

Temperature Effect and the Concept of Effective Void Ratio

by

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Introduction

Effects of heat and temperature on consolidation behavior of soil have been a subject of much research mostly of experimental nature. A summary of pertinent literature is presented below.

Temperature Effect on Void Ratio

The significance of testing temperature on void ratio was first pointed out by Casagrande (1936) and Gray (1936, 1938). They reported results of consolidation tests on remolded samples of an organic silty clay. A reduction in void ratio was observed for an increase in consolidation temperature from 11°C to 22°C. In their tests, different rooms at different temperature were employed and to change the sample temperature, the consolidometer was removed from a room at a given temperature and transferred to another room at the desired temperature. Their findings were later supported by results of tests by Lambe (1960), Campanella (1965) and Campanella and Mitchell (1968) who performed triaxial consolidation tests on four samples of remolded illite and by findings of Habibagahi (1969, 1973).

A different conclusion was however reached by Finn (1951) who carried out nine consolidation tests on remolded samples of slightly organic silt. Two tests were run at 80°F (26.5°C), three at 70°F (21°C), two at 50°F (10°C), and two at 40°F (4.5°C). It was concluded that temperature had no significant effect on void ratio. This observation was later substantiated by test results of Buchanan (1964) in testing remolded samples of sodium and calcium illite and by findings of Matlock and Dawson (1951).

Temperature Effect on Coefficients of Compressibility and Consolidation

The effects of temperature on coefficients of compressibility, a_v , and consolidation, c_v , was first reported by Lewis (1950) who tested samples of London clay at a number of temperatures between 15°C and at 5°C. He concluded that a change in testing temperature had little influence on a_v but c_v was significantly affected. Furthermore, satisfactory temperature corrections to c_v were made by making corrections for variations in kinematic viscosity of water with temperature. The same conclusions were reached by several other investigators (Buchanan, 1964, Finn, 1951, Habibagahi, 1969). Simon (1965) carried out temperature controlled consolidation tests on samples of normally consolidated Fornebu clay and

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observed an 11% decrease in the coefficient of volume compressibility, m_v , with a decrease of temperature from 21°C to 6.5°C. The same trend was later observed by Plum and Esrig (1969) in testing samples of illite.

In summary, the data available thus far on temperature effect on primary consolidation appears to be far from conclusive and more work in this area is needed for a better understanding of temperature effects on soils which in turn leads to a better evaluation of the factors affecting physical properties of soils. The purpose of this study is to present and analyze the data obtained in performing temperature controlled consolidation tests on normally consolidated and overconsolidated samples of a slightly organic soil. It is found that the temperature effects can be clearly explained in light of the effects of temperature on soil-water structure system.

Experimental Set Up

Apparatus

All tests were carried out in stainless steel consolidometers with a fixed ring, 2.5 inches (6.35 cm) inside diameter, and .75 inches (1.9 cm) in height. Two types of rings were employed, one was made of stainless steel with a coefficient of thermal expansion of 17.8×10^{-6} 1/°C and the other of invar steel which has a very low coefficient of thermal expansion of 0.9×10^{-6} 1/°C. Each cell was surrounded by a temperature controlled water bath which maintained a constant temperature around the sample. Water temperature fluctuations were always kept within .5°C of the desired testing temperature.

Sample Preparation and Soil Tested

A sufficient amount of soil was sampled and brought to the laboratory. The soil was air dried, pulverized, passed No. 40 sieve and then stored. The soil tested was a slightly organic silty clay (OH) with a liquid limit of 70% and a plasticity index of 25%. The basic clay mineral of the soil was illite (70%) and chlorite (20%). The organic carbon content of the soil as obtained by wet-combustion procedure Allison (1960) was 5.7%.

Laboratory Consolidation Tests

The processed soil was mixed at a room temperature of 23°C to a water content slightly above liquid limit and allowed to hydrate for twenty four hours at 100% humidity. The soil was then reworked to attain a uniform moisture content prior to start of consolidation tests. In each test the sample was loaded in increments to 8 ksf (3.9 kg/cm²), unloaded to .0625 ksf (0.0305 kg/cm²), reloaded to 64 ksf (31.2 kg/cm²) and then unloaded to zero pressure. A typical loading, unloading, and reloading cycles is illustrated in Figure 1 which shows the relationship between void ratio and logarithm of pressure for the first and second loading cycles at a constant temperature of 25°C.

Analysis of Results

Void Ratio-Pressure Relationship

The influence of testing temperature on void ratio pressure curves is presented in Figures 2 to 6. In Figure 2 the temperature effect on void

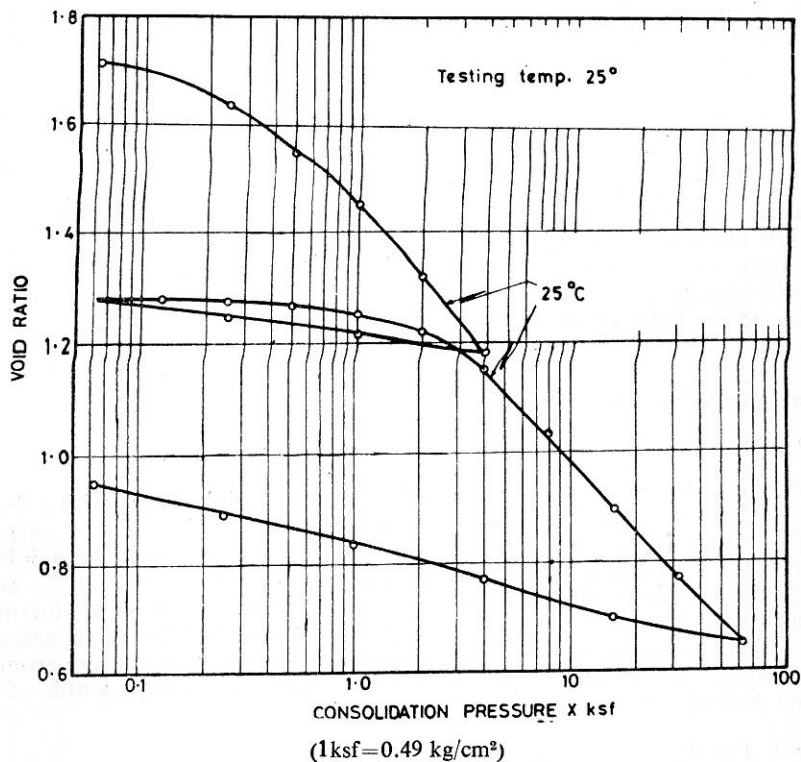


FIGURE 1. Typical cycles of loading

ratio of normally consolidated sample is shown. After performing the first loading and unloading cycle at a constant temperature of 25°C, the temperature was increased to 50°C under an applied reloading pressure of 64 ksf (31.2 kg/cm²). The increase in temperature resulted in a reduction of void ratio under constant pressure.

This reduction in void ratio is attributed to thermal loading of the sample as can be seen more clearly by examining Figure 3 which shows the deformation-time plot for a constant consolidation pressure of 64 ksf (31.2 kg/cm²). After reaching equilibrium under the applied pressure (1440 minutes), an increase of 25°C in the testing temperature was introduced. This resulted in an additional thermal compression of the sample as shown by curve A. If curve A is replotted as a new load increment it will appear as shown in Figure 4 which represents the thermal deformation-logarithm of time relationship of the sample. This relationship is quite similar to that obtained under mechanical loading of the same soil involving dissipation of excess pore water pressure. The mechanism of generation of excess pore water pressure due to an applied thermal load is examined in detail subsequently after defining the concept of effective void ratio.

