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PERMAFROST

BY KARL TERZAGHI,* MEMBER

(Presented at a joint meeting of the Boston Society of Civil Engineers and the Structural Section B.S.C.E., held on December 12, 1951.)

INTRODUCTION

THE term *permafrost* is an abbreviation for permanently frozen ground. In the sediments and soils of the northern part of North America and Asia the permafrost layer extends from the coast of the Arctic Ocean to a line which coincides very roughly with the 0°C mean annual temperature isotherm. Along the Arctic coast the permafrost layer has a thickness of many hundred feet. With increasing distance from the coast the thickness of the permafrost layer decreases. At its southern fringe the permafrost layer consists of irregular belts and patches of frozen ground separated from each other by unfrozen ground. Islands of permafrost are also common in those parts of the valleys in high mountain chains which have a mean annual temperature of less than 0°C.

From a practical point of view the most important consequences of the existence of the permafrost consist in the volume changes associated with thawing and freezing, and in the loss of strength due to thawing. Engineering operations of any kind change at least to some extent the temperature conditions in the subsoil, whereupon the position of the upper boundary of the permafrost layer also changes. If it moves up the water table rises and the ground surface may heave and if it moves down, important unequal settlement may ensue.

This article provides the engineer with the basic information required for anticipating the effect of his operations on the permafrost

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in the subsoil and for adapting the design of his structures to the anticipated changes.

The subject is divided into four parts. The first part deals with all those physical properties of soils which have a direct or indirect bearing on permafrost. In the second part the effects of freezing and thawing on partly or wholly saturated soils are discussed. The third part covers the origin of permafrost and the factors which govern the processes of permafrost aggradation and degradation. The fourth part deals with the methods of subsoil exploration for engineering purposes in permafrost regions.

On account of the complexity of the phenomena involved, refined mathematical methods can hardly be used to advantage in permafrost engineering. This statement applies particularly to engineering operations along the southern margin of the permafrost layer. Therefore in this paper the physical aspects of the permafrost phenomena have been presented in very elementary terms, involving crude approximations.

Factors Determining Effects of Freezing and Thawing.

The physical effects of freezing and thawing on a partially or completely saturated coarse-grained soil depend on the grainsize characteristics of the soil and on its relative density. The effects of these processes on clay soils depend on the Atterberg limits and the relative consistency. Hence in order to anticipate the effects of freezing and thawing on a given soil it is necessary to determine these properties by means of laboratory tests. If a soil contains air in addition to water, the degree of saturation of the soil must also be ascertained.

Every soil may be encountered in a dry, moist or saturated state. The degree of saturation, S_r, is defined by the ratio

$$S_r = \frac{\text{Volume occupied by the water}}{\text{Total volume of voids}}$$

The quantity of water contained in the soil is given by the natural water content w,

$$w = 100 \times \frac{\text{Weight of water}}{\text{Dry weight of the soil}}$$

Below the water table the soil is completely or almost com-

pletely saturated and all the forces other than gravity which act on the water are inconsequential. Gases escape from the water in an upward direction through the voids of the soil, unless the gas pockets are located beneath layers of silt, clay, or frozen layers of saturated soil.

Above the water table all the forces which act on the water particles including gravity are small compared to the capillary forces. Within a sheet-like zone known as the *capillary fringe* located immediately above the water table, the soil is completely saturated. Above this zone the degree of saturation depends primarily on the grain-size characteristics of the soil. The corresponding water content is known as *soil moisture*. If the degree of saturation in the zone of soil moisture does not exceed a few per cent the water content of the soil consists of isolated water particles centering about the points of contact between grains (*discontinuous soil moisture*). At higher degree of saturation the water particles are interconnected by films or threads (*semi-continuous soil moisture*).

