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THE EFFECT OF TEMPERATURE CHANGES DURING UNDRAINED TESTS

by

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CORPS OF ENGINEERS
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A Report of an Investigation by

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and

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FOREWORD

The work described in this report was performed under Contract No. DA-22-079-CIVENG-62-47, "Shear Properties of Undisturbed Weak Clays," between the U. S. Army Engineer Waterways Experiment Station and the University of California.

The general objective of the research, which was begun in February, 1962, is to investigate the influence of pore-water pressures on the strength characteristics of undisturbed weak clays. Work on this project is conducted under the supervision of Professor H. Bolton Seed, Professor of Civil Engineering, and Clarence K. Chan, Associate Research Engineer. The project is administered by the Office of Research Services of the College of Engineering.

The phase of the investigation described in this report was conducted by J. M. Duncan, Assistant Professor of Civil Engineering and R. G. Campanella, formerly Assistant Specialist. The report was prepared by J. M. Duncan and R. G. Campanella. This is the fourth report of investigations performed under this contract. The previous reports were "The Effects of Sampling and Disturbance on the Shear Strength of Soft Clays," Report No. TE-64-1, February, 1964; "The Effect of Anisotropy and Reorientation of Principal Stresses on the Shear Strength of Saturated Clay," Report No. TE-65-3, November, 1965; and "Errors in Strength Tests and Recommended Corrections," Report No. TE-65-4, November, 1965.

SUMMARY

Increasing the temperature of a saturated sample of soil while preventing drainage will result in an increase in the pore-water pressure in the sample. Assuming that the electrical force fields between particles are not appreciably affected by changes in temperature, a relationship between changes in temperature and changes in pore-water pressure has been derived which is in good agreement with experimental data.

The analysis shows that the most significant factor in the relationship between temperature and pore pressure in samples during temperature increase is the slope of the swelling curve for the soil. Soils which swell the least for a given percent reduction in effective stress are the ones which will undergo the greatest increase in pore pressure for a given increase in temperature. The increase in pore pressure caused by increase in temperature results in a decrease in undrained strength, just as an increase in pore pressure due to disturbance results in a decrease in strength.

The derived relationship may be used to calculate the change in effective stress associated with the change in temperature from field to laboratory. It may also be used to calculate the acceptable range of fluctuation in laboratory temperature for undrained tests on any soil.

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LIST OF SYMBOLS

ENGLISH LETTERS

$\bar{A}$	pore pressure parameter, the ratio of the change in pore pressure caused by a change in deviator stress to the change in deviator stress - (dimensionless)
$\bar{A}_f$	the value of $\bar{A}$ at failure
a_{vs}	coefficient of swelling = $-\frac{de}{dp}$ on swelling curve - (cm^2/kg)
C_s	change in void ratio on swelling curve for one log (base 10) cycle change in pressure - (dimensionless)
c_{sw}	change in volumetric strain on swelling curve for one log (base e) cycle change in pressure - (dimensionless)
e	void ratio - (dimensionless)
f	as a subscript indicates final
m_{vs}	$d\epsilon_v/dp = a_{vs}/(1 + e)$ - (cm^2/kg)
o	as a subscript indicates original or initial value
p'	major principal effective stress during consolidation, also used without the prime - (kg/cm^2)
T	temperature - (°F or °C)
V_{pp}	volume of a pore pressure measuring device
V_s	volume of solid particles - (cm^3)
V_t	total volume of a soil sample - (cm^3)
V_{wT}	specific volume of water at temperature T - (cm^3/gm)
X	coefficient of cubic thermal expansion - (per °F)

GREEK LETTERS

Δ	a prescript indicating a change in the quantity appended
ϵ_v	volumetric strain = $\Delta V/V$ - (dimensionless or percent)
σ	normal stress - (kg/cm^2) - prime indicates effective normal stress

I. ANALYSIS OF THE EFFECT OF TEMPERATURE CHANGES

Volumetric Strains due to Temperature Changes

Mineral Grains

When the temperature of a mineral grain (solid particle) increases, its volume increases according to the relationship

$$\Delta V_s = -X_s \; V_s \; \Delta T \qquad (1)$$

or

$$\epsilon_{vs} = \left(\frac{\Delta V}{V}\right)_s = -X_s \; \Delta T \qquad (2)$$

where ΔV_s = change in volume of a solid particle - (cm^3)

X_s = coefficient of cubical thermal expansion of a solid particle - (per °F)

V_s = volume of a solid particle - (cm^3)

ΔT = change in temperature - (°F)

and $\epsilon_{vs}=\left(\frac{\Delta V}{V}\right)_s$ = volumetric strain of a solid particle - (dimensionless).