The effect of temperature on void ratio of normally consolidated and overconsolidated samples is depicted in Figure 5 which shows the results

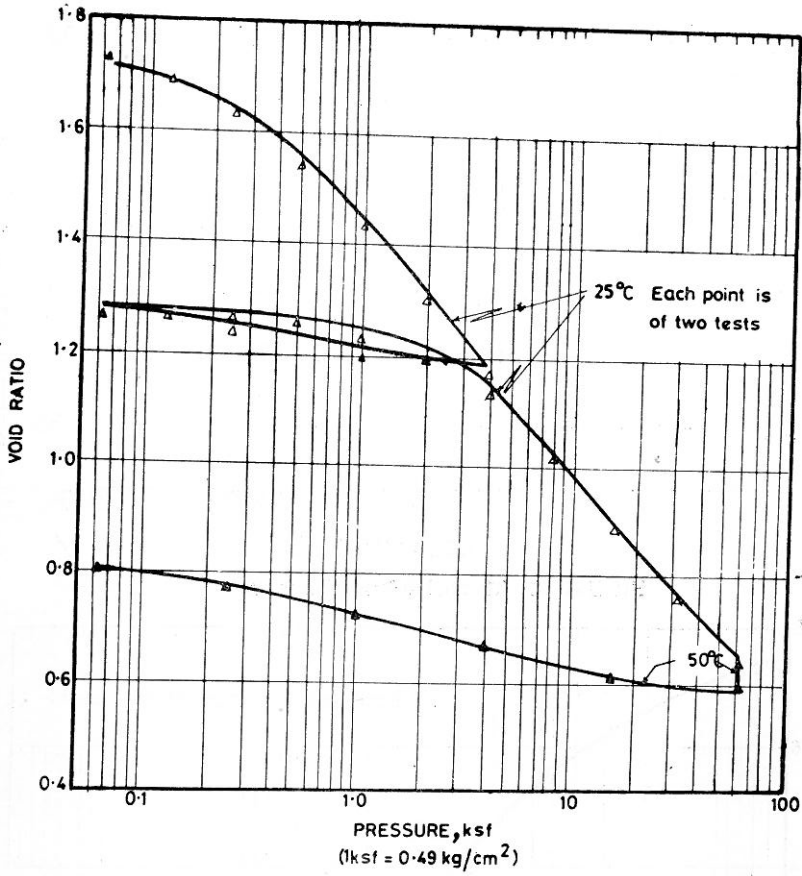


FIGURE 2. Temperature effect on void ratio of normally consolidated soil

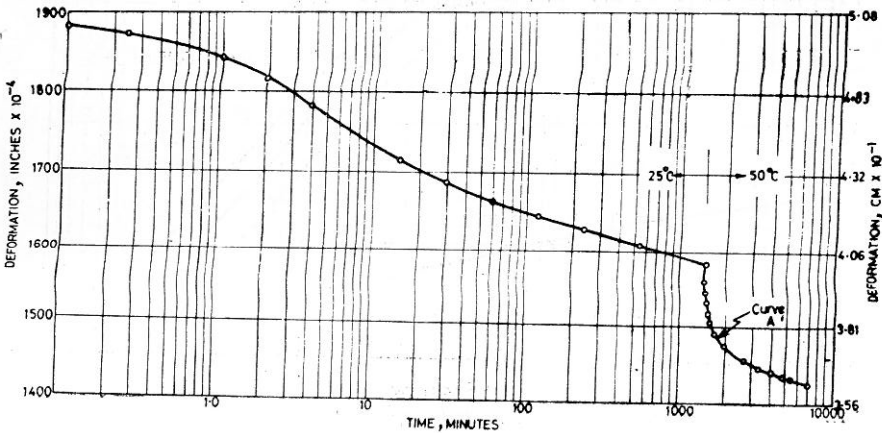


FIGURE 3. Temperature effect on sample deformation, consolidation pressure = 64 ksf (31.20 kg/cm²)

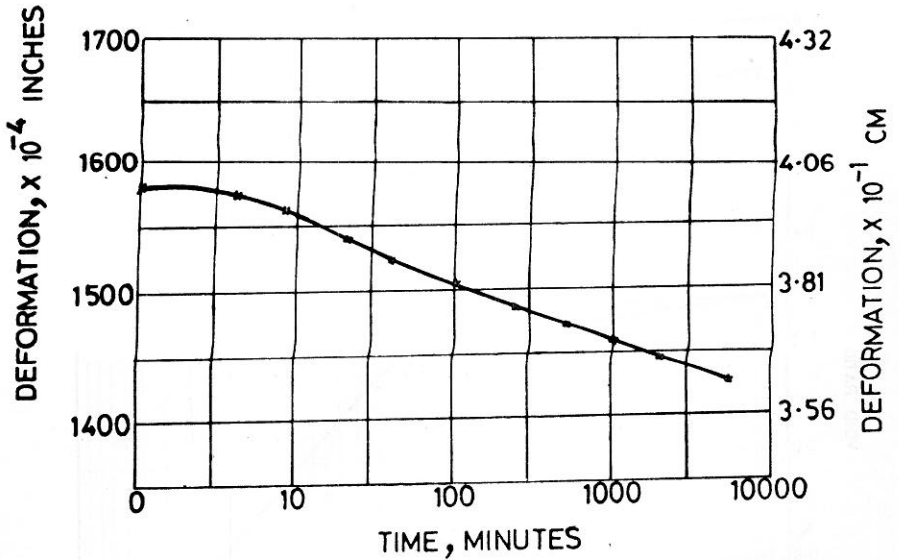


FIGURE 4. Thermal deformation vs time

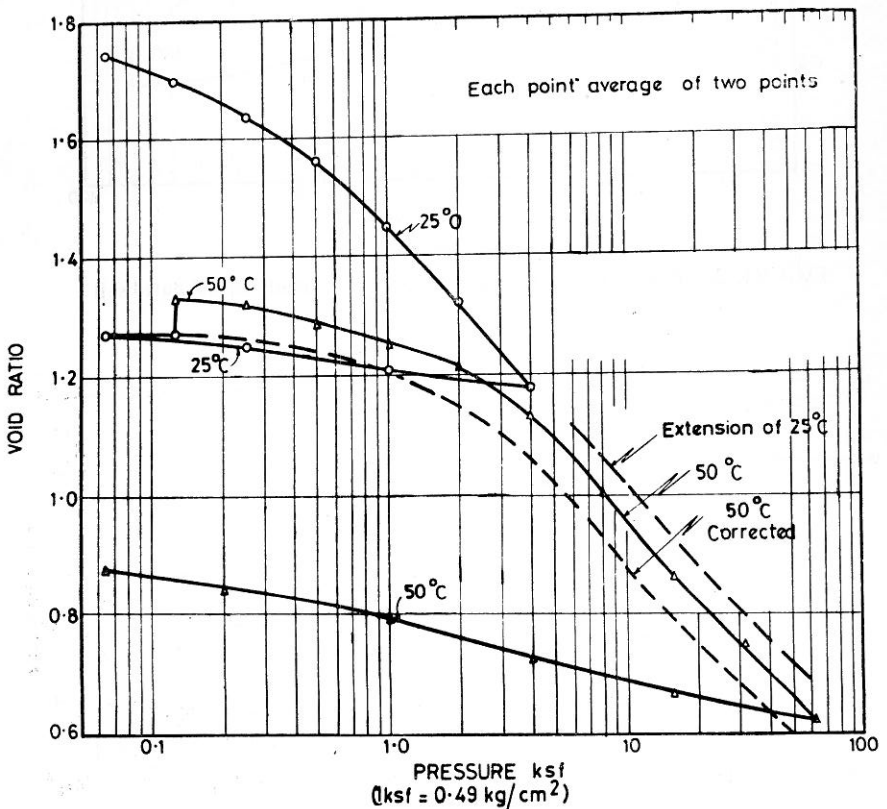


FIGURE 5. Temperature effect on void ratio of normally consolidated and overconsolidated sample