Fig. 1 represents the relation between effective grainsize and

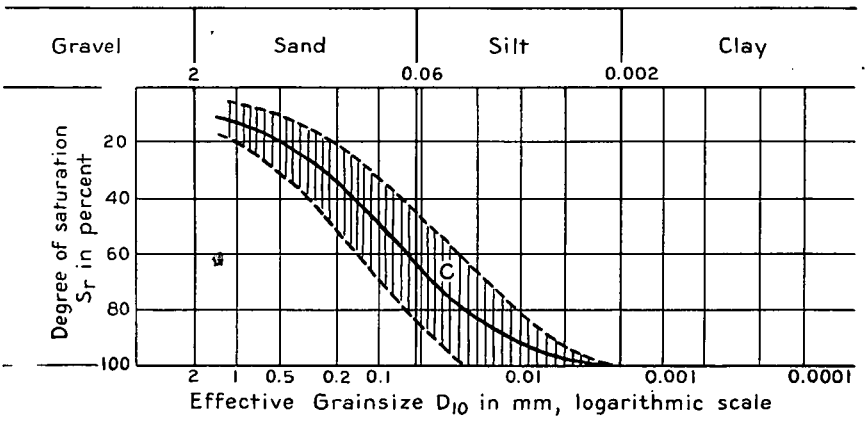

FIG. 1.—APPROXIMATE RELATIONSHIP BETWEEN EFFECTIVE GRAIN SIZE AND DEGREE OF SATURATION IN THE ZONE OF SOIL MOISTURE IN TEMPERATE ZONES WITH MODERATE RAINFALL.

degree of saturation of fairly homogeneous soils located above the capillary fringe in temperate zones with moderate rainfall. The diagram is based on the results of field observations. The important scattering of the values from the average is due to the fact that the

degree of saturation depends not only on the grainsize characteristics of the soil but also on the seasonal variations of the rainfall, type of soil cover, rate of evaporation, organic content of the toplayer and various other factors.

Experience has shown that the soil moisture of relatively homogeneous sedimentary deposits is remarkably independent of the elevation above the capillary fringe except within the uppermost one or two feet. This is probably due to the steady downward movement of the soil moisture associated with the periodic replenishment of the water content from above. However the water content varies with the seasons. In the flood plain deposits of the Connecticut River south of Hartford, Conn., consisting of fine silty sand with a coefficient of permeability of (1 to 20) $\times 10^{-4}$ cm/sec the degree of saturation ranged between 75% and 77% after a dry spell and between 80% and 91% after a prolonged wet spell. (Terzaghi 1942).

If the soil consists of lenses or layers with very different effective grain size or if its structure is erratic the degree of saturation depends not only on grainsize and climatic factors but also to a large extent on the details of stratification. This interdependence is illustrated

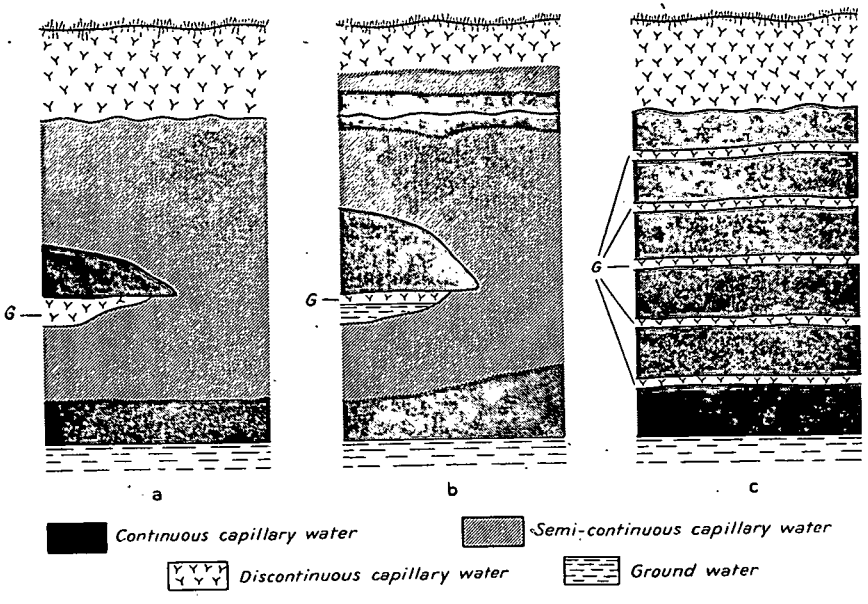

FIG. 2.—DIAGRAMS ILLUSTRATING THE INFLUENCE OF THE SOIL PROFILE ON THE DISTRIBUTION OF SOIL MOISTURE AND ITS SEASONAL VARIATIONS.