The negative sign is a result of the fact that an increase in volume is considered negative whereas increase in temperature is considered positive. Although the coefficient X_s is different for every mineral, typical values of X_s are (from the Handbook of Chemistry and Physics) 2.0×10^{-5} per °F for quartz, 1.7 to 3.4×10^{-5} per °F for marble, 0.9 to 1.7×10^{-5} per °F for slate, and 1.4×10^{-5} per °F for granite. It seems likely that the order of magnitude would be the same for all minerals, and that the actual value would not be much different from 2 or 3×10^{-5} per °F.

Soil Skeleton and Pore Space

If the length of each member of a truss was changed by the same fraction, any dimension on the truss would change by the same fraction. If each member underwent the same volumetric strain, then the whole truss would undergo that volumetric strain. Similarly, if each mineral grain in a soil sample underwent the volumetric strain indicated by equation 2, the soil sample would undergo the same volumetric strain and the void space of the sample would also undergo this same volumetric strain.

Consider a sample containing a volume of solid particles, V_s. When the temperature of the sample is changed, the change in void space is

$$\Delta V_v = - e\, X_s\, \Delta T \cdot V_s \tag{3}$$

where ΔV_v = change in volume of voids - (cm^3)
and e = void ratio - (dimensionless).

Pore Water

When the temperature of water increases, its volume changes, and except near freezing, an increase in temperature causes an increase in volume. Although the relationship between temperature change and volumetric strain is not linear for water as it is for minerals, the specific volume of water at various temperatures is available in the Handbook of Chemistry and Physics. At some temperature, T_1, water will have a specific volume V_{wT1}; at another temperature, T_2, the specific volume will have changed to V_{wT2}. The volumetric strain associated with the change in temperature from T_1 to T_2 is given by

$$\epsilon_{vw} = \left(\frac{\Delta V}{V}\right)_w = \frac{V_{wT1} - V_{wT2}}{V_{wT1}} \approx V_{wT1} - V_{wT2} \tag{4}$$

where $\epsilon_{vw} = \left(\frac{\Delta V}{V}\right)_w$ = volumetric strain of water - (dimensionless)
and V_{wT} = specific volume of water at temperature T - (cm^3/gm)
The approximation indicated in equation 4 is possible because the specific volumes of interest are within two percent or so of one cm^3/gm.

In a saturated sample of soil containing a volume of solids, V_s, the volume of water in the pores is equal to $e \cdot V_s$ and the change in volume of this much water due to a change in temperature from T_1 to T_2 would be

$$\Delta V_w = e \cdot V_s (V_{wT1} - V_{wT2}) \tag{5}$$

where ΔV_w = change in volume of water - (cm^3).

A Soil Sample

If a sample were allowed to change its volume freely as temperature changed, then volumetric strain might occur for either of the following reasons:

(1) Volume change of the individual soil particles would be reflected in a volume change of the entire sample. This has been discussed previously and is expressed by equations 2 and 3.

(2) Changes in the electrical force fields between particles might cause the volume of the sample to increase or decrease.

The simplest approach is to proceed as if volume change due to changes in the interparticle forces is negligible. On the basis of this assumption, the following formulation is possible: If the sample were free to drain as its temperature changed, an amount of water would be expelled from (or drawn into) the sample which would be the difference between the change in volume of the pore space and the change in volume of the water in the pore space. For a sample containing a unit volume of solids this would be

$$\text{volume of water expelled} = e\left[(V_{wT1} - V_{wT2}) - X_s\,(T_2 - T_1)\right]V_s\ . \qquad (6)$$

If the sample were prevented from draining, the effect would be the same as if the water had been expelled during the change in temperature (at constant pore pressure), and then forced back into the sample (causing an increase in pore pressure). The volumetric strain of the sample causing the increase in pore pressure would be

$$(\epsilon_v)_{\Delta u} = \frac{\text{volume of water expelled}}{\text{volume of sample}} \qquad (7)$$

where $(\epsilon_v)_{\Delta u}$ = volumetric strain of a sample which causes an increase in pore pressure - (dimensionless).

