# Temperature Effects on the Engineering Properties and Behavior of Soils

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•THE 14 papers covered in this general report treat a number of aspects of temperature and heat effects on the engineering properties of soils. Collectively they provide concepts and data that make possible a greatly improved understanding of the importance of thermal influences in soil mechanics and thermal treatment of soils for property improvement. In this report the most significant aspects of each of the contributed papers are summarized and appraised, relevant supplementary data and analyses are introduced as appropriate for illustration of points under discussion, and questions are raised concerning areas needing further study.

The topics discussed in the various papers may be listed as follows:

Topic	Author (s)
1. Temperature Effects on Engineering Properties	
<ul> <li>a. Volume change phenomena, pore pressure effects, compressibility</li> <li>b. Strength</li> <li>c. Elasticity</li> <li>d. Creep and stress relaxation</li> <li>e. Swelling—salt heave</li> </ul>	Plum and Esrig Noble and Demirel, Sherif and Burrous, Laguros Murayama Noble and Demirel, Murayama Blaser and Scherer
2. Thermal Stabilization	
a. Effects of preliminary heat treatment	Barata, Chandrasekharan et al, Wöhlbier and Henning
b. Fusion of soils	Post and Paduana
c. Ice barriers	Takashi
3. Temperature Effects in Pavement Subgrades	
a. Field observations b. Analysis methods	Richards, Fang Paaswell

# TEMPERATURE EFFECTS ON SOIL PROPERTIES

# Volume Change Phenomena, Pore Pressure Effects

Volume and pore water pressure variations that develop in saturated soils due to temperature changes can be expressed in terms of the thermal expansion of the soil components, compressibility of the soil, and physicochemical effects, as shown recently by Campanella and Mitchell (6). They showed that under fully drained conditions and constant confining pressure a volume of water,  $\Delta V_{DR}$ , will drain from a saturated soil as a result of a temperature increase,  $\Delta T$ , as given by

$$(\Delta V_{DR})_{\Delta T} = \alpha_W V_W \Delta T + \alpha_S V_S \Delta T - \alpha_S V_m \Delta T - (\Delta V_{St})_{\Delta T}$$
(1)

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where

 $\alpha_{\rm S}$  = thermal coefficient of cubical expansion of mineral solids;

 $\alpha_{\dot{W}}$  = thermal coefficient of expansion of soil water;

 $V_w$  = volume of pore water;

- $V_s$  = volume of mineral solids;
- $V_{m}$  = total volume of soil specimen; and
- $\Delta V_{st}$  = change in volume of soil structure due to temperature-induced changes in interparticle forces.

Under undrained conditions and constant confining pressure, a change in temperature leads to a change in pore water pressure,  $\Delta u$ , given by

$$\Delta u = \frac{n\Delta T(\alpha_{s} - \alpha_{w}) + \alpha_{st}\Delta T}{m_{w} + nm_{w}}$$
(2)

where

n = porosity;

 $\alpha_{st}$  = physicochemical temperature coefficient of soil structure volume change;

 $m_v$  = compressibility of soil structure; and

 $m_w$  = compressibility of water.

The results of drained triaxial tests showed that significant permanent volume decreases may occur during initial temperature increases ( $\alpha_{st}$  is negative). This is illustrated by test results for remolded illite shown in Figure 1. It may be seen that a significant irrecoverable volume of water was drained from the sample during the first heating cycle. The data suggest also that a temperature increase under drained conditions at constant effective stress followed by cooling may have exactly the same effect as overconsolidation using a pressure increase followed by unloading.

Additional data presented by Campanella and Mitchell (6) showed that pore pressure changes accompanying temperature changes under undrained conditions can be predicted reasonably well using the concepts leading to Eq. 2. The most important factors controlling pore pressure changes appear to be the thermal expansion of the pore water, the compressibility of the soil structure, and the initial effective stress. The results of tests on several clays showed that for each 1-deg F change in temperature the pore water pressure changes by about 0.75 to 1.0 percent of the initial effective stress. For less compressible materials the pore water pressure change is considerably greater. Changes of this magnitude indicate clearly the need for temperature control in the laboratory if unambiguous results are to be obtained, and provide also some indication of possible differences in properties between soils in situ and in the laboratory where temperatures may be different.

In their paper, Plum and Esrig (25) present test results and interpretations to (a) provide further insight into the effects of heating a saturated soil on compressibility, (b) determine how stress history affects observed volume changes associated with temperature change, and (c) gain insight into the phenomenon of pore pressure development due to temperature change.

In studying the effect of temperature on consolidation characteristics, Plum and Esrig found that the compression index of two clays varied both with temperature and pressure. Figure 2 shows the void ratio vs logarithm of pressure relationship for illite at two temperatures. Each specimen was initially consolidated under a pressure of 1.7 psi at a temperature of 24 C. It may be seen that below a consolidation pressure of about 30 psi the sample tested at 50 C was more compressible than the sample tested at 24 C. At pressures greater than 30 psi little difference in compressibility is apparent.

The results of triaxial consolidation tests on illite at different temperatures have been presented by Campanella and Mitchell (6), as shown in Figure 3. Unfortunately data points were not obtained in these tests for stresses less than 2 kg per sq cm. Figure 3 shows equal compression indices for three temperatures at pressures greater than 2 kg per sq cm. Since consolidation was started from the same initial water content for all



Figure 1. Relationship between volume of water drained from sample and time during temperature changes at constant stress.