by Fig. 2. Fig. 2 *a* and *b* represent a stratum of fine silty sand, containing a lens of gravel. The top layer, with a thickness of 5 ft., contains a network of root holes. The ground surface is covered with grass. Fig. 2 *a* shows the distribution of the soil moisture after a dry spell. During a wet spell, the thickness of the layers of continuous soil moisture increases and a perched water table develops in the lense of gravel. Fig. 2 *c* illustrates the influence of thin but continuous layers of gravel on the saturation conditions in a bed of fine, silty sand. If the annual rainfall changes or if the grass cover is replaced by an impervious toplayer, both the average degree of saturation and the saturation pattern will change.

If a mass of soil in a state of partial saturation like that shown in Fig. 2 is invaded by freezing temperatures, the water table rises because all the layers which are completely saturated become impermeable. Nevertheless it is inevitable that large amounts of air are trapped in the soil located beneath saturated and frozen layers and lenses.

Strength of Thawed and Frozen Soils.

Under the influence of concentrated loads both thawed and frozen soils commonly fail by shear. According to Coulomb's classical concept the resistance s per unit of area against failure by shear along a section through any material is

$$s = c + p \tan \phi \qquad (1)$$

wherein p is the unit pressure on the surface of sliding, ϕ the angle of internal friction and c the cohesion (shearing resistance for $p = 0$). The validity of this equation is subject to various limitations (see for instance Terzaghi and Peck 1948, pp. 78 to 93), but in connection with the following discussions these limitations can be disregarded.

If the shearing resistance of a material is determined by Equation (1) the unconfined compressive strength q_u or the unit load under which a cylindrical specimen fails is equal to

$$q_u = 2c \tan \left(45 + \frac{\phi}{2}\right) \qquad (2)$$

For coarse-grained soils like clean sand or gravel in a dry or completely saturated but unfrozen state, $c = 0$ and $q_u = 0$. In other words, these soils owe their capacity to sustain concentrated loads

exclusively to internal friction. The angle of internal friction of these soils commonly lies between 35° and 45° and its value depends on the relative density and the shape of the grains. Fine and very fine sands in a moist state have a slight apparent cohesion which disappears as soon as the sand is submerged.

The compressive strength q_u of a saturated silt or clay depends on its geologic history and on the physical and chemical properties of the clay constituents. The q_u value determines the consistency of the clay, which is commonly designated by one of the following terms:

Consistency	Very soft	Soft	Medium	Stiff	Very stiff	Extremely stiff
q_u in kg/sq cm	<0.25	0.25-0.5	0.5-1.0	1.0-2.0	2.0-4.0	>4.0

If soil is exposed to freezing temperatures, the free water contained in the voids of the soil freezes whereupon the ice interconnects the soil particles. Therefore the strength of the soil increases. The unconfined compressive strength q'_u of the frozen soil depends on the unconfined compressive strength q_i of the ice, on the degree of saturation S_r, and the angle of internal friction ϕ in Equation (2).

The unconfined compressive strength q_i of ice depends on the temperature of the ice, the structure of the ice and the rate of loading. According to Brown the q_i value of river ice increased from 21.0 kg/sq cm at —2.2°C to 62.0 kg/sq cm at —16°C (Dorsey 1940, p. 449). E. Bucher obtained for ice produced by the freezing of saturated snow at —3.5°C an average value of 34 kg/sq cm (Haefeli 1938, p. 140). Russian investigators reported that the q_i value for the top layer of a sheet of river ice increased from 20.7 at —2°C to 38.4 kg/sq cm at —23°C whereas that of the middle part of the layer increased from 35.8 to 76.0 kg/sq cm, as a result of the same temperature change. They also reported that the compressive strength decreased with increasing rate of loading. By increasing the rate of loading from 20 to 50 kg per sq cm and minute the q_i value was reduced from 60 to 24 kg per sq cm (Muller 1947, p. 36).