Substituting the expression for the volume of water expelled given by equation 6, the volumetric strain causing an increase in pore pressure may be expressed as

$$(\epsilon_v)_{\Delta u} = \frac{e}{1+e}\left[(V_{wT1} - V_{wT2}) - X_s\,(T_2 - T_1)\right]\ , \qquad (8)$$

which could also be written

$$(\epsilon_v)_{\Delta u} = \frac{e}{1+e} \left[\epsilon_{vw} - \epsilon_{vs} \right]. \qquad (9)$$

Pressure Changes due to Volumetric Strains

The swelling curves for most soils are linear, or approximately so, on a semi-logarithmic plot, i.e., e vs log p'. The equation of such a swelling curve is given by:

$$e = e_o - C_s \log \left(\frac{p'}{p_o'}\right) \qquad (10)$$

where e_o = void ratio at pressure p_o' on a swelling curve - (dimensionless)

C_s = change in void ratio per log (base 10) cycle change in pressure on a swelling curve - (dimensionless)

p' = effective pressure - (kg/cm^2)

and p_o' = initial effective pressure - (kg/cm^2) .

By differentiating equation 10 with respect to effective stress, it can be seen that

$$a_{vs} = - \frac{de}{dp'} = + \frac{C_s}{2.3p'} \qquad (11)$$

where a_{vs} = instantaneous slope of the swelling curve on an arithmetic plot - (cm^2/kg)

The compressibility (or "expansibility") of the soil is given by the expression

$$m_{vs} = \frac{a_{vs}}{1+e} = \frac{C_s}{(1+e)(2.3)(p')} = \frac{c_{sw}}{p'} \qquad (12)$$

where m_{vs} = compressibility (or "expansibility"), the slope of a swelling curve plotted as volumetric strain vs. effective stress - (cm^2/kg)

and c_{sw} = volumetric strain per log (base e) cycle change in pressure - on a swelling curve - (dimensionless).

Thus, the instantaneous compressibility (or "expansibility") of such a soil is inversely proportional to the effective stress. The change in effective stress due to a given volumetric strain may be found by rearranging equation 11 in the form

$$dp' = - \frac{2.3p'}{C_s} \frac{de}{1+e} (1+e) \tag{13}$$

Then, by substituting in terms of c_{sw}, the expression becomes

$$dp' = - \frac{p'}{c_{sw}} d\epsilon_v \tag{14}$$

where dp' = a differential change in effective pressure - (kg/cm^2)
and $d\epsilon_v$ = a differential change in volumetric strain - (dimensionless).

By integrating both sides of equation 14, the ratio of the effective pressure after an increase in volume to the effective pressure before the increase in volume is found to be given by the expression

$$\ell n \frac{p'_f}{p'_o} = \frac{\epsilon_v}{c_{sw}} \tag{15}$$

where p'_f = final effective pressure, after a change in volume - (kg/cm^2)
and p'_o = initial effective pressure, before a change in volume - (kg/cm^2)

Poulos (1964) measured c_{sw} in triaxial swelling tests and found that it was practically independent of the effective principal stress ratio. This means that c_{sw} can probably be determined with sufficient accuracy from the swelling curve of a consolidation test, and applied to triaxial tests where the sample is under isotropic pressure, i.e.,

$$\ell n \frac{\sigma'_f}{\sigma'_o} = \frac{\epsilon_v}{c_{sw}} \tag{16}$$

where σ'_f = isotropic effective stress acting on a triaxial sample after a change in volume - (kg/cm^2)
and σ'_o = isotropic effective stress acting on a triaxial sample before a change in volume - (kg/cm^2).

Changes in Pore Pressure in Undrained Tests due to Changes in Temperature

Equation 9 expresses the volumetric strain causing an increase in pore pressure in terms of the known volumetric strains of the mineral grains and the water. Equation 16 expresses the ratio of initial to final effective stress associated with a given volumetric strain when the volume increases. Substituting $(\epsilon_v)_{\Delta u}$ given by equation 9 into equation 16, the ratio of the isotropic effective stress acting on an undrained sample after and before a change of temperature from T_1 to T_2 is given by

$$\ell n \; \frac{\sigma'_{T2}}{\sigma'_{T1}} = \frac{e}{1+e} \; \frac{\epsilon_{vw} - \epsilon_{vs}}{c_{sw}} \tag{17}$$

where σ'_{T2} = isotropic effective stress acting on a triaxial sample at temperature T_2 - (kg/cm^2)

σ'_{T1} = isotropic effective stress acting on a triaxial sample at temperature T_1 - (kg/cm^2)

ϵ_{vw} = volumetric strain of water caused by a change in temperature from T_1 to T_2 - (dimensionless)

and ϵ_{vs} = volumetric strain of solid particles caused by a change in temperature from T_1 to T_2 - (dimensionless)