Figure 2. Influence of temperature on consolidation of illite.

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Figure 3. Effect of temperature on isotropic consolidation behavior of saturated illite.

three specimens, the specimens at the higher temperatures must have been more compressible at lower pressures to account for the void ratio differences at a pressure of 2.0 kg per sq cm. Thus the results in Figure 3 are in accordance with those of Plum and Esrig, rather than contrasting as they suggest in their paper.

The effect of heating followed by cooling and its similarity to the effects of overconsolidation may be seen in Figure 4. Plum and Esrig point out that laboratory testing of natural soils that have been heated and recooled during handling could lead to an incorrect evaluation of the maximum past pressure in the field.

The amount of temperature-induced consolidation is related to soil compressibility the higher the compressibility the greater the consolidation for a given temperature increase. Plum and Esrig show that a given increment in temperature at constant effective stress has an effect equivalent to some increment of pressure at constant temperature. This may be seen from Figure 5, which shows the volumetric strain of illite as a function of overconsolidation ratio for a temperature increase from 24 to 50 C. For specimens with an overconsolidation ratio greater than about 1.7, this temperature increase caused no consolidation; i.e., the preconsolidation had reduced the compressibility of the soil structure sufficiently so that even after the weakening caused by the temperature increase it could still carry the applied effective stress without appreciable further consolidation.

Plum and Esrig present the results of one test conducted at quite high consolidation pressure (loading from 180 to 209 psi) and conclude that the secondary compression is only slightly affected by temperature increases. Other investigators (6, 14, 28), however, have concluded that secondary compression rates are increased significantly by temperature increases. Further consideration of this question therefore appears in order.

# Temperature Sensitivity of Double Layer Repulsive Forces

Both Plum and Esrig (25) and Laguros (12) in their papers have attempted explanations of observed behavior using arguments based on changes in interparticle repulsive forces caused by temperature changes. Plum and Esrig postulate that the volume decrease associated with heating a soil results because the double layers expand, thereby increasing repulsive forces and decreasing the effective stress and strength at interparticle contacts. Contradictory statements can be found in the literature concerning the effects of heating on double layer thickness and interparticle repulsive forces.

According to the Gouy-Chapman theory the approximate double layer thickness is given by

$$\frac{1}{\varkappa} = \text{approximate double layer thickness} = \sqrt{\frac{\epsilon kT}{8\pi n e^2 v^2}}$$
(3)

#### where

- $\epsilon$  = dielectric constant;
- k = Boltzmann's constant;
- T = absolute temperature;
- n = electrolyte concentration;
- e = electronic charge; and
- $\mathbf{v}$  = cation valence.





Figure 4. Effect of heating and cooling on void ratio vs pressure relationship of illite.

Figure 5. Effect of overconsolidation ratio on volume change of illite on heating from 24 to 50 C.

It may be seen from Eq. 3 that, for all other parameters constant, an increase in temperature should cause an expansion of the double layer. In reality, however, it is apparently impossible to vary temperature without causing a variation in the dielectric constant. If values of dielectric constant for the soil water are assumed equal to those for pure free water, then the decrease in dielectric constant with increasing temperature. Thus one might conclude that an increase in temperature should cause a depression of the double layer.

These considerations, however, neglect the fact that Eq. 3 is only an approximation of the double layer thickness. Thus direct extensions using this equation to predictions of interparticle repulsive forces may be misleading. A more refined analysis is possible using the swelling pressure equations developed by Bolt (4, 5). The concentration of cations at the mid-plane between interacting parallel particles is given by

$$v \sqrt{\beta C_0} (x_0 + d) = 2 \sqrt{C_0/C_c} \int_{\phi=0}^{\pi/2} \frac{d\phi}{\sqrt{1 - (C_0/C_c)^2 \sin^2 \phi}}$$
 (4)

where

- v = valence of cation;
- $\beta = \frac{8\pi F^2}{1000 c PT}$
- $3 = \overline{1000 \text{ } \epsilon \text{RT}}$
- F = Faraday constant;
- R = gas constant;
- d = half-distance between plates;
- $x_0$  = constant related to surface density of charge on the clay (it is generally less than 4 Å);
- $C_0$  = concentration of bulk liquid pressed out from the clay;
- $C_c$  = cation concentration at the mid-plane between particles;
  - $\phi$  = variable related to central concentration, the value of which is not needed for evaluation of the complete elliptic integral.

The swelling pressure (repulsive pressure between plates) is given by the difference between the osmotic pressures in the central plane and in the bulk solution according to

$$P_{s} = RT C_{0} (C_{c}/C_{0} + C_{0}/C_{c} - 2)$$
(5)





Figure 6. Effect of temperature on double layer repulsions assuming no variation in dielectric constant.

Figure 7. Effect of temperature on double layer repulsions assuming dielectric constant dependent on temperature.

The relationship between repulsive pressure and plate spacing has been evaluated for different temperatures, assuming all other quantities constant, and is shown in the form of  $(P_S/C_0)$  vs  $(d'\sqrt{C_0/\epsilon})$  in Figure 6. For this analysis the quantity  $(x_0 + d)$  in Eq. 4 has been replaced by d', since d'  $\simeq$  d at all but very small particle spacings. Figure 6 shows that an increase in temperature from 0 to 50 C causes a definite increase in repulsions for all spacings. This increase is of the order of 30 to 40 percent for the 50 C increase in temperature.