Another important mechanical property of ice is its capacity to creep. If a block of ice is permanently kept under a load which is considerably smaller than the failure load ($q_i \times$ loaded area), the block gradually flattens out. At a load of less than about 2 kg per sq cm on and at a temperature of about —5°C this slow deformation or "creep" was found to be imperceptible. However, under loads of

more than 2 kg per sq cm the rate of creep increased rapidly with increasing load (Haefeli 1938, p. 139). The capacity of the ice to creep under relatively very low deviator stresses is responsible for the movement of glaciers.

Because frozen soil owes its cohesion chiefly or entirely to that of the ice, the behavior of a frozen soil under stress must have at least some features in common with the behavior of pure ice under similar stress conditions. Since the angle of internal friction of ice is equal to zero, equation (2) requires that the cohesion c of ice be equal to $q_i/2$.

If the degree of saturation of a soil is smaller than 100%, the freezing of the soil moisture imparts to the soil the character of a mild sandstone. The grains of this sandstone-like material are interconnected by minute patches of ice. On the other hand, if a saturated soil freezes, it turns into a block of ice, reinforced by a skeleton of solid soil particles. The strength q'_u of such a soil is likely to be approximately equal to

$$q'_u = q_i \tan\left(45 + \frac{\phi}{2}\right) \tag{3}$$

wherein q_i is the unconfined compressive strength of the ice and ϕ the angle of internal friction of the soil. Equation (3) is identical with the equation for the confined compressive strength of crystalline rocks such as marble.

The q'_u values reported by Russian investigators range between 22 kg/sq cm at —0.5°C and 30 kg/sq cm at —2°C for saturated sand and between 5 kg/sq cm at —0.5°C and 23 kg/sq cm at —2°C for saturated silt (Muller 1947, p. 40). Conclusive information regarding the relation between temperature, relative density and q'_u value for the principal types of soils in a saturated state and the creep of frozen soil under moderate loads is not yet available.

Thermal Properties of Soils.

For a given surface-temperature regime the position of the boundaries of the active and the permafrost zone with reference to the ground surface depends on the thermal conductivity and the heat capacity of the strata located within and below the zones of ground frost. In the following text all the numerical values referring to the thermal properties of soils are given in cm gm sec (cgs) units.

The *thermal conductivity,* k_h, in cm gm sec (cgs) units, is the quantity of heat which flows through a layer, 1 cm thick, per unit of time and square centimeter of the layer, at a thermal gradient of 1°C per centimeter. The *heat capacity,* c_h, in cgs units is equal to the quantity of heat per gram of the weight of a body, required to raise the temperature of the body by 1°C.

The essential relations between the thermal conductivity of a soil, k_h, in cal cm^{-1} sec^{-1} $(°C)^{-1}$, the porosity, the dry density in lbs per cu ft and the degree of saturation are graphically represented in Fig. 3. The diagram was prepared chiefly on the basis of the data contained in Kerstens final report entitled "Laboratory Research for the Determination of the Thermal Properties of Soils", St. Paul District, Corps of Engineers, June 1949. The abscissas represent the porosity n and the dry weight w_d respectively. The dry weight has been computed on the assumption that the average density of the solid soil particles is 2.70. On this assumption

$$w_d = (1 - n) \times 2.7 \text{ gms per cu cm} \qquad (4)$$

The ordinates represent the thermal conductivity.

As n approaches 100% the k_h value of a saturated unfrozen porous substance approaches the k_h value for water which is roughly $1'5 \times 10^{-3}$ cgs units and that of a frozen saturated porous substance the k_h value for ice which is approximately 5.3×10^{-3} cgs. The k_h value of a very porous, saturated frozen material can hardly be smaller than that of ice, and the k_h value of a very porous material with air-filled voids cannot be smaller than the k_h value of the most effective insulating materials such as dry asbestos or cotton, which ranges between 0.12 and 0.22×10^{-3} cgs. On the other hand, as the porosity n of a soil approaches a value of zero, the k_h value of the soil must approach the average k_h value of its mineral constituents. These relations determine the position of the horizontal tangents or asymptotes of the curves shown in Fig. 3. Since the thermal conductivity of ice is much higher than that of water, the thermal conductivity of frozen soils is higher than that of unfrozen ones and the difference between the two values must increase from zero for $n = 0$ to about 38×10^{-3} cgs for $n = 100\%$.

