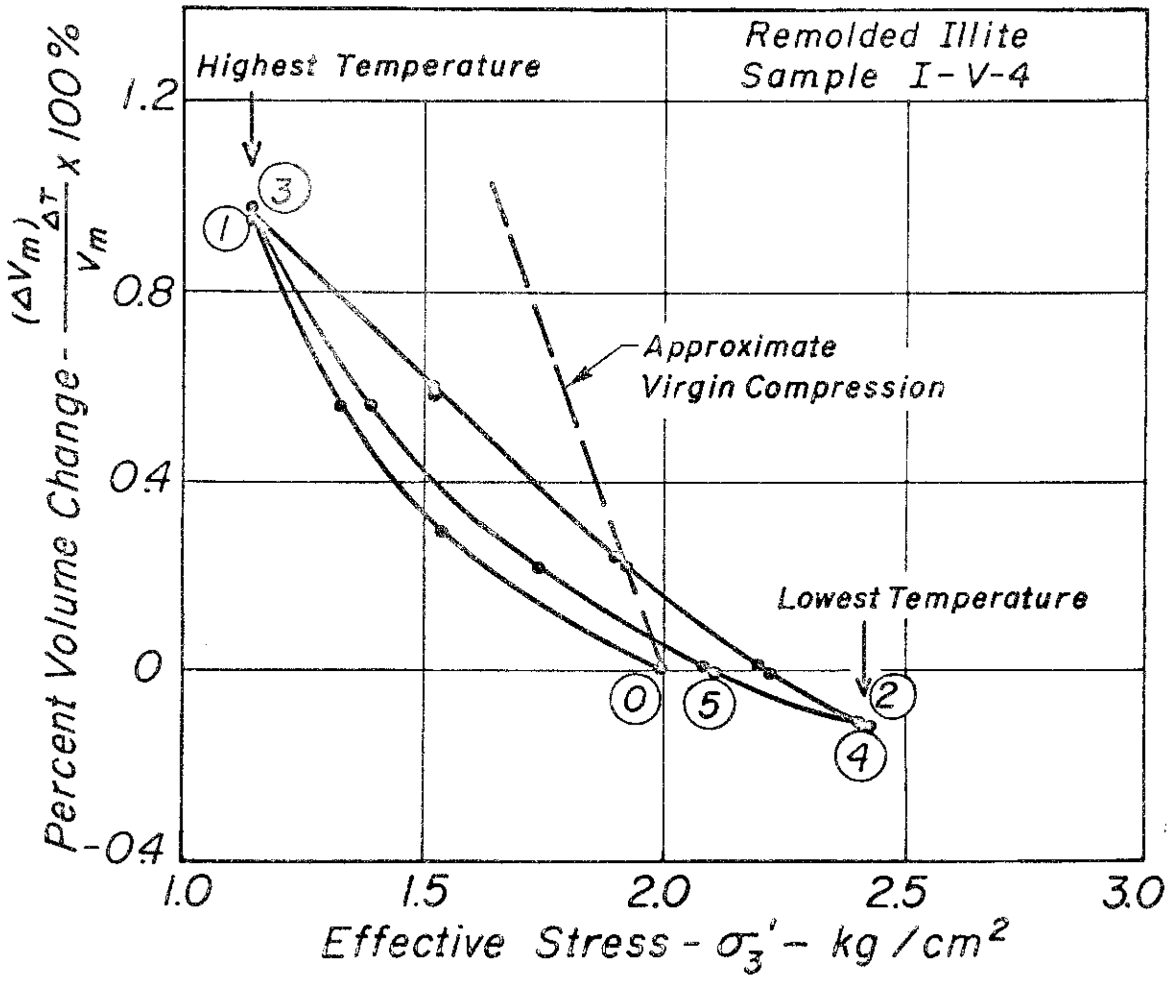


FIG. 16 UNIT VOLUME CHANGE VERSUS OBSERVED EFFECTIVE STRESS DURING TEMPERATURE CHANGES UNDER UNDRAINED CONDITIONS

in the form of percent volume change against observed effective stress. Also indicated on the plot is the approximate location of the virgin compression curve, indicating that during temperature variations under undrained conditions the soil was either rebounding or recompressing with respect to virgin compression. Since temperature cycling causes repeated rebound and recompression, it appears that the observed temperature-pore pressure hysteresis behavior may be explainable in terms of the observed hysteresis associated with the rebound and recompression portions of a consolidation curve.