of two tests. In these tests the temperature was maintained at 25°C for the first loading and unloading cycles and changed to 50°C at the first reloading pressure (.125 ksf or 0.06 kg/cm²). The increase of the testing temperature resulted in an initial swelling of the sample followed by a gradual drop of the $e-\log P$ curve. The magnitude of this drop can be seen from the difference between the dashed curve, which represents the estimated location of the reloading curve at 25°C, and the solid reloading curve at 50°C. The tests were carried out in an Invar steel ring. The swelling phenomenon is mainly the result of gas generation in the sample due to increase in temperature. In some tests gas bubbles could be seen as they were forming around the consolidometer loading cap. A corrected curve for 50°C is then obtained by eliminating the initial swelling. This curve is shown in Figure 5 by the dashed curve at 50°C.

In another series of two tests the first loading and unloading cycles were performed at 50°C and the second loading and unloading cycles at 25°C. The results are summarized in Figure 6. The temperature change took place at the beginning of the second loading cycle consolidation pressure of .125 ksf (0.06 kg/cm²). The thermal unloading of the sample did not significantly alter the void ratio of the overconsolidated sample. This confirms the previous observation that the swelling of the over consolidated sample with an increase in the testing temperature is due to gas generation, otherwise with thermal unloading an opposite behavior would have been expected. A thermal unloading of the sample resulted in an upward shift of the $e-\log P$ curve.

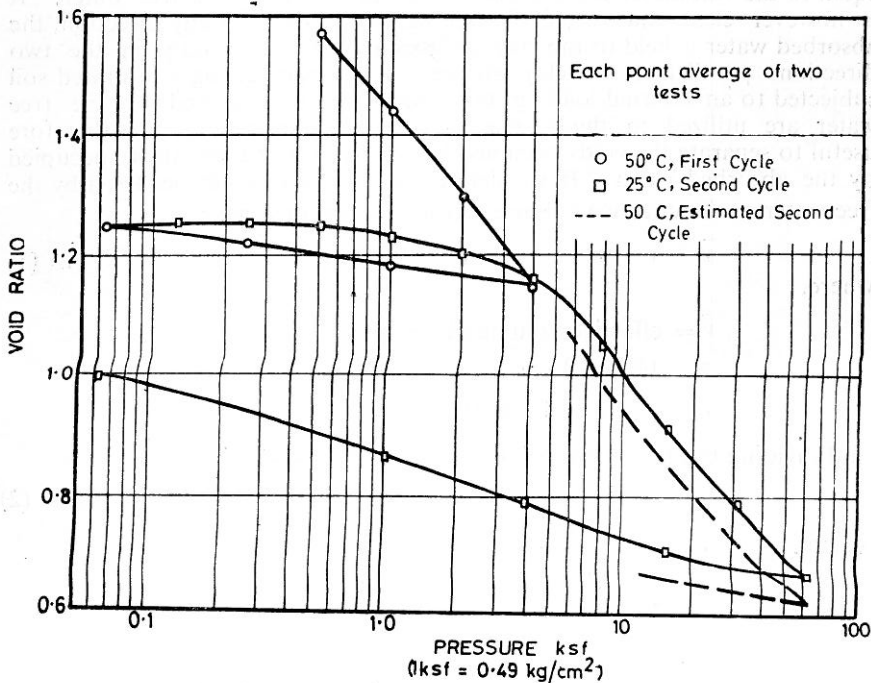


FIGURE 6. Temperature effect on void ratio of normally consolidated and overconsolidated sample

Effective Void Ratio

In order to explain the temperature effect on void ratio, it is necessary to look into the soil-water system as a whole and examine the temperature effect on various components. In general the structure of a saturated fine-grained slightly organic soil is made up of four parts, the soil solids, the organic matter, the free fluid and the adsorbed fluid. This is illustrated in Figure 7 which shows an exaggerated view of these components.

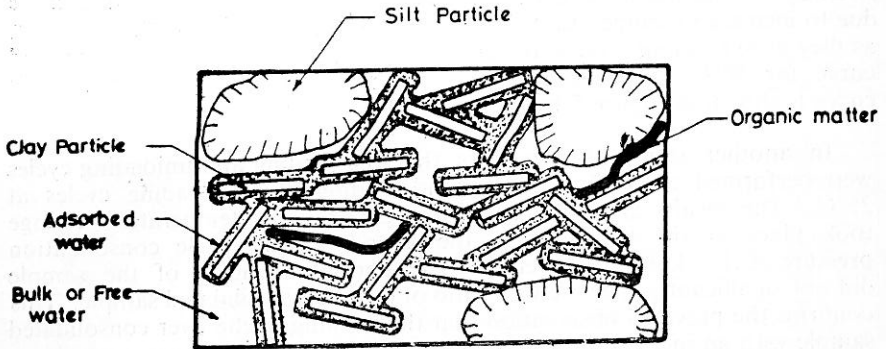


FIGURE 7. Model of a saturated slightly organic soil

Under an applied consolidation pressure, the volume of voids is assumed equal to the volume of the adsorbed water plus that of the free water. It is however clear that while the free water can move in any direction, the adsorbed water is held to the clay surfaces and can move only in the two directions parallel to the clay surface. When considering a saturated soil subjected to an external loading, the voids that are occupied by the free water are utilized in the dissipation of excess pore water. It is therefore useful to separate the voids occupied by the free water from those occupied by the adsorbed water. If we define the volume of voids occupied by the free water as the effective volume of voids, \bar{v} , we then have:

$$\bar{v} = v - v_o \quad \dots (1)$$

where,

\bar{v} = effective volume of voids

v = total volume of voids

v_o = volume of adsorbed water

and dividing both sides by volume of solids we have:

$$\bar{e} = e - e_o \quad \dots (2)$$

where,

\bar{e} = effective void ratio

e = total void ratio

e_o = adsorbed water void ratio

= void ratio of the sample when all of the water in the sample is adsorbed water

From Equation (2) it is clear that \bar{e} can be computed if e and e_0 are known. For a given sample e can be computed by performing standard laboratory tests and e_0 can be obtained as follows.