The thermal conductivity of the soil forming minerals ranges between a maximum of more than 20×10^{-3} cgs for quartz and less than 10×10^{-3} cgs for micaceous minerals. (Birch et al., 1942, p.

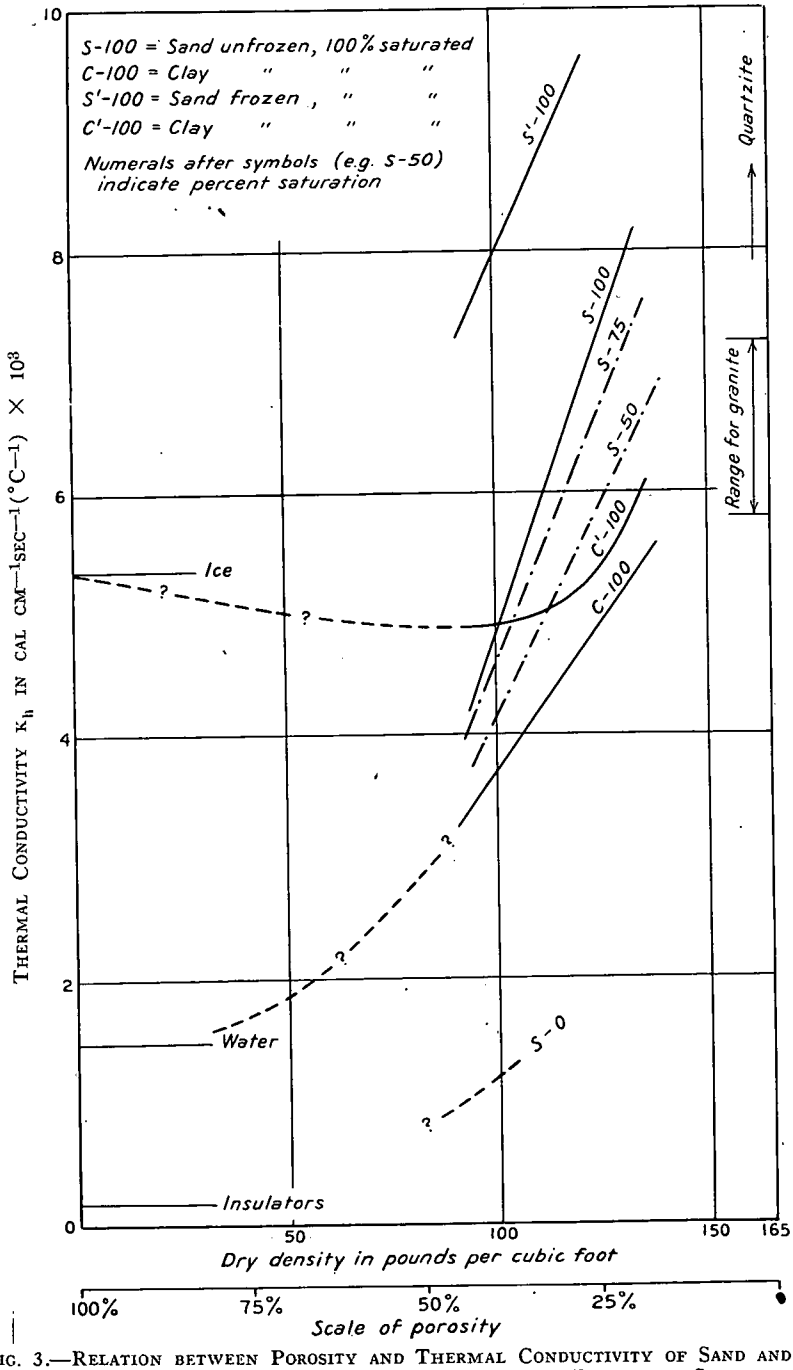

FIG. 3.—RELATION BETWEEN POROSITY AND THERMAL CONDUCTIVITY OF SAND AND CLAY IN AN UNFROZEN AND IN A FROZEN STATE, AT DIFFERENT DEGREES OF SATURATION.