Equation 17 may also be written as

$$\frac{\sigma'_{T2}}{\sigma'_{T1}} = \exp\left(\frac{e}{1+e} \cdot \frac{\epsilon_{vw} - \epsilon_{vs}}{c_{sw}}\right) \; . \tag{18}$$

Equations 17 and 18 show that the ratio of the effective stress acting on an undrained sample before and after a given change in temperature is independent of the absolute magnitude of the effective stress. The most important parameter affecting this ratio is the slope of the swelling curve for the soil. A soil which swells relatively little when subjected to a given decrease in effective stress, that is, a soil which has a relatively small value of c_{sw}, will undergo a relatively large increase in pore pressure when subjected to a given increase in temperature.

An increase in the temperature of the pore pressure measuring device which is connected to the sample, as well as an increase in temperature of the sample itself, may affect the pore pressure in an undrained sample. Since the coefficients of thermal expansion of brass, copper and stainless steel, out of which pore pressure measuring systems are usually constructed, are less than that for water at room temperature, the increase in volume of water in a pore pressure measuring system will be larger than the increase in volume of the measuring system itself. Thus an amount of water equal to the difference of the increases in volume will be expelled into the sample. The resulting increase in the volume of water in the sample will cause a volumetric strain of the sample which will result in a decrease in effective stress.

By means of an analysis similar to the one described previously, it may be shown that the ratio of the effective stress in a sample after and before a change in temperature of the pore pressure measuring device to which the sample is connected may be expressed by

$$\frac{\sigma'_{T1}}{\sigma'_{T2}} = \exp \frac{V_{pp}}{V_t} \cdot \frac{\epsilon_{vw} - \epsilon_{vpp}}{c_{sw}} , \qquad (19)$$

where V_{pp} = volume of the pore pressure measuring device - (cm^3)

V_t = total volume of the sample - (cm^3)

ϵ_{vw} = volumetric strain of the water in the pore pressure measuring device - (dimensionless)

and ϵ_{vpp} = volumetric strain of the pore pressure measuring device - (dimensionless).

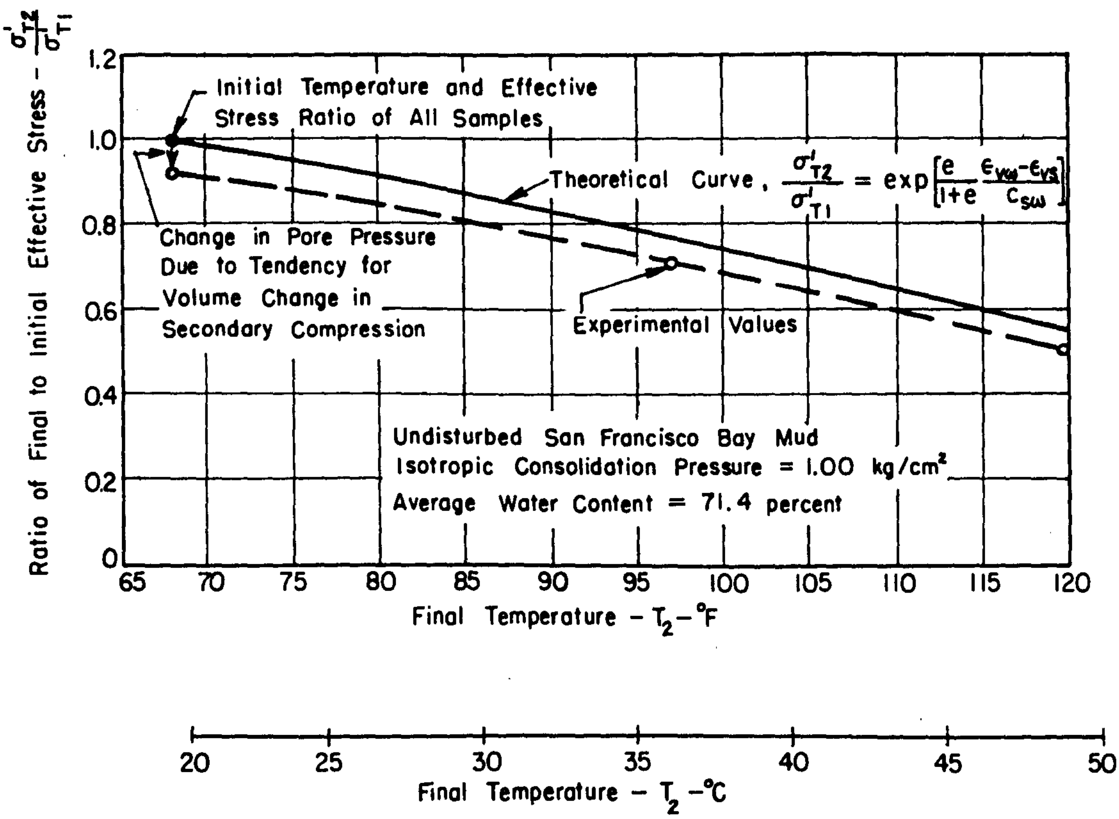
It may be noted that the change in pore pressure which is caused by an increase in temperature of the pore pressure measuring device will increase with the ratio of the volume of the device to the volume of the sample. Thus a system which contains a large volume of water will cause a larger volumetric strain of a sample for a given increase in temperature than will a system which contains a small amount of water. The volume of water in a pore pressure measuring system includes the volume of water in all tubes, valves and fittings in the system.