If, however, the dielectric constant is assumed to vary with temperature, then the result shown in Figure 7 is obtained, using  $\epsilon$  values for pure water. It may be seen that under these conditions the repulsive pressures are unchanged over the temperature range from 0 to 100 C.

Unfortunately the dielectric constant of soil water and its variation with temperature are not known with any certainty. The presence of cations in solution and the electric field in the vicinity of particle surfaces may be expected to have an influence on the dielectric constant. Harris and O'Konski (11) present low-frequency dielectric constant data showing that as ionic concentration increases the value of  $\epsilon$  decreases. For example, values at 20 C for NaCl solution are

Concentration	<u> </u>
0	80
2M	59.5
4M	46.0
5M	42.0

Within the double layer the concentration decreases from a very high value at the particle surface to that of the bulk solution (usually less than 1M) at large distances. Thus a varying  $\epsilon$  with distance might be expected.

Van Olphen (31) suggests that the value of  $\epsilon$  in the stern layer adjacent to particle surfaces may be in the range of 3-6. Low (15) cites data suggesting values of the same order in thin films on clay surfaces. Thus it would appear that the appropriate value of  $\epsilon$  for use in double layer computations should be less than that for pure water.

This uncertainty in values of  $\epsilon$  makes computations of the type leading to Figures 6 and 7 questionable on a quantitative basis. Regardless of the actual values, however,

an increase in temperature can be expected to decrease the dielectric constant because of the greater difficulty of ordering water molecules at higher temperatures. Thus the qualitative picture presented by Figure 7 is considered more realistic than that in Figure 6, and it is concluded that temperature variations should have little effect on interparticle double layer repulsions.

In view of these considerations it does not appear that increased double layer repulsive forces can account for the major part of the compression (through reduction of interparticle effective stresses) that occurs due to heating. It is more likely that the interparticle contact structure itself is weakened because of the increased thermal energy of the constitutent atoms.

#### Strength

Sherif and Burrous (29) present a useful summary of previous work on the influence of temperature changes on the shearing

strength of soils. While most published data indicate that an increase in temperature causes a strength reduction, not all results are in complete agreement. The data do suggest, however, that for all other factors constant (e.g., structure and water content), lower strengths are associated with higher temperatures. Test results presented by Sherif and Burrous are in accord with this.

Specimens of kaolinite were prepared by triaxial consolidation to effective stresses in the range of 20 to 98 psi. Consolidation was done at 75 C. The consolidated specimens were then tested in unconfined compression (stress-controlled) at temperatures of 75, 100, 125, and 150 F. Figure 8 shows the results of these tests in the form of the logarithm of compressive strength vs moisture content. It may be seen that the relationship is linear and unique for each temperature.

From the form of the results Sherif and Burrous concluded that a given increase in temperature has an effect on compressive strength analogous to a given increase in water content, independent of the value of the initial water content. From Figure 8 it may be seen that the strength,  $\sigma_0$ , at 75 F (consolidation temperature) is given by

$$\sigma_{\rm o} = \exp \left( {\rm A - Bw} \right) \tag{6}$$

where w is the water content and A, B are material constants.

The strength at any other temperature is given by

$$\sigma_{T} = \exp (A - Bw - C\Delta T) = \sigma_{O} \exp (-C\Delta T)$$
(7)

where

 $T = T - T_0 (T_0 = 75 F)$  and C = parameter characteristic of the strength change caused by temperature change.

Sherif and Burrous conclude that the quantity  $C\Delta T/B$  might be regarded as an equivalent moisture content increase, w'. They also tested the following equation and found that it fit the data well:

$$\sigma_{\rm T} = \frac{{\rm T}_{\rm o}}{{\rm T}} \exp \left[ {\rm A} - {\rm B} \left( \frac{\gamma_{\rm WO}}{\gamma_{\rm W}} \right) {\rm w} \right]$$
(8)



Figure 8. Effect of temperature on the undrained strength of kaolinite.



Figure 9. Effect of consolidation and test temperature on strength of alluvial clay.

where  $\gamma_{WO}$  and  $\gamma_W$  are the densities of water at  $T_O$  and T, respectively. Unfortunately they do not indicate the physical basis for this equation.

Laguros (12) indicates a progressive increase in strength as temperature is increased from 35 to 105 F for three soils. For a fourth soil, maximum strength was observed at 70 F. Although this would appear at first glance to contradict the results of Sherif and Burrous, it should be noted that in the Laguros tests specimens were compacted at the test temperature. Consequently, those compacted at the higher temperatures had significantly higher densities (and possibly different structures) than those prepared at low temperatures. As a result the strength values reflect the added variable of density. Laguros' results for liquid limit show a consistent decrease with increase in temperature, which would be expected, since the liquid limit is also a



Figure 10. Mechanical model of clay skeleton proposed by Murayama.

measure of strength, and at the high water contents associated with the liquid limit the initial structure and density are of little consequence.

Noble and Demirel (23) present the results of a large number of direct shear tests on a highly plastic clay that show the effects of both temperature of consolidation and temperature at test. Figure 9 summarizes the results of these tests. All specimens were tested at temperatures equal or less than the consolidation temperature. The results show clearly that the higher the consolidation temperature the greater the shear strength at any given test temperature, thus supporting the interpretation given for Laguros' results. For a given consolidation temperature, however, the strength decreases in a regular manner with increasing test temperature.