When we have void ratio of the sample equal to e_0 , all of the voids are filled up with adsorbed water and thus we expect zero permeability. It is assumed that the system is saturated and voids are interconnected. Now, if e is plotted versus k , at $k=0$, the value of e_0 can be obtained. Such a plot for the soil tested is presented in Figure 8. The $e-k$ relationships

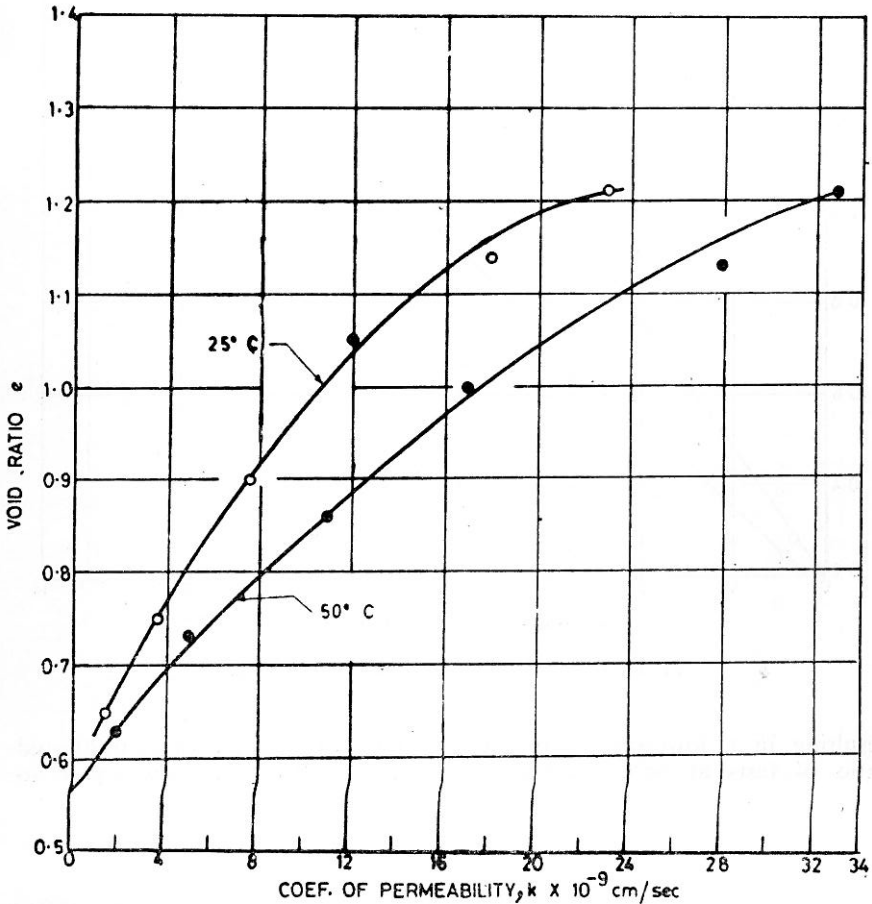


FIGURE 8. Void Ratio vs coefficient of permeability for two different temperatures

on this diagram, turn out to be curves. In order to define the intercept more clearly it is desirable to replot the relationship between e and k such that a straight line is obtained. For the samples tested e^3 versus k at any temperature is close to a straight line as shown by Figure 9. From this figure e_0 at 25°C equals to 0.60 and at 50°C equals to 0.53.

The temperature therefore influence e_0 . As temperature is increased adsorbed water is transformed into liquid water and then dissipated.

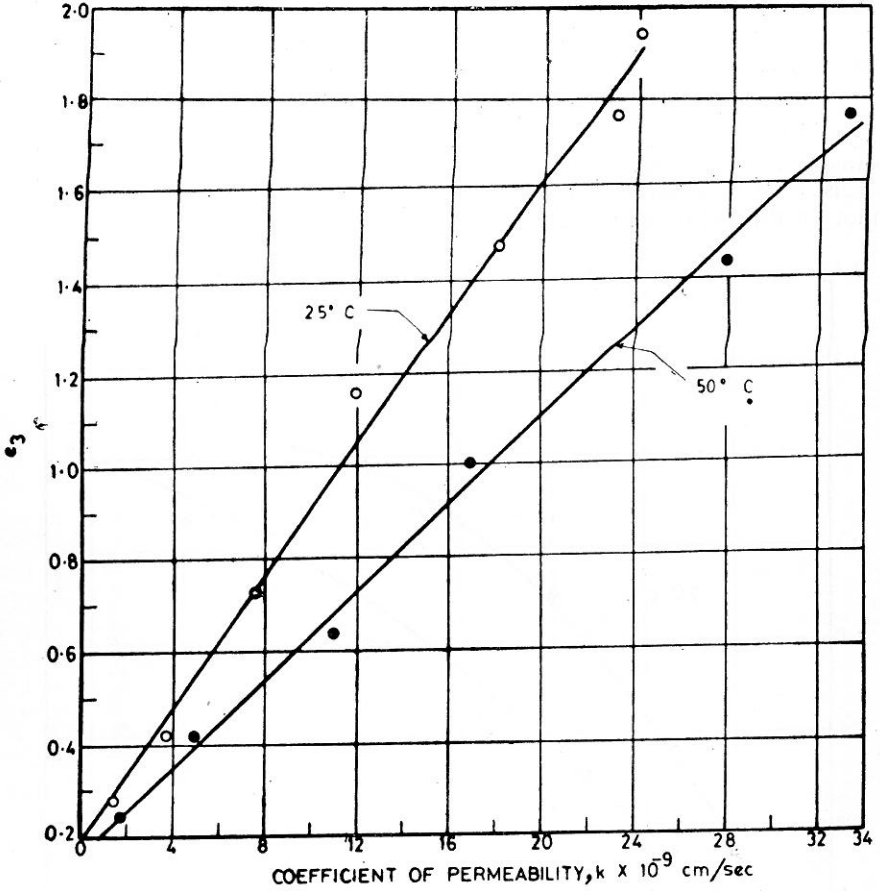


FIGURE 9. e^3 vs k for different temperatures

resulting in a lowering of the total void ratio. The drop in total void ratio of tests at 50°C compared to those at 25°C is then equal to (at equal pressure)

$$\Delta e = 0.6 - 0.53 = 0.07$$

which agrees very well with the test data of total void ratio versus logarithm of pressure at 25°C and 50°C shown in Figure 5.

The effective void ratio remains constant irrespective of temperature. Hence a more general compression diagram can be obtained by plotting the effective void ratio versus effective consolidation pressure. Such a plot is presented in Figure 10 which is obtained from Figure 5 by making the necessary corrections to obtain \bar{e} . From an examination of Figure 10 it is clear that the plot of \bar{e} vs $\log \bar{P}$ is not influenced by the testing temperature.

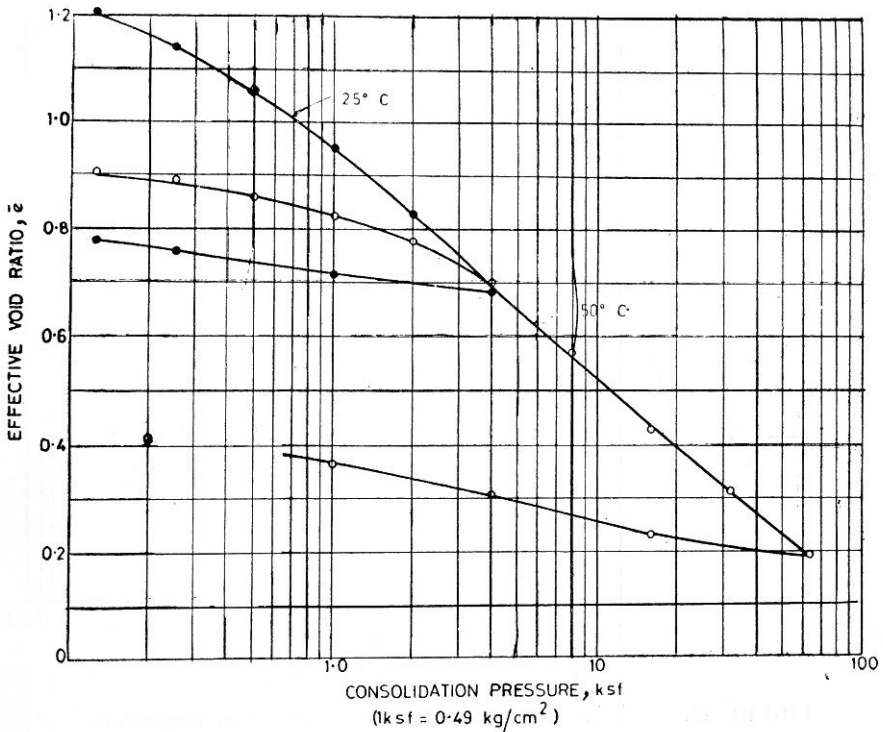


FIGURE 10. Effective void ratio vs effective pressure at two different temperatures