II. COMPARISON OF EXPERIMENTAL AND THEORETICAL RESULTS

Using equipment specially constructed for controlling and changing the temperature of triaxial samples during testing (Mitchell and Campanella, 1963), a series of three consolidated undrained tests was performed on undisturbed San Francisco Bay Mud (L.L = 88, P L. = 43). The data obtained from these tests have been compared with the theoretical results obtained by means of equation 18. The data also indicate the effect of temperature change on the undrained strength of soft, saturated clay.

Three samples of undisturbed Bay Mud were trimmed and consolidated to 1.00 kg/cm^2 at 68°F. Then the drainage valves were closed and the temperatures of two of the samples were increased: one to 95.8°F, and the other to 119.8°F. The temperature of the third sample was kept at 68°F. When all samples had reached temperature equilibrium, the changes in pore pressure were measured, and undrained stress-controlled tests were performed on the samples. The pore pressure measuring transducers used in the tests had been calibrated for changes in temperature, and the effect of change in volume of the water in the pore pressure measuring system was negligible, because the pore pressure trandsducers were built into the bases of the triaxial cells, so that the volume of the pore pressure measuring devices was very small compared to the volume of the samples.

The experimental data and the corresponding theoretical curve are shown in figure 1. The theoretical curve was calculated using equation 18, with $X_s = 3 \times 10^{-5}$ per °F, values for the volumetric strain of water calculated using published tables, and $c_{sw} = 0.01$ determined from a consolidation test on undisturbed Bay Mud. The vertical displacement between the experimental and the theoretical curves is believed to be due to the change in pore pressure which occurs due to a tendency for volume change in secondary compression, as indicated by the fact that the pore pressure in the sample which was maintained at 68°F increased during the time the temperatures of the other samples were being changed. Probably a longer consolidation period would have minimized this displacement by reducing the tendency for secondary compression.