## Elasticity

The elasticity properties of most engineering materials are temperature-dependent. There is no reason to believe that the elasticity characteristics of soils should not be temperature-dependent also, although prior to the contribution by Murayama (19) specific data have not been readily available. To investigate the temperature-dependence of the elasticity of clays, Murayama has used the rheological model shown in Figure 10, which was proposed for the description of the viscoelastic behavior of soil in his earlier work (20, 21, 22).

In this model the dashpot coefficient,  $\eta$ , is derived using the theory of rate processes and is given by

$$\eta = \frac{\sigma_2}{2A\sigma_{20}\sinh(B\sigma_2/\sigma_{20})}$$
(9)

where

 $\sigma_2$  = stress on dashpot;

 $\sigma_{20}$  = initial stress on dashpot; and

A,  $\tilde{B}$  = constants dependent on soil structure and temperature.

The two springs in the model are assumed linear with constants  $E_1$  and  $E_2$ .

The purpose of the present study was to determine the effect of temperature on these two constants. Murayama proposes that the elasticity of a clay skeleton may be due partly to flexure of clay particles, but is caused mainly by physicochemical interparticle forces, and these forces should be temperature-sensitive. The stress relaxation test was used as a means for evaluating  $E_1$  and  $E_2$  at different temperatures. All tests were run on undisturbed specimens of the highly plastic Osaka alluvial clay tested in unconfined compression. The maximum past pressure on the undisturbed specimens was 3.8 kg per sq cm.

Solution of the rheological model shown in Figure 10 for the case of stress relaxation leads to

$$\sigma = \frac{E_1 E_2}{E_1 + E_2} \left\{ \left( \epsilon_0 + \frac{\sigma_2}{E_1} \right) - \left( \epsilon_0 - \frac{\sigma_2}{E_1} \right) \frac{E_1}{BE_2} \log[AB(E_1 + E_2)t] \right\}$$
(10)

which holds for

$$\epsilon_{0}E_{1} > \sigma \geq \frac{E_{1}E_{2}}{E_{1} + E_{2}} \left[ \epsilon_{0} \left( 1 + \frac{2E_{1}}{BE_{2}} \right) + \frac{\sigma_{2}}{E_{2}} \left( 1 - \frac{2}{B} \right) \right]$$

For very long time,

$$\sigma_{t \to \infty} = \frac{E_1 E_2}{E_1 + E_2} \left( \epsilon_0 + \frac{\sigma_2}{E_2} \right)$$
(11)

These relationships are valid only for the range of stress where the stress vs strain relationship is linear. The rate of stress relaxation is given by

$$-\frac{d\sigma}{d\log t} = \frac{\sigma_{t\to\infty} - \sigma_L}{(E_2/E_1)B}$$
(12)

Although the various equations presented by Murayama can be used to compute the model parameters from the results of stress relaxation tests at different temperatures, they cannot be used directly for the prediction of the effects of temperature on stress relaxation. Whereas the parameters A and B in Eqs. 10 and 12 are known functions of temperature, the initial stress,  $\sigma_0$ , for a given initial strain,  $\epsilon_0$ , and the final stress,  $\sigma_{t\to\infty}$ , are also functions of temperature, and these latter functions are unknown. The model is assumed by Murayama to be valid for stress relaxation, since the test results follow a form predicted by the derived equations.

On this basis the effect of temperature on the elastic moduli  $E_1$  and  $E_2$  may be examined. The test results showed that for initial strains ( $\epsilon_0$ ) of up to about 1.0 percent,



Figure 11. Influence of temperature on elastic constants in Murayama's model.



Figure 12. Elastic constants according to the Murayama model as a function of temperature.

the values of  $E_1$  and  $E_2$  were constant with variations in initial strain, but they decreased with further increase in initial strain, as shown in Figure 11. The results show also that as temperature increases the elastic moduli,  $E_1$  and  $E_2$ , decrease. This temperature-sensitivity is shown more clearly in Figure 12, where values of  $E_1$  and  $E_2$  for  $\epsilon_0 <$ 1.0 percent are plotted vs temperature. Whether or not these values represent true elastic moduli, of course, depends on the validity of the rheologic model used for their evaluation.

Figure 13 shows the relationship between initial stress and initial strain in the stress relaxation tests at different temperatures. If the slopes of the straight-line portions of these curves are taken to represent the elastic modulus of the material, then it may be seen that an increase in temperature causes a significant decrease in modulus. In this case the conclusion does not depend on the validity of a rheologic model for its validity.

# **Creep and Stress Relaxation**

A number of investigators have treated creep and stress relaxation phenomena in soils using the theory of rate processes (10) in recent years, including Murayama and Shibata (20), Mitchell (16), Christensen and Wu (8), and Andersland and Akili (1). Mitchell, Campenella and Singh (17) have provided data that substantiate the assumption that soil deformation can be considered as a thermally activated process. Noble and



Figure 13. Effect of temperature on the stiffness of Osaka clay.

Demirel (23) have used a similar approach in the study of temperature effects on the deformation behavior of two soils—a highly plastic clay and a low plastic silt.

Noble and Demirel envisage that interparticle bonds form in a cohesive soil due to an oriented structure in the adsorbed water layer's adjacent clay particles. Deformation of the system then occurs by distortion and breaking of these bonds. The unstable condition that exists following the breaking of a bond is considered the "activated state" in which the "contact zone" between particles consists essentially of oriented water. This concept of the nature of bonding differs somewhat from that proposed by Mitchell, Campanella, and Singh (<u>17</u>), who concluded that interparticle contacts must be effectively solid-to-solid, and subsequent considerations by Mitchell, Singh, and Campanella (18) that suggest that individual atoms in the contact structure are the activating flow units and that a contact may contain many interparticle bonds. In both cases, however, temperature would be expected to have an effect on the ease with which bonds can be broken.