Coefficient of Compressibility

Variation of the coefficient of compressibility with temperature is summarized in Figure 11, which shows the results of four consolidation tests. The first loading cycle of these tests were performed at 25°C. The second loading cycle of two of the tests were at 25°C and that of the other two tests at 50°C. From an examination of the second loading cycle in Figure 11 it can be seen that the coefficient of compressibility for the overconsolidated condition ($p \leq 3.2$ ksf, 1.57 kg/cm²) has increased with temperature. As the sample approaches normally consolidated condition ($p \geq 3.2$ ksf, 1.56 kg/cm²) the influence of temperature diminishes and for normally consolidated state temperature has no effect on the a_v .

The behavior of the overconsolidated sample is attributed partly to gas generation in the sample as was discussed previously and partly to the fact that with an increase in temperature the overconsolidated sample undergoes a structural breakdown which results in an increase in a_v . For remolded normally consolidated sample the structural breakdown has already taken place during remolding and thus no further breakdown takes place with temperature. These observations are supported by the previously reported data on other soil types in that a_v is independent of the testing temperature when tests have been performed on remolded normally consolidated soils (Andrade, 1930, Campanella and Mitchell, 1968, Finn, 1951, Gray, 1938) and dependent on temperature for undisturbed or sedimented soil samples (Plum and Errig, 1969, Simons, 1965) which undergo a structural

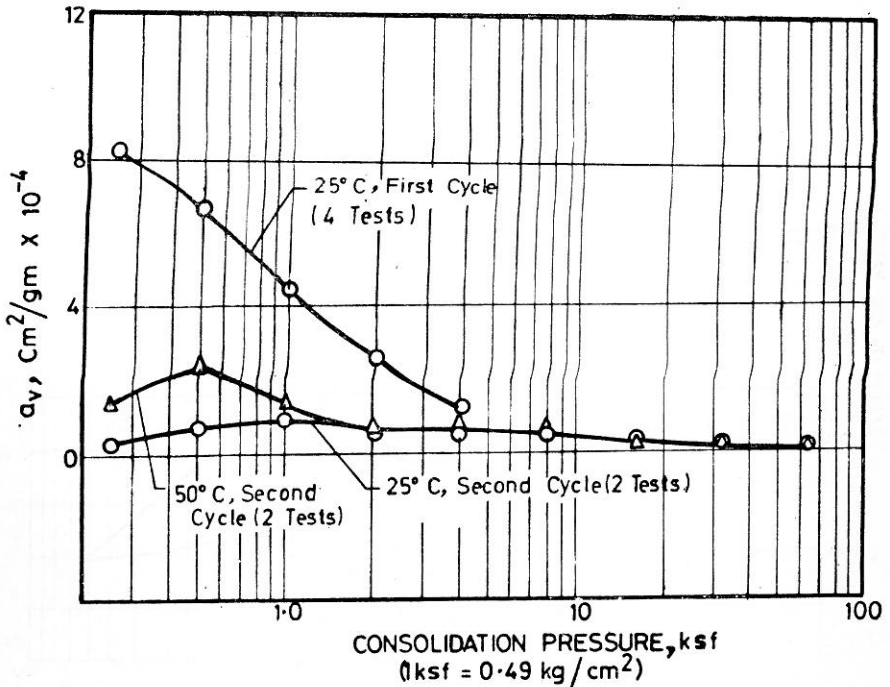


FIGURE 11. Variations of coefficient of compressibility with temperature

weakening with an increase in temperature. It is therefore concluded that a_v is independent of testing temperature for normally consolidated remolded samples and a function of temperature for overconsolidated, or undisturbed, or sedimented soil samples.

Coefficients of Consolidation and Permeability

Theoretical Considerations: Using the soil structure model presented in Figure 7, we observe that for the temperature ranges studied herein, soil solids will not be seriously influenced with a change in the testing temperature as compared to the pore fluid. A change in the temperature affects the viscosity of the fluid phase which in turns influences the coefficients of permeability and consolidation.

Newton was the first to formulate an hypothesis regarding the magnitude of force necessary to overcome viscous resistance. Thus the following equation was developed:

$$\tau = \eta \frac{dv}{dy} = \eta \dot{\gamma} \quad \dots(3)$$

in which τ is the shearing or frictional force between the layers per unit of contact area, $dv/dy = \dot{\gamma}$ is the velocity gradient and η is known as coefficient of viscosity or viscosity.

Many empirical relationships have been presented relating viscosity with temperature. These formulas have in general three or more arbitrary

constants. Andrade (1930) formulated the following relationship which has only two arbitrary constants.

$$\eta = Ae^{-\frac{B}{T}} \quad \dots(4)$$

where η is the coefficient of viscosity, e is the base of natural logarithms (equal to 2.718), T is the absolute temperature, and A and B are constants. This formula fits experimental data for a large number of fluids; however, the fit is poor for water and other associating liquids. Andrade therefore modified the formula to the following form to reduce this discrepancy,

$$\eta = A'e^{-\frac{B'}{T-\theta}} \quad \dots(5)$$

where A' , B' and θ are constants. Even though this formula fits the test data for water better than Equation (4), it has an additional constant which is not desirable. Thus, many investigators have used Equation (4) for water (Low, 1959, Paaswel, 1967) because of its simpler form. From an examination of equation (4) it is clear that as temperature is increased viscosity decreases. The relationship for water is presented in Figure 12 for a temperature range of 0 to 109°C.

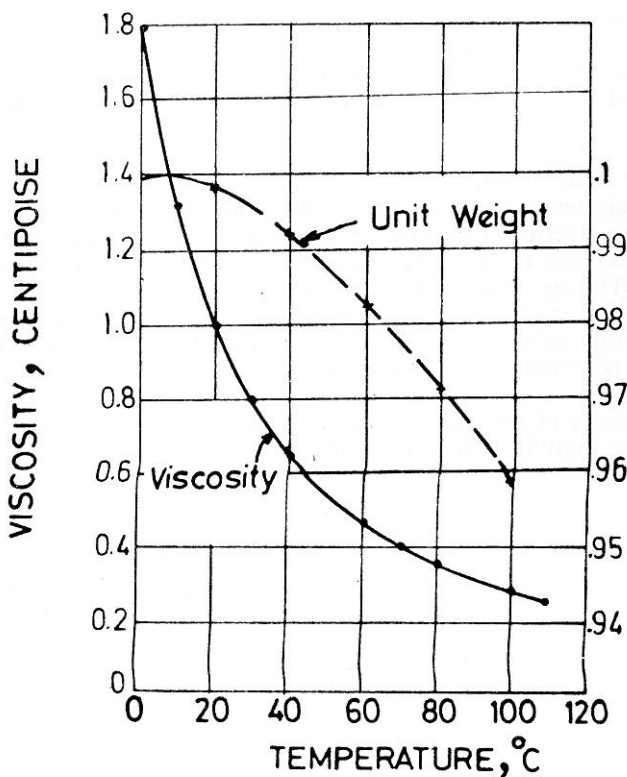


FIGURE 12. Viscosity and unit weight of water at various temperatures at 1 atmosphere