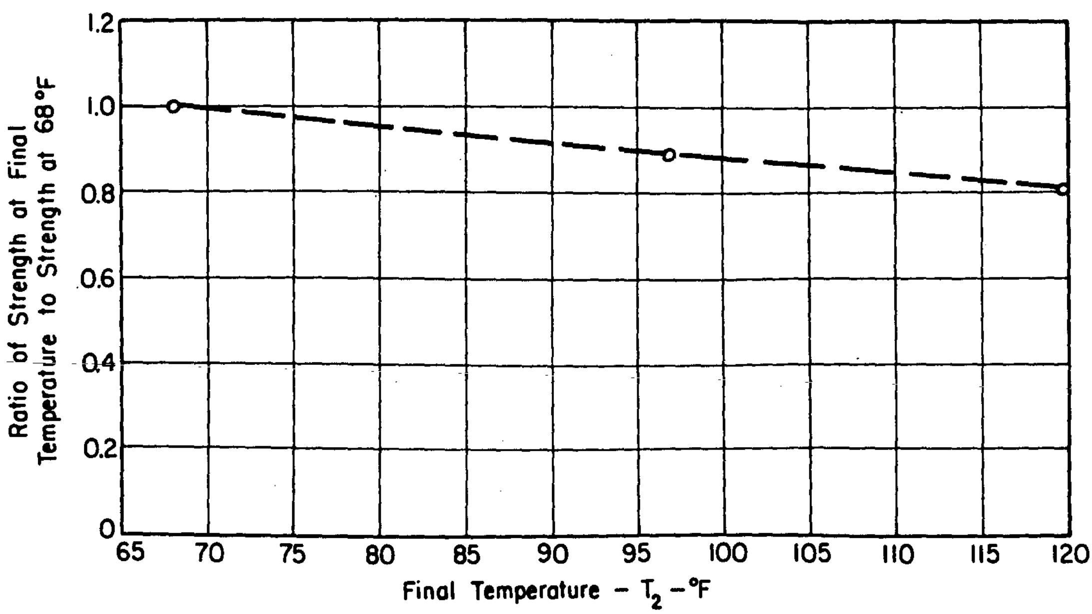


Fig. 1 - VARIATION OF EFFECTIVE STRESS AND STRENGTH RATIOS FOR SAMPLES OF UNDISTURED BAY MUD ISOTROPICALLY CONSOLIDATED AT 68 °F AND SUBJECTED TO A TEMPERATURE CHANGE WITHOUT ALLOWING DRAINAGE.

It is worthy of note that if the experimental and theoretical results were compared for a single change in temperature, it would appear that the agreement between experiment and theory was not good. The discrepancy would be especially pronounced for a small change in temperature, in which case the increase in pore pressure caused by the tendency for volume decrease in secondary compression would be large compared to the increase in pore pressure caused by increase in temperature. For example, if only the results of a test where the temperature of a sample was increased by 10°F were available, the experimentally determined decrease in effective stress (interpolated from the experimental curve in figure 1) would be about 16 percent of the initial value, whereas the theoretical decrease in effective stress is equal to about eight percent of the initial value. Thus, although the agreement between the single experimental and theoretical results would not be good, the comparison in figure 1 shows that the discrepancy in the case of small changes in temperature probably results from the tendency for volume change in secondary compression, and that generally the agreement between theory and experiment is good when this fact is taken into account. Increase in pore pressure due to a tendency for volume change in secondary compression would only be expected to be of such great importance for samples which have been consolidated in the laboratory for a relatively short period of time, of the order of one day. Undisturbed samples from the field, on the other hand, would probably not exhibit this tendency to an appreciable degree.

Additional data illustrating the effect of an increase in temperature on the pore pressure in an undrained soil sample have been published by Seed, Mitchell and Chan (1960). A saturated sample of silty clay (L.L.= 38, P.L. = 24) was consolidated to 3.00 kg/cm^2 at 68°F. While preventing drainage of the sample, the temperature of the sample and the pore-pressure measuring device was increased to 77°F; this increase in temperature resulted in an increase in the pore water pressure of 0.80 kg/cm^2 and reduction of the effective stress acting on the sample to 2.20 kg/cm^2. The value of c_{sw} for this clay, determined from consolidation tests, is 0.0038. The theoretical effective stress after the increase in temperature, calculated neglecting the influence of the

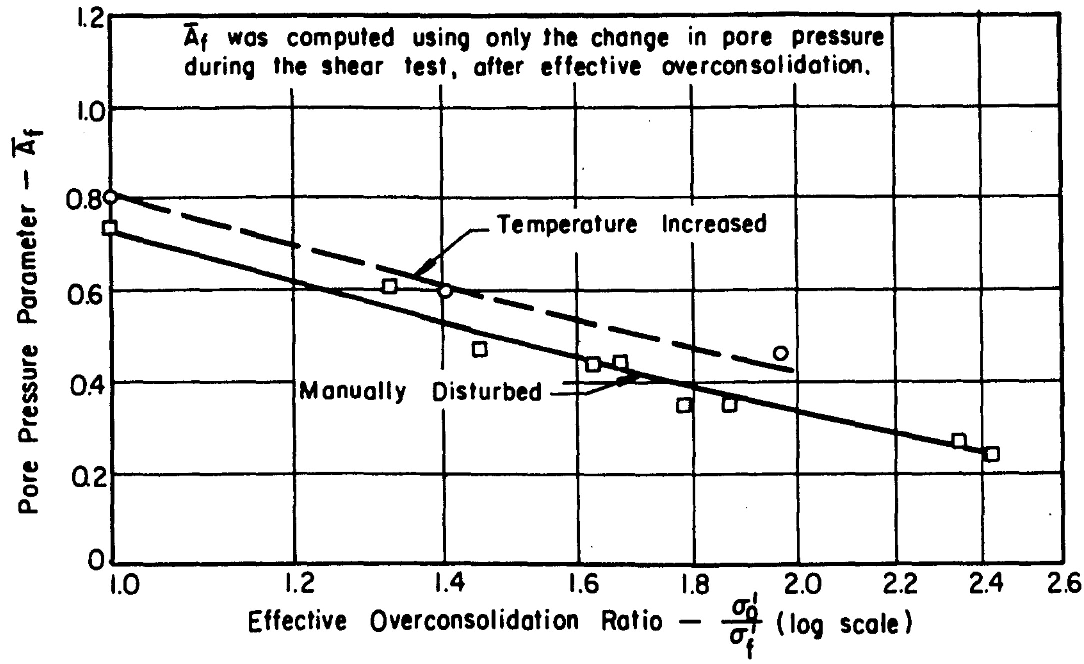
the pore pressure measuring system, is equal to 2.65 kg/cm^2. Including the effect of the change in temperature of the pore pressure measuring system, calculated using equation 19, the theoretical effective stress after the increase in temperature is 2.59 kg/cm^2. Although the agreement between the measured and theoretical value in this case is not good, the experimental values are probably affected by a tendency for volume change due to secondary compression as were the data obtained in the undrained tests on undisturbed Bay Mud.