The basic equation for deformation rate  $\dot{\delta}$  developed by Noble and Demirel is

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \exp(\beta \tau)$$
 (13)

where

- A = frequency factor, dependent on entropy of activation, but assumed independent of temperature, T;
- $\Delta H$  = activation enthalpy;
  - k = Boltzmann's constant;
  - $\tau$  = applied shear stress; and
  - $\beta = \frac{\beta'}{kT}$  = coefficient to convert  $\tau$  into an energy term.

The parameter  $\beta'$  is interpreted as the average volume of material containing one bond. For given values of T,  $\delta$ , and A, the shear resistance is

$$\tau = \frac{1}{\beta} \left( \frac{\Delta H}{kT} + \ln \delta - \ln A \right)$$
(14)

Noble and Demirel assume that the same form of relationship can be used to express the peak failure stress,  $\tau_m$ , and that the consolidation stress,  $P_c$ , the normal stress during shear,  $P_{nS}$ , and the water content, w, can be accounted for by using the following equation:

$$\tau_{\rm m} = \frac{1}{\beta} \left( \frac{\Delta H}{kT} + \ln \dot{\delta} - \ln M' \right) + \rho P_{\rm c} + \mu P_{\rm ns} - \gamma w \tag{15}$$

where  $\rho$ ,  $\mu$ , and  $\gamma$  are linear coefficients.

Finally, they noted that the temperature of consolidation,  $T_c$ , should be important (see Fig. 6) as well as the temperature of shear,  $T_s$ , thus leading to the following two equations:

For strength:

$$\tau_{\rm m} = \frac{1}{\theta} \left( \frac{\Delta H}{k T_{\rm s}} + \ln \dot{\delta} + \alpha T_{\rm c} - \ln M \right) + \rho P_{\rm c} + \mu P_{\rm ns} - \gamma w$$
(16)

For deformation rate:

$$\ln \dot{\delta} = \ln M - \frac{\Delta H}{RT_{s}} + \alpha T_{c} + \beta \tau - \beta \rho P_{c} - \beta u P_{ns} + \beta \gamma w$$
(17)

Both constant rate of strain direct shear tests and constant stress creep tests, using stresses only slightly less than the peak strength, were run on the two soil types noted above at several different temperatures. From the results of these tests the linear coefficients in Eqs. 16 and 17 were determined by regression analysis and compared. The values of some of the coefficients deduced in this way show considerable variation (up to more than 100 percent) with variation in normal stress. Similar variations are observed when the values deduced from the creep tests are compared with those deduced from the shear tests. Nonetheless, the authors consider the values sufficiently close to tend to confirm the hypothesis of a single deformation mechanism that is independent of test procedure, and that Eqs. 16 and 17 are suitable for its description.



Figure 14. Influence of temperature on the initial and final stresses in stress relaxation tests on Osaka clay.

The coefficient  $\beta'$  was lowest at the highest normal stresses, which is consistent with the idea of increased numbers of bonds under higher stresses and a decreased volume of influence per bond. For the clay, the value of  $\beta'$  was determined as equivalent to a volume of  $0.6711 \times 10^{-18}$  cm<sup>3</sup>, which is considered by the authors to be of the same order of size as a clay particle. This seems abnormally low, since a clay plate of dimensions  $1\mu \times 1\mu \times 0.1\mu$  would have a volume of  $1.0 \times 10^{-13}$  cm<sup>3</sup>. Both this fact and the observation that the equivalent volume for the silt was even smaller support the hypothesis of Mitchell, Singh, and Campanella (<u>18</u>) that there may be many bonds per particle contact.

Values of activation energy were found to range from 5 to 8 kcal/mole for the silt and from 12 to 21 kcal/mole for the clay. For both soils the higher values were obtained from the direct shear test results. These values are somewhat lower than those summarized by Mitchell, Singh, and Campanella (18) for a variety of soils.

The results of the stress relaxation tests presented by Murayama (19) show that for a given increment of strain both the initial stress and the fully relaxed stress decrease with increasing temperature, as may be seen from Figure 14. His results show also that the rate of stress relaxation,  $-(d\sigma/d \log t)$ , decreases with increasing temperature. While this may at first appear surprising, it should be noted that the relaxation process is not necessarily slower at the higher temperatures, because the difference in initial and fully relaxed stresses was greater at the lower temperatures. Since the ratio,  $E_2/E_1$ , was found to be constant and independent of temperature and the coefficient B decreased only slightly with increasing temperature, the observed behavior of  $-(d\sigma/d \log t)$  is consistent with the predictions of Eq. 12.

# Swelling

Mechanisms of clay swelling have been studied extensively in the past because of the many practical problems associated with construction on or with expansive soils. Attention has been focused on such causes as osmotic pressures, capillary stresses, straightening of bent particles, and ionic and particle surface hydration. Blaser and Scherer (3) provide convincing evidence that significant volume changes may also be associated with what they term "salt heave." Their study grew out of experiences with soils in the southern Nevada area that contain substantial amounts of sodium sulfate.