The coefficient of permeability, k , as obtained from Darcy's law is a function of the properties of the permeant such as its viscosity. The properties of the permeating fluid are in turn influenced by temperature and therefore, k is a function of temperature. A more general definition of the coefficient of permeability is one which is independent of the properties of the permeant. Absolute permeability, K , as defined by the following equation is an attempt in this direction:

$$K = \frac{k\eta}{\gamma} \quad \dots(6)$$

in which γ is the unit weight of permeating fluid. It has been stated that the absolute permeability is constant for all fluids and all temperatures provided that the soil structure and void ratio are unchanged (Musket, 1937 Taylor, 1948, Terzaghi, 1943). Tests by Wyckoff et al (1934) on flow of water, air and carbon tetrachloride through sand and sandstone showed K is in fact fairly independent of the type of the permeant. Lambe (1954) performed a similar series of tests on clay and noticed that only at high void ratios was K somewhat different from one test to another. He therefore, suggested that another fluid property such as its polarity might have to be included in Equation (6) when used for clays. In his tests Lambe replaced the permeating water with dioxene, acetone, and dry nitrogen. Physico-chemical properties of these fluids differ considerably. In particular, the dipole moments varies from zero for nitrogen to 1.84 debye for water is a highly polar fluid. At 20°C the dielectric constant is 1 for nitrogen and 80 per water (Weast, 1968). Since temperature affects some of the physico-chemical properties of soil, it appears that K may not be completely independent of temperature.

For the tests reported herein, water was used as the permeating fluid. The maximum temperature change used in this study was 25°C which is too small to have a serious influence on dipole moment and dielectric constant. For example, the dielectric constant of water changes from 70 at 50°C to 78 at 25°C (Dorsey, 1968). Thus, the influence of polarity on K is limited for the tests reported herein. In using Equation (6), it is also assumed that the total void ratio is not affected by the testing temperature. This assumption is invalid as was discussed in previous section.

The influence of the temperature on void ratio can be accounted for by the following modification of Equation (6):

$$K = \frac{k\eta}{e_o \gamma} \quad \dots(7)$$

where e_o is the void ratio of the adsorbed water. From the test results computed K values at 25°C and 50°C, obtained by Equation (7), are in fact identical.

Since K does not significantly change with temperature the following relationship can be derived for coefficients of permeability of tests at two different temperatures T_1 and T_2 :

$$\frac{k_1 n_1}{e_{o1} \gamma_1} = \frac{k_2 n_2}{e_{o2} \gamma_2}$$

$$\frac{k_1}{k_2} = \frac{n_2 e_{o1} \gamma_1}{n_1 e_{o2} \gamma_2} \quad \dots(8)$$

And for coefficients of consolidation we have

$$\frac{C_{v1}}{C_{v2}} = \frac{k_1(I+e_1) a_{v2} \gamma_{w2}}{a_{v1} \gamma_{w1} k_2(I+e_2)} \quad \dots(9)$$

Variation of the unit weight of water with temperature to be used in Equations (8) and (9) is presented in Figure 12.

$$\text{Since } e = e_o + \bar{e}, a_{v1} = a_{v2}, \text{ and } \frac{k_1}{k_2} = \frac{\eta_2 e_{o1} \gamma_{w1}}{\eta_1 e_{o2} \gamma_{w2}}:$$

$$\frac{C_{v1}}{C_{v2}} = \frac{(I + \bar{e}_1 + e_{o1}) \eta_2 e_{o1}}{(I + e_2 + e_{o2}) \eta_1 e_{o2}}$$

Assuming $\eta_2 e_{o1}^2$ and $\eta_1 e_{o2}^2$ are small, the equation simplifies to:

$$\frac{C_{v1}}{C_{v2}} = \frac{\eta_2 e_{o1}}{\eta_1 e_{o2}} \quad \dots(10)$$

Test Results

The effect of testing temperature on the coefficients of permeability and consolidation is illustrated in Figures 13, 14, 15, 16, 17 and 18. The relationship between the coefficient of permeability versus logarithm of pressure at various temperatures for normally consolidated and overconsolidated samples is shown in Figures 13 and 14. For any given consolidation pressure, an increase in the temperature resulted in an increase in the computed coefficient of permeability. The relationship between the total void ratio versus logarithm of the coefficient permeability is plotted in Figure. 15.

The $e - \log K$ curves for 25°C and 50°C are "parallel" to each other with $e - \log K$ of the higher temperature located below that of the lower temperature. Since these curves are plotted in terms of the total void ratio, they represent the combined effect of temperature on both void ratio and the coefficient of permeability. In order to examine the temperature effect on the coefficient of permeability alone, it is necessary to plot the effective void ratio versus the coefficient of permeability as shown in Figure 16. An examination of Figure 16 reveals that at any given effective void ratio, an increase in testing temperature has resulted in an increase in the coefficient of permeability.

Comparisons of Experimental Results with Theoretical Prediction

The ratios of the coefficients of permeability at 50°C to that at 25°C for normally consolidated and overconsolidated samples obtained from Figures 13, 14 and 16 are summarized in Table 1. The ratio of the coefficient of consolidation at 50°C to that at 25°C for normally consolidated samples