A typical consolidation curve showing unit volume change against effective stress is shown in Fig. 17. The slope of the curve at any point is the soil compressibility which has a different value at each effective stress. If the temperature of an undrained sample is increased, the pore pressures increase and effective stresses decrease. Therefore, the sample behaves as if unloaded along the rebound curve 0-1-2 as shown in Fig. 17. If the sample temperature is then decreased, the pore pressures decrease and the effective stresses increase; therefore, the sample behaves as if reloaded along the recompression curve 2-3-4. Comparison of the slopes of the rebound and recompression curves just before and after the temperature reversal at point 2 shows that m_v is larger during increasing temperature than during decreasing temperature. Since Eq. 16a indicates that pore pressure changes are inversely proportional to m_v, the pore pressure after the temperature reversal should be lower than the pore pressure before the temperature reversal when compared at the same temperature. This was precisely the observed temperature-pore pressure hysteresis behavior shown in Figs. 12, 13, and 15. Similar reasoning concerning the change in compressibility during

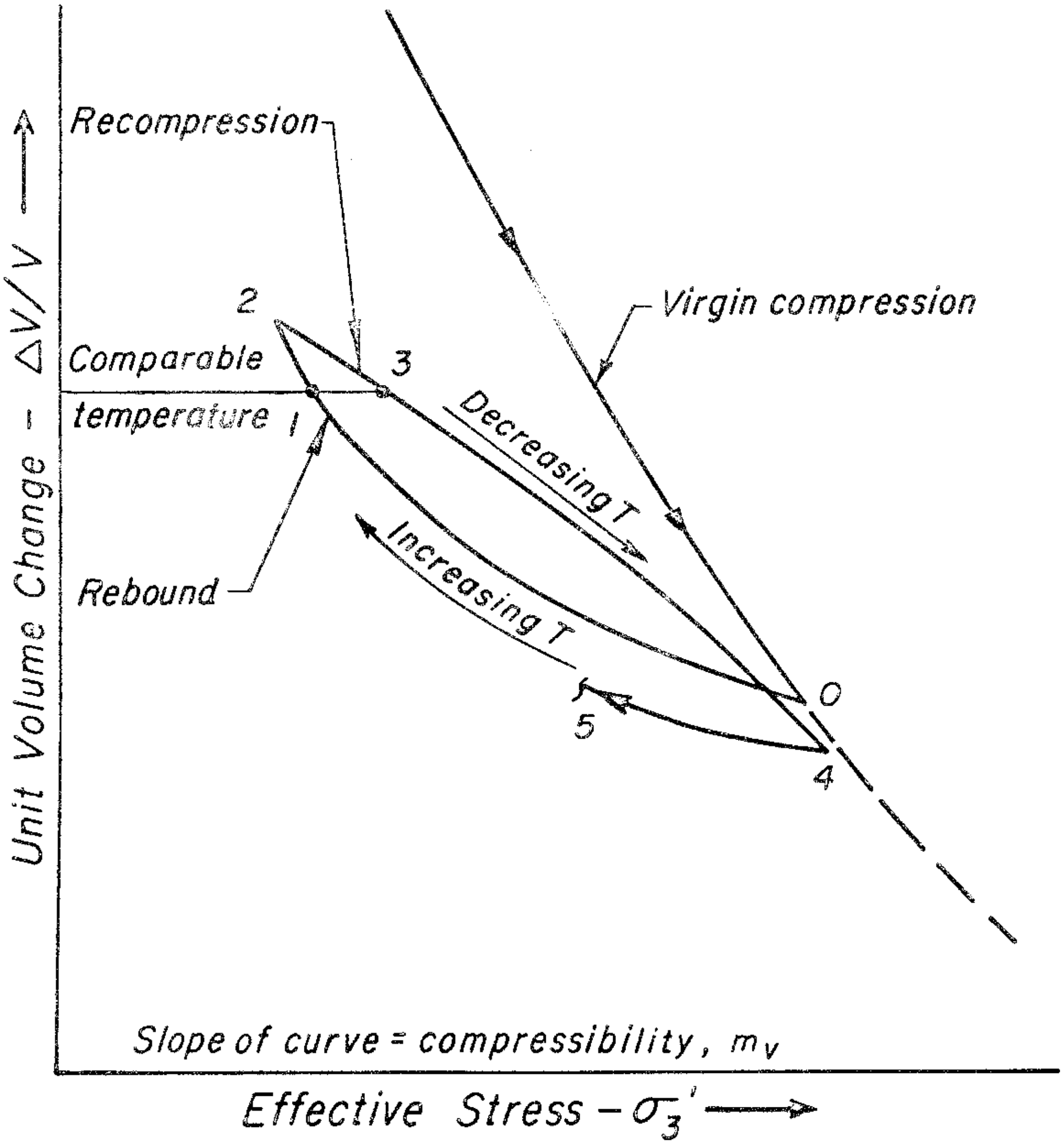


FIG. 17 TYPICAL RELATIONSHIP BETWEEN UNIT VOLUME CHANGE AND EFFECTIVE STRESS DURING REBOUND AND RECOMPRESSION

the temperature reversal at the low temperature (points 3-4-5) also explains the observed temperature-pore pressure hysteresis.

There are two other factors which could affect the shape of the pore pressure-temperature relationship. One is the nonlinearity of changing pore water volume with temperature, which is reversible. The other is the boundary condition which dictates that pore pressures cannot exceed the cell pressure. Neither of these could explain the observed hysteresis behavior.

SUMMARY AND CONCLUSIONS

Analyses have been presented for the interpretation of volume changes and pore pressure changes due to temperature variations in saturated soils. These analyses consider volume changes due to thermal expansion of the soil components, compressibility of the soil, and physico-chemical effects.