Blaser and Scherer have demonstrated that damage to light structures and floors was caused as a result of the influence of temperature on the hydration state of sodium sulfate. During warm weather there is an upward migration of water, evaporation, and deposition of salts. Thus the sulfate concentration may be high in the upper few feet. At temperatures above 90 F a large amount of  $Na_2SO_4$  is soluble in water (as much as



Figure 15. Effect of heat treatment on the dry strength of kaolinite.



Figure 16. Effect of heat treatment on the rewet strength of kaolinite.

52 gm per 100 gm water). Temperatures below about 55 F cause crystallization of sodium sulfate, and the crystals that form can bind up to 10  $H_2O$  molecules per molecule of Na<sub>2</sub>SO<sub>4</sub>. These crystallization and crystal hydration processes can cause an expansion against the soil structure and consequent heave of the ground surface.

The authors have found that soils with more than 15 percent of the particles finer than 0.005 mm and containing more than 0.2 percent  $Na_2SO_4$  in solution are subject to salt heave, provided there is sufficient free moisture in the soil and ambient daily temperature variations range from 90 F in the daytime to below 40 F at night. These conditions are met mainly in the late fall and early spring in the Nevada area.

Methods for testing soils for determination of salt heave potential and for treating salt heave-susceptible soils are described by Blaser and Scherer in their very important contribution.

#### THERMAL STABILIZATION

# Effects of Preliminary Heat Treatment

Three papers are concerned with the effects of heat treatment on the engineering properties of soils. Wöhlbier and Henning (32) investigated the effects of heat treatment up to 600 C on the strength and compression properties of kaolinite. Chandrase-kharan et al (7) report on the effects of heat treatment on the pulverization and stabilization of two tropical soils. Barata (2) illustrates the beneficial effects of preliminary drying on the CBR of a residual weathered gneiss soil after rewetting.

Wöhlbier and Henning prepared specimens of kaolinite at an initial water content slightly less than the liquid limit by vacuum extrusion. After careful drying for 28 days the specimens were heated for a period of 48 hours to temperatures up to 600 C, using a heating rate of 60 C per hour. Although heating to 400 C had no effect on the kaolinite, heating to 500 C caused breakdown of the clay mineral structure. After heating to this temperature the clay was no longer plastic.

Some specimens were tested at the end of the heat treatment period, whereas the remainder were resaturated prior to test using a back-pressure technique. Consolidated-undrained triaxial tests were used with confining pressures up to 25 kg per sq cm. The effect of heat treatment on the strength of specimens prior to rewetting is shown in Figure 15, and after resaturation in Figure 16. It is clear from Figure 15 that the strength of the dried specimens was essentially unaffected by temperature of heat treatment in the range of 200 to 400 C. Unfortunately the authors do not present results for dried but unheated specimens for comparison purposes. It may be seen that heating above 400 C leads to a substantial increase in strength.

The results for the resaturated samples, Figure 16, are of the same form, although the curves for 300 and 400 C show a small but significant increase in strength with increase in treatment temperature. The increase in strength resulting from pretreatment at 500 and 600 C is by comparison quite large. It is of interest to note that in all cases the failure envelopes are curved in form. Heat treatment was also observed to cause a significant increase in the stiffness of the clay, and this stiffness increase is important even for treatment temperatures less than 400 C where the strength is not significantly increased.

The beneficial effects on properties of heat treatment at temperatures sufficiently high to cause a change in the clay mineral structure are well demonstrated by the results of this study. Methods for the application of heat treatment, at the high temperatures required, to large masses of soil in the field have been developed and are described by Litvinov (13). The economics of such treatment may be unfavorable, however.

Chandrasekharan et al  $(\underline{7})$  note that insufficient pulverization and inadequate mixing often limit the effectiveness of additive stabilization in highly plastic clays. In their investigation the influences of heat treatment on the pulverization characteristics of a lateritic soil and a black cotton soil were investigated, and the effect of heat treatment and aggregate size on the effectiveness of cement stabilization of black cotton soil was studied. A range of pretreatment temperatures of 100 to 400 C was used. Among their principal findings were the following:

1. Sodium chloride addition in an amount of about 3 percent serves as an aid to pulverization.

2. Heat treatment at 250 C for two hours was found to be the optimum to facilitate easy pulverization.

3. The plasticity of black cotton soil was reduced by treatment at temperatures above 300 C.

4. Soil-cement specimens made of the heat-treated soil were stronger or weaker than the untreated soil depending on whether the coarser or finer aggregate sizes were used.

Barata (2) points out that because of the high daytime temperatures in Brazil there may be significant evaporation from compacted subgrades before paving is completed. Specimens of a weathered gneiss soil, classified as an A-4 (1) in the HRB system, were compacted using the modified AASHO procedure. The samples were then dried for 3 hours at 40 C (104 F) prior to the 4-day soaking period. The CBR was then measured and compared with the value for an identical sample not subjected to drying prior to soaking.

The results of these tests showed that the preliminary drying after compaction resulted in an increase in the CBR of about 6 percent for samples compacted dry of optimum and by about 15 percent for samples compacted wet of optimum. An explanation is given in the paper for the observed behavior in terms of the density increases that occur during drying followed by the volume increases during soaking, and the relationships between these values and contours of equal CBR on the density vs moisture content diagram.

It would appear that the results obtained from this relatively small study are significant and that further investigations should be undertaken to ascertain the full range of property changes that might be obtained using this method in practice.