III. DECREASE IN STRENGTH IN UNDRAINED TESTS DUE TO INCREASE IN TEMPERATURE

The increase in pore pressure caused by an increase in temperature results in a decrease in strength. The relative strengths of the samples of Bay Mud, which were measured after the samples came to undrained equilibrium at their final temperatures, are shown in the lower half of figure 1.

The rate of decrease of effective stress with temperature increase is about 0.7 percent per °F, but the rate of decrease of strength is only about 0.4 percent per °F. The reason that these two rates are different is that the samples are effectively overconsolidated at the higher temperatures. The pore pressure parameter $\bar{A}_f$ decreases with increasing effective overconsolidation ratio as shown in the upper part of figure 2. Also shown is the decrease in $\bar{A}_f$ when effective overconsolidation is caused by disturbance rather than temperature change, (Seed, Noorany and Smith, 1964). Whether the effective stresses are reduced by disturbance or temperature increase, the effect of the decrease in effective stress on the strength of the soil is somewhat offset by the fact that $\bar{A}_f$ also decreases.

The relative strengths of the samples are shown as a function of the effective overconsolidation ratio in the bottom of figure 2, where it may be seen that manual disturbance and increase in temperature have about the same effect on the strength. Thus it may be concluded that even the most carefully handled samples may suffer a reduction in strength due to change in temperature if the temperature of the sample increases when the sample is brought from the field to the laboratory.



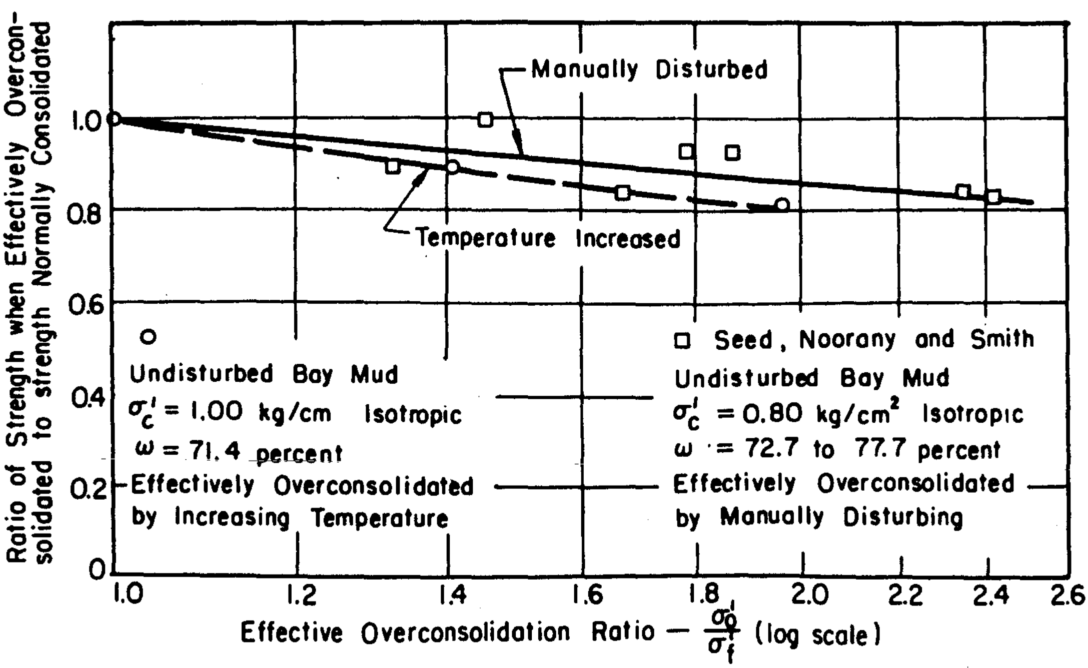


Fig. 2 — COMPARISON OF THE EFFECTS OF DISTURBANCE AND TEMPERATURE CHANGE ON THE PORE PRESSURE PARAMETER $\bar{A}_f$ AND THE STRENGTH OF UNDISTURBED BAY MUD.