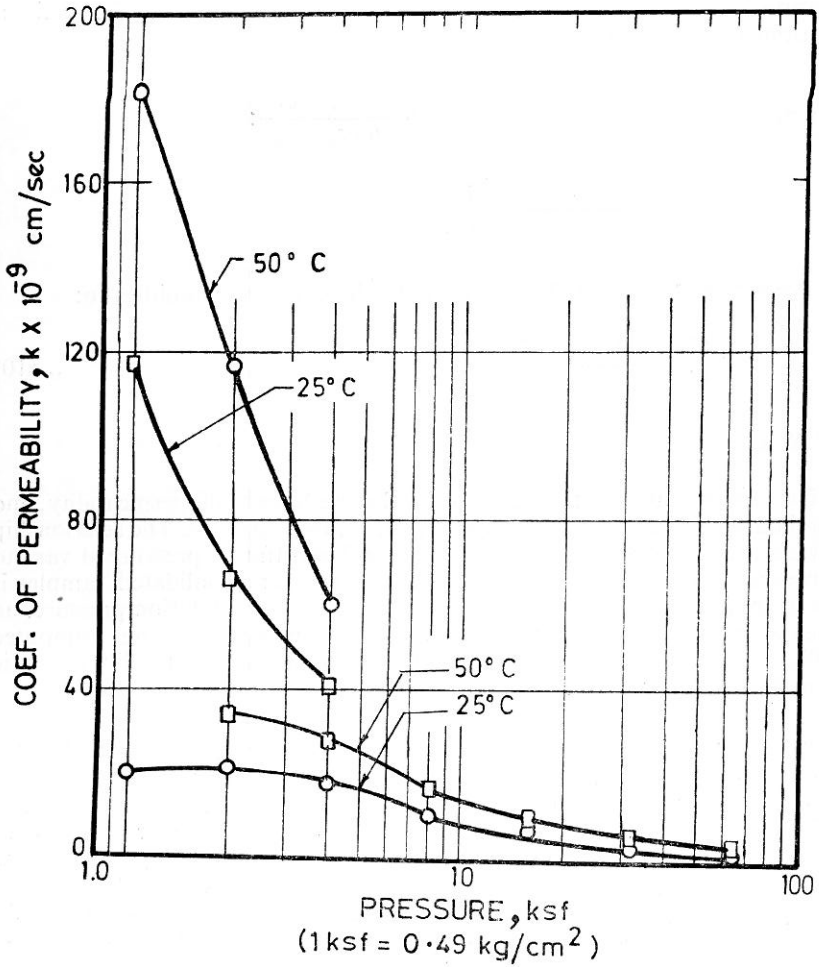


FIGURE 13. Variations of the coefficient of permeability with temperature

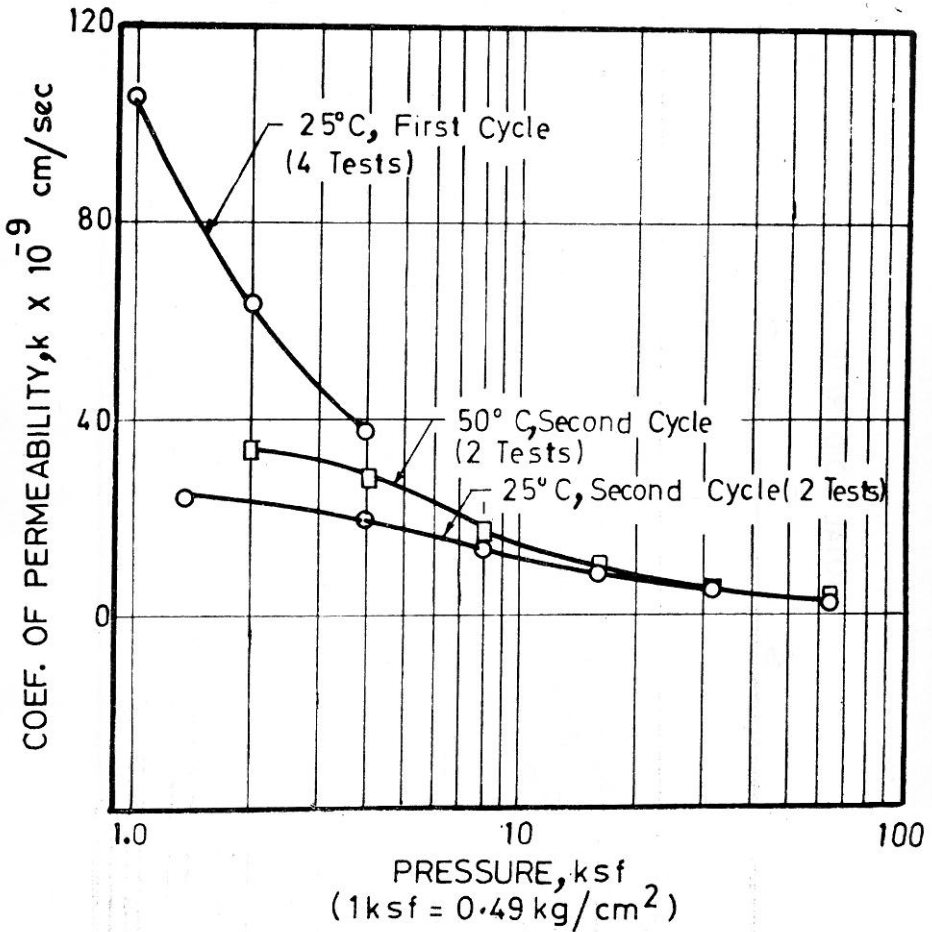


FIGURE 14. Variations of the coefficient of permeability with temperature

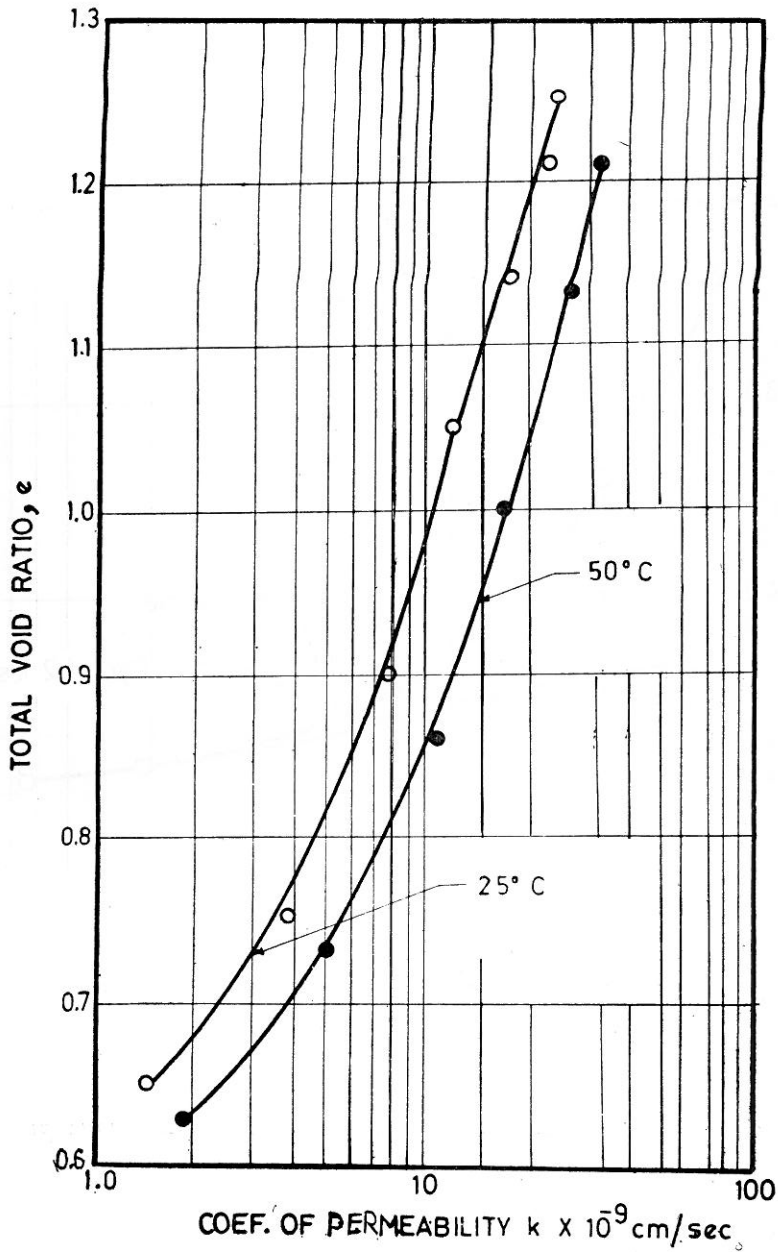


FIGURE 15. Total void ratio vs K at different temperatures

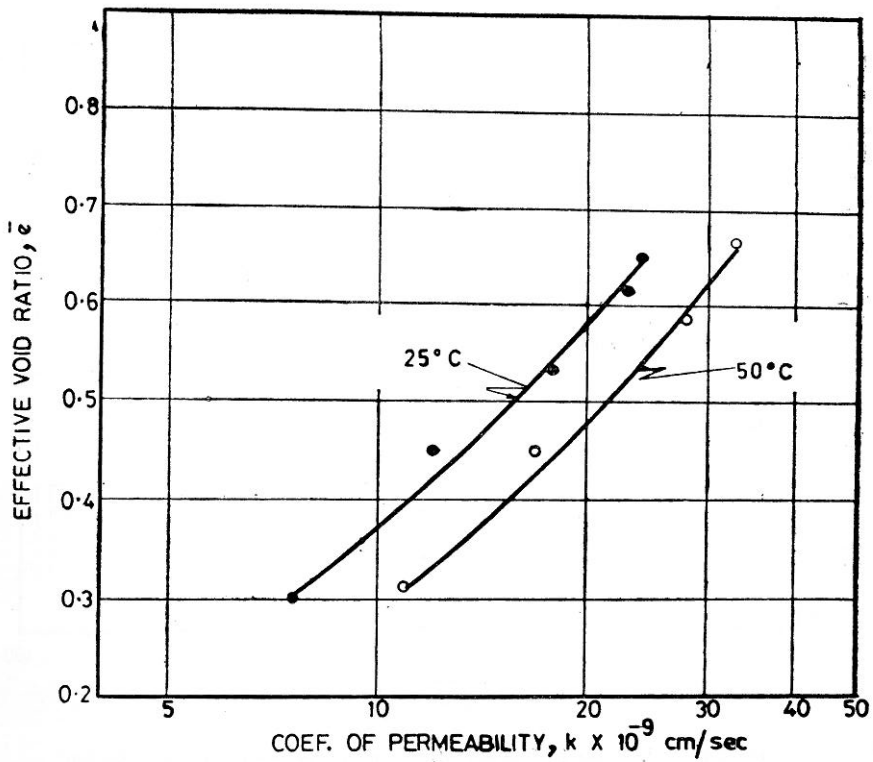


FIGURE 16. Effect of temperature on the coefficient of permeability

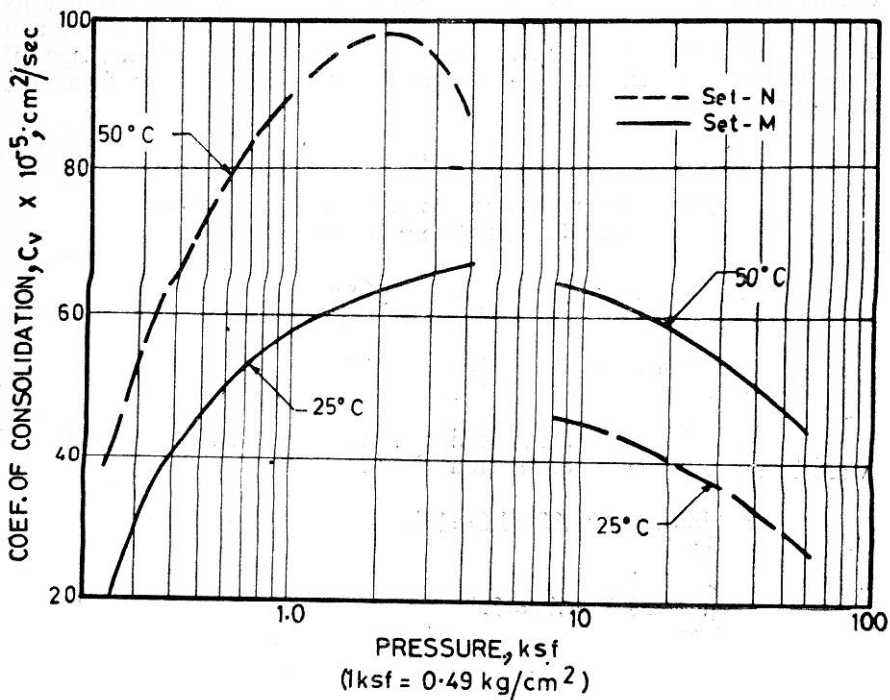


FIGURE 17. Effect of the temperature on the coefficient of consolidation

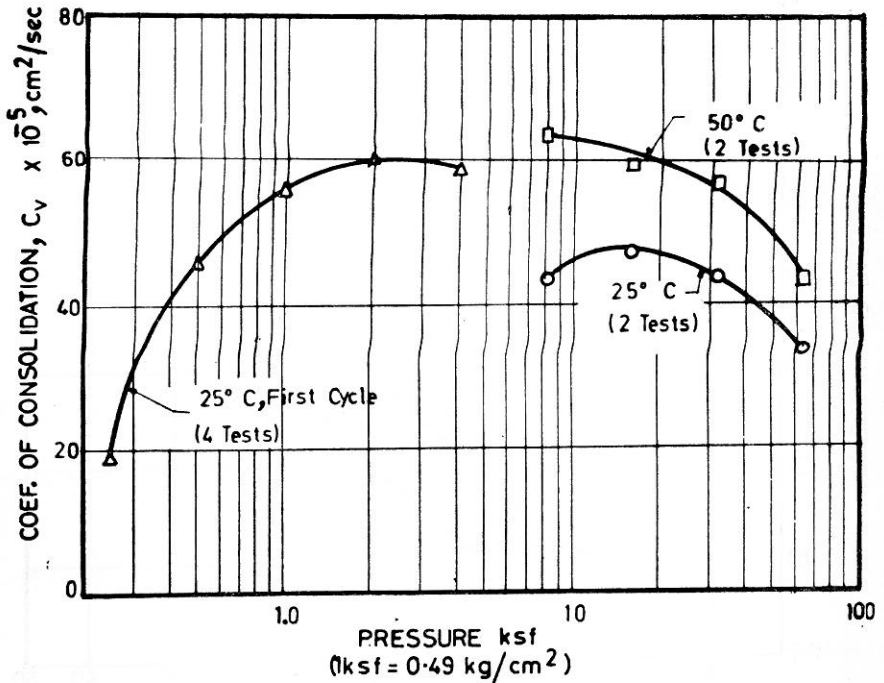


FIGURE 18. Effect of temperature on the coefficient of consolidation

obtained from Figures 17 and 18 is also given in Table 1. The coefficients of consolidation for overconsolidated samples showed too much scatter and are not presented. The ratios represent average value over the range of consolidation pressures used in the experiment. The scatter from the average value is within 5%.

TABLE I

Ratios of the Experimental Coefficients of Permeability and Consolidation at Different Temperatures

State of the sample	$\frac{k_{50}}{k_{25}}$	$\frac{Cv_{50}}{Cv_{25}}$
Normally Consolidated	1.4	1.4
Overconsolidated	1.45	—

The theoretically predicted ratios using Equations (6) and (8) and viscosity and unit weight values from Figure 12 are:

$$\frac{k_{50}}{k_{25}} = 1.63 \times .88 \times 1.0 = 1.44$$

$$\frac{Cv_{50}}{Cv_{25}} = 1.60 \times .88 = 1.44$$

From a comparison of the predicted values with the experimental values (Table 1) it is observed that the predicted values are in good agreement with the experimental results. It can therefore be concluded that at a constant effective void ratio the increase in the coefficients of permeability and consolidation with increase in temperature is principally due to reduction in the viscosity of the pore fluid.

Conclusions

1. Void ratio of the soil tested is affected by the testing temperature. For the same consolidation pressure an increase in the testing temperature results in a lowering of the void ratio.
2. Effective void ratio is not influenced by the testing temperature.
3. The effect of temperature on the coefficients of permeability and consolidation can be predicted accurately by means of eqns. 6 and 8 respectively.
4. At constant effective void ratio, an increase in the testing temperature results in an increase in the coefficients of consolidation and permeability.
5. The coefficient of compressibility is not a function of temperature when testing a normally-consolidated remolded soil.

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