# Fusion of Soils

Post and Paduana  $(\underline{26})$  have discussed the melting of soils and its possible application to soil stabilization. They show how the approximate melt range of a soil may be determined using a phase equilibria approach. Most soils exhibit incongruent melt ranges rather than a single melt point. Because there are always minor constituents and trace elements in soils, the actual melt ranges will nearly always be about 100 C lower than those deduced from phase diagrams.

Melt range data are presented for several soils and are found to have values between about 1250 and 1750 C, with fat clays near the high end and feldspar-rich clays at the low end of the scale. Soils with a high alumina or lime content may have higher melt ranges. Experimental results are presented to show that melt ranges may be lowered



Figure 17. Effect of Na<sub>2</sub>CO<sub>3</sub> additive on the melt temperature of a granitic soil.

significantly through the addition of a fluxing agent. Figure 17 shows the relationship between melt temperature and amount of sodium carbonate added to a granitic soil. It may be seen that the fusion temperature is reduced up to 350 C through the addition of  $Na_2CO_3$ . The data show also that the actual values of fusion temperature are lower than predicted values as a result of soil impurities.

Post and Paduana state that equipment is now available for producing the heat energy needed for stabilization of soils by fusion. Since soils melt incongruently over a temperature range it is only necessary to reach the initial melt range to achieve cementation of the soil mass. The authors suggest that field tests should be conducted using mobile downdraft heaters and that further studies of fluxing agents be made. Past experience with heat treatment to temperatures sufficient only to cause loss of structural water and mineral transformations (600 C $\pm$ ) indicates that the required quantities of heat are large and the technology is complex, however. It would appear desirable, therefore, to investigate more completely the engineering properties of fused materials, new methods for heat supply, and whether the economics of such treatment are favorable in comparison with other methods for stabilizing soils.

# Ice Barriers

Takashi's paper (30) deals with the problem of forming an ice barrier to cut off groundwater flow. In the presence of flowing groundwater, completion of an ice curtain may be difficult, and in some cases impossible, because of the heat brought into the freezing zone by the flowing groundwater.

The problem of joining two frozen soil cylinders with water flowing between is considered. Equations are developed that give approximately (a) the quantity of flow between the frozen soil cylinders, (b) the pressure distribution along a line between the cylinders in the direction of flow, (c) the flow velocity, and (d) the head difference (dam-up head) that will exist across the ice barrier at the completion of freezing. An equation is then developed that gives the temperature of flowing groundwater after its passage between the frozen cylinders.

The following equation is then derived, which gives a critical value of head drop to be expected across the completed barrier,  $\Delta p_{crit}$ , in terms of soil properties, temperatures of groundwater and coolant, and geometry of the freezing system:

$$\Delta p_{crit} = \frac{2\pi k}{\lambda \gamma c} \frac{\theta_{f} - \theta_{c}}{\theta_{\infty} - \theta_{f}} \cdot F\left(\frac{2a}{L}, b_{crit}\right)$$
(18)

where

k = thermal conductivity of frozen soil;

- $\lambda$  = coefficient of permeability of the unfrozen soil;
- $\gamma$  = density of water;
- c = specific heat of groundwater;
- $\theta_{f}$  = groundwater freezing point;
- $\theta_{\infty}$  = initial temperature of groundwater;
- $\theta_{\rm c}$  = surface temperature of freezing pipe;

 $F\left(\frac{2a}{L}, b_{crit}\right) = a$  nearly linear function of the freezing pipe radius and spacing = 0.6 to 1.9 for (2a/L) = 0.06 to 0.26;

- a = radius of freeze pipe; and
- L = distance between freeze pipes.

If the actual head drop that will develop across the barrier is less than  $\Delta p_{crit}$ , then completion of the ice curtain will be possible. Takashi illustrates by example the influence of each of the quantities in Eq. 18 on  $\Delta p_{crit}$  and shows that seepage flow has an important influence on the freezing process when the permeability is greater than about  $10^{-3}$  m/sec. In this situation the theory presented can be used to determine whether or not an ice curtain can be completed using a given freezing system. The material in this paper should be of great value to engineers contemplating the use of ice barriers for temporary groundwater shut-off.

# TEMPERATURE EFFECTS IN PAVEMENT SUBGRADES

# **Field Observations**

Both Fang (9) and Richards (27) present data on temperatures in and beneath highway pavements and consider the influence of these temperatures on the performance of the soil-pavement system. In this review only those parts of their papers that deal with temperature effects in the subgrade will be considered.

Richards (27) describes the results of an Australia-wide study of temperature beneath sealed pavements. Eighteen field test sites were chosen as representative of the wide range of soil and climatic conditions in Australia. These sites were identical in all respects except soil type and climate. Thermistors were used for temperature measurement at a large number of points at each site. The effect of daily temperature variation was not studied, but it is known to be small below a depth of a few inches. Pavement temperatures over most of Australia range from 5 C to over 60 C.

Individual temperature readings often showed large lateral variations (5 C $\pm$  near the surface) at the same depth and time. Richards suggests that this variation is due more to variations in heat gain or loss at the surface than to differences in thermal diffusivity of the materials. He does not indicate why there should be such variations across a given surface.

From the large amount of accumulated data, some of which are given in the paper, several pertinent trends have been observed. The summer temperatures and the mean temperatures at all depths are significantly greater under a bituminous seal than under natural ground surfaces. In most cases the winter temperature is slightly greater under a bituminous seal.