250). The coarsest grainsize fractions are commonly dominated by quartz and the finest ones by micaceous minerals, including the clay minerals. This may be one of the reasons why the curve S_{100} representing the relation between n and k_h for saturated sand is located high above the corresponding curve C_{100} for clay soils.

In the field, below the water table, all soils are completely saturated except in pockets and layers containing trapped air. For saturated, unfrozen sand, the relation between k_h and the porosity n is indicated in Fig. 3 by the curve S-100 and for saturated frozen sand by S′-100. The corresponding curves for clay are marked C-100 and C′-100.

The factors which determine the degree of saturation of a soil located above the water table have been discussed in an earlier section of this paper. In a general way the average degree of saturation of soils located above the water table increases with decreasing effective grainsize D_{10} as shown in Fig. 1. With decreasing degree of saturation S_r the thermal conductivity k_h of a soil decreases. In Figure 3 the relation between k_h and the porosity n of unfrozen sand at a degree of saturation of 75%, 50%, and 0%, is indicated by the dotted lines S-75, S-50 and S-0. It can be seen that the effect of a reduction of the degree of saturation from 100% to 50% on k_h is unimportant compared to that of a reduction from 50% to zero.

The heat capacity, c_h, of soils, like their thermal conductivity, k_h, depends on the average mineral composition of the soil, the porosity and the degree of saturation. However, the ratio between the extreme values of the heat capacity of soils is very much smaller than the ratio between the extreme thermal conductivities. The heat capacity of all the mineral constituents of soils is close to 0.19, that of ice is about 0.43 and that of water 1.0. The heat capacity c_h of a soil with a water content w in grams per gram of the dry weight is roughly equal to

$$c_h = \frac{0.19 + w}{1 + w} \text{ cal gm}^{-1}(^\circ\text{C})^{-1} \tag{5}$$

If the soil freezes the heat capacity decreases to the value

$$c'_h = \frac{0.19 + 0.43\,w}{1 + w} \tag{6}$$

Both the thermal conductivity, k_h, and the heat capacity, c_h, de-

pend to some extent on the temperature, but the effect of the temperature is not important enough to require consideration in connection with permafrost problems. The knowledge of k_h and c_h forms the only reliable basis for a rational explanation of frost phenomena such as permafrost and it also provides the basis for estimating the rate of permafrost growth or degradation due to a change in the thermal regime of the subsoil.

According to the laws of thermodynamics the rate at which the temperature of a body with given dimensions and with a unit weight w_t (solid and water combined) adapts itself to a change in the temperature of the surrounding medium is determined by the ratio

$$a = \frac{k_h}{c_h \times w_t} \tag{7}$$

known as *diffusivity*. In the cm gm sec (cgs) it has the dimension cm^2sec^{-1}. The rôle of diffusivity in thermodynamics corresponds to that of the *coefficient of consolidation* c_v (cm^2sec^{-1}) in soil mechanics, which determines the rate at which the water content of a saturated clay stratum adapts itself to an increase of the load on the stratum. The following numerical values of the diffusivity in the cgs system of different materials illustrate the range of this value.

Copper	1,133	$\times 10^{-3}$ cm^2sec^{-1}	Dense saturated sand	8	$\pm \times 10^{-3}$ cm^2sec^{-1}
Iron	173	$\times 10^{-3}$ cm^2sec^{-1}	Soft saturated clay	4	$\pm \times 10^{-3}$ cm^2sec^{-1}
Quartzite	45	$\times 10^{-3}$ cm^2sec^{-1}	Fresh snow	3.3	$\times 10^{-3}$ cm^2sec^{-1}
Granite	15	$\times 10^{-3}$ cm^2sec^{-1}	Dry soil	2.5	$\pm \times 10^{-3}$ cm^2sec^{-1}
Ice	11.2	$\times 10^{-3}$ cm^2sec^{-1}	Water	1.4	$\times 10^{-3}$ cm^2sec^{-1}

As indicated by this list, the diffusivity of ice is very much higher than that of water. Consequently the diffusivity of frozen soil is considerably higher than that of the same soil in a thawed condition. On account of this fact the average temperature of a body of saturated frozen soil increases much more rapidly than that of a body of unfrozen soil with equal dimensions, at equal difference between the initial temperature