The results of drained triaxial tests on saturated remolded illite showed that significant permanent volume decreases may occur during temperature variations but only during initial temperature increases. This behavior is explained in terms of irreversible physico-chemical structural adjustments required so that the soil can carry the applied effective stress. The results also indicate that a complete analogy may exist between temperature induced volume changes and stress induced volume changes, i.e. consolidation,* suggesting that a temperature increase cycle may be equivalent to overconsolidation. This effect, however, is likely to be small. The results of isotropic consolidation tests at different temperatures showed that the compression index of

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*A similar conclusion has also been reached by Paaswell (2).

remolded illite was essentially independent of temperature; however, the higher the temperature the lower the void ratio at any given consolidation pressure.

The results of undrained tests on three saturated clays showed that significant pore pressure increases developed during temperature increases and pore pressure decreases developed during temperature decreases, forming a hysteresis loop.

Theoretical analysis provided a reasonably good estimate of pore pressure changes and indicated them to be dependent mainly on soil compressibility and pore water volume change caused by a temperature change. The observed temperature-pore pressure hysteresis behavior can be explained in terms of changes in soil compressibility during rebound and recompression.

A temperature induced pore pressure parameter, F, was defined as the change in pore pressure per unit change in temperature per unit effective stress. The results of many tests on different saturated clays showed that F had a range of only .0075 to .010 per °F.

From a practical standpoint, pore pressure changes due to temperature variations may be significant. When saturated soil is removed from the field and taken to the laboratory, the temperature of the soil will usually increase since the field temperature of the soil is often less than normal laboratory temperature. The temperature increase will cause an increase in pore pressure and a decrease in effective stress. If unconsolidated-undrained tests are performed on the samples, the measured strength will be too low. This effect has been discussed by Duncan and Campanella (9). Knowledge of the F parameter of a soil allows the change in effective stress due to temperature variations to be approximated and an estimate to be made of the laboratory temperature control that must be provided if pore pressure measurements during undrained testing are to be meaningful.

Acknowledgements

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APPENDIX I

References

1. Finn, F. N., "The Effect of Temperature on the Consolidation of Soils," ASTM Special Technical Publication No. 126, June 1951, pp. 65-72.

2. Paaswell, R. E., "Temperature Effects on Clay Soil Consolidation," Journal of the Soil Mechanics and Foundations Division ASCE, Vol. 93, No. SM3, May 1967, pp. 9-22.

3. Lambe, T. W., "The Structure of Inorganic Soil," Proceedings of the American Society of Civil Engineers, No. 315, 1953.

4. Lambe, T. W., "The Structure of Compacted Clay," Transactions, American Society of Civil Engineers, Reprint Paper 3041, Vol. 125, 1960.

5. Lambe, T. W., "Residual Pore Pressures in Compacted Clay," Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1961, p. 207.

6. Ladd, C. C., "Physico-Chemical Analysis of the Shear Strength of Saturated Clays," D.Sc. Dissertation, M.I.T., Cambridge, Mass., 1961.

7. Scott, R. F., Principles of Soil Mechanics, Addison-Wesley Publishing Company, Inc., 1963.

8. Mitchell, J. K., "Shearing Resistance of Soils as a Rate Process," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM1, Proceedings Paper 3773, Jan. 1964.

9. Duncan, J. M. and Campanella, R. G., "The Effect of Temperature Changes During Undrained Tests," Report TE-65-10 to U.S. Army Engineers Waterways Experiment Station, Soil Mechanics and Bituminous Materials Research Laboratory, University of California, Berkeley, Nov. 1965.

10. Murayama, S. and Shibata, T., "Rheological Properties of Clays," Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, 1961, pp. 269-273.

11. Mitchell, J. K. and Campanella, R. G., "Creep Studies on Saturated Clays," Symposium on Laboratory Shear Testing of Soils, ASTM-NRC, Ottawa, Canada, ASTM Special Technical Publication No. 361, Sept. 1963.

12. Henkel, D. J. and Sowa, V. A., Discussion, Symposium on Laboratory Shear Testing of Soils, ASTM-NRC, Ottawa, Canada, ASTM Special Technical Publication No. 361, Sept. 1963.

13. Andersland, O. B. and Akili, W., "Stress Effect on Creep Rates of a Frozen Clay Soil," Geotechnique, Vol XVII, No. 1, March 1967, pp. 27-39.

14. Mitchell, J. K., Campanella, R. G. and Singh, A., "Soil Creep as a Rate Process," in-press.

15. Leonards, G. A. and Girault, P., "A Study of the One-Dimensional Consolidation Test," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, Paris, 1961, p. 213.

16. Handbook of Chemistry and Physics, 40th Edition, Chemical Rubber Publishing Co., 1958-59.

17. Leonards, G. A. and Ramiah, B. K., "Time Effects in the Consolidation of Clay," Symposium on Time Rate of Loading in Soils, ASTM Special Technical Publication No. 254, 1959.