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The maximum observed temperature gradient over distances of 10 to 15 cm was only about 0.3 C/cm. Richards concludes that a gradient of this magnitude should not cause a moisture flow in excess of about  $10^{-7}$  cm/sec. Since gradients exist at any point for only a short time as a temperature wave moves through the soil profile, thermal moisture transfer in a lateral direction is likely to be small. If it does occur, the gradients are such to cause moisture movement down and away from the centerline. Thus the thermal contribution to overall moisture flow would be to give an increase in stability.

A point of practical importance relative to soil stabilization is brought out by Richards. The temperature of stabilized base courses may rise to 50 to 60 C during the curing period for summertime construction. Under these conditions the usual procedure of curing laboratory specimens at 20 to 25 C may be unduly conservative.

Probably the most significant conclusion Richards draws from this study in terms of the design and performance of pavements is that the major influence of temperature variations is on the load-spreading properties of bituminous layers and treated bases.

Fang (9) has summarized the pavement and subgrade temperature data accumulated during the AASHO Road Test. A brief description of the techniques used to obtain the temperature data are presented. Data are presented that show the effect of temperature on deflections of the soil-pavement system and seasonal variations in the strength characteristics of the soil-pavement system. The effect of temperature on the behavior of the subgrade soil cannot be isolated from many of these data because the measured values include either contributions from the pavement structure or from moisture content variations.

The data indicate that frost penetration was greater under rigid pavements than flexible pavements. This is attributed in part to the greater thermal conductivity of portland cement concrete. Temperature vs depth measurements confirm the earlier statement that daily air temperature variations have little effect on subgrade soil.

# Analysis Methods

The paper by Paaswell (24) has been included in this section of the report since the topic treated, while potentially important in a range of problems, is particularly pertinent to the study of temperature variations beneath pavement structures. The temperature profile that develops in a soil as a result of transient temperature changes at one boundary is studied.

A series of tests on partially saturated soils showed that uniform temperature profiles will be established even when the initial moisture content variations through the sample depth varies erratically by as much as 2 percent. In general, as the saturation increased the amount of moisture movement near the points of maximum temperature gradient decreased. This would be expected since the tests were run on sealed samples and, therefore, as the saturation increases there is less vapor phase transport and less free space for water to move into. From the results of these tests Paaswell concludes that the soil-water system can be treated as a homogeneous unit for the purpose of making preliminary temperature distribution analyses.

The heat diffusion equation was solved for the case of a cylindrical volume of soil subjected to a sinusoidal temperature variation with time at the top boundary. Predicted temperatures as a function of time for different depths were compared with measured values, and the agreement was fairly good for the first pulse. Paaswell concludes with a discussion of the complex interrelationships between pore pressure, water flow, and volume change that develop during a temperature change.

# CONCLUSIONS

This general report represents an attempt at summarizing the most significant concepts and conclusions advanced by the authors of the 14 papers included in Part III of this volume. The papers treat a variety of topics relating to the influences of temperature and heat on the engineering behavior of soils. A few of the main points are here singled out that, by virtue either of their great importance or because they are controversial, should be subjects of further study. 1. Evidence on the influence of temperature on secondary compression rates is conflicting, with some investigators indicating little effect and others indicating that secondary compression rates are greatly increased as temperature increases. Can this question be resolved on the basis of data already available, and if not what further work is needed?

2. Volume changes and pore pressure variations that develop when saturated soils are subjected to a temperature change are reasonably well understood. Expressions are available for their estimation in terms of the thermal expansion of the water and soil grains, the compressibility of the soil structure and water, and some type of physicochemical temperature coefficient of soil structure volume change. Arguments have been presented to account for the volume decrease (under drained conditions) of a soil structure with increased temperature based on (a) changes in double layer repulsive forces with temperature and (b) variation in the strength of the interparticle contact zone structure with strength. On the basis of the analysis presented in this paper, can we draw positive conclusions concerning the influence of temperature on double layer repulsions? What are the effects of temperature variations on other types of interparticle forces? And which of the two arguments stated above is the most reasonable to account for the behavior?

3. The preponderance of available data indicates that, for other factors constant, an increase in temperature leads to a decrease in strength. What changes, if any, should be made in laboratory testing, analysis, and design procedures for field application?

4. The temperature-dependence of deformation rates in conjunction with rate process theory has been used in a number of recent studies to draw conclusions concerning the mechanism of deformation and interparticle bonding characteristics in soils. Critical discussion of this approach is needed.

5. A "salt heave" swelling mechanism that is dependent on temperature changes has been identified by Blaser and Scherer. Further study of this phenomenon should be made. Are there other temperature-related chemical phenomena, heretofore unrecognized, that should be investigated as possible causes of volume or strength change?

6. The partial drying of compacted soil prior to soaking appears to have a beneficial effect on strength. Are there additional data beyond that presented by Barata? Are there conditions where pre-drying may be detrimental?

7. It is clear that heating soils to high temperatures can result in marked improvements in stress-deformation properties. What is the outlook for economical thermal treatment methods in the future?

8. A method has been developed by Takashi for determination of whether or not a frozen ice barrier can be successfully completed under flowing groundwater conditions. How well can this method be expected to work in practice? Have alternative approaches been developed?

9. While it has been well established that temperature variations have a significant effect on the load distributing characteristics of a pavement structure, it would appear that temperature variations in the subgrade have little influence on behavior, with the exception of frost effects. Is this a reasonable conclusion?

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