IV. CONCLUSIONS

An increase in temperature causes a reduction in the effective stress acting on an undrained sample of soil; the reduction in effective stress induced by a given change in temperature is larger for clays of low plasticity which swell relatively little for a given reduction in effective stress than for clays of higher plasticity which swell more. As the effective stress decreases due to temperature increase, the undrained strength decreases; the reduction in strength accompanying a given decrease in effective stress is about the same when the decrease in effective stress is caused by temperature increase as when the decrease in effective stress is caused by disturbance. There are two cases where these effects may be of practical importance:

(1) When a sample is brought from the field to the laboratory: If the laboratory is warmer than the field ground temperature, the strength measured in UU tests will be too low simply as a result of the change in temperature.

(2) When a consolidated undrained test is made in the laboratory: If the laboratory temperature changes during the undrained part of the test, the results of the test may be influenced.

Equation 18 may be used in the first case to make an estimate of the ratio of the effective stress in the laboratory to the effective stress in the field if both temperatures are known. The decrease in strength due to increase in temperature could also be estimated if the variation of strength with effective overconsolidation ratio were known. The in-situ temperature of San Francisco Bay Mud was determined by measuring the temperature of auger cuttings as soon as they were brought up from bore holes; the measurements, which were made in February, showed that the temperature increased from 55°F at 10 feet below the surface to 60°F at 20 feet. Since the samples are stored and all laboratory tests are made at 68°F, the effective stress in the laboratory would be reduced to about 95 percent of that in the field due to temperature increase; disturbance would reduce the laboratory effective stress even more.

In the second case the question to be answered is "How closely must the temperature of a laboratory be controlled to prevent fluctuations in temperature from affecting the results of undrained tests?" The temperature of a "constant temperature" room in many laboratories may vary by one degree Farenheit during the course of a day. This is adequate control for undrained tests on soils such as Bay Mud; even if the change in temperature of the sample was as large as the change in ambient temperature in the laboratory, the effective stress in the sample would only change by about 0.7 percent, and the strength would only change by about 0.4 percent. However, a change in temperature of one degree Farenheit might have an appreciable effect on the results of undrained tests on clays of low plasticity.

An extreme example of the effect of temperature on the results of undrained tests would be the case of an undrained test performed on a clay of low plasticity in a laboratory without temperature control; in this case the air temperature in the laboratory might increase by 5°F during the course of the test, and the sample temperature might increase by 2.5°F. Poulos (1964) measured c_{sw} for Canyon Dam Clay (L.L. = 34, P.L. = 15) using triaxial tests; he found that value of c_{sw} varied from 0.0009 to 0.0017 depending on the amount of swell and the consolidation pressure. If the clay being tested was similar to Canyon Dam clay, having $c_{sw} = 0.001$, then the effective stress acting on an undrained sample after the temperature of the sample had increased from 68°F to 70.5°F would be only about 85 percent of the effective stress acting on the sample before the increase in temperature. This amount of change in effective stress could have a drastic effect on the results of an undrained test. In order to reduce the change in effective stress due to temperature increase to one percent, the temperature of the laboratory would have to be controlled so that the temperature of the sample would not change more than about 0.2°F when soils such as Canyon Dam clay are tested.

LITERATURE CITED

Mitchell, James K. and Campanella, R. G. (1963) "Creep Studies on Saturated Clays", ASTM Special Technical Publication No. 361, pp. 90-110.

Poulos, S. J. (1964) "Control of Leakage in the Triaxial Test," Harvard Soil Mechanics Series No. 71.

Seed, H. B., Mitchell, J. K. and Chan, C. K. (1960) "The Strength of Compacted Cohesive Soils," Proceedings of the ASCE Research Conference on the Shear Strength of Cohesive Soils, pp. 877-963.

Seed, H. B., Noorany, I. and Smith, I. M. (1964) "Effects of Sampling and Disturbance on the Strength of Soft Clays," University of California Institute of Engineering Research Report No. TE-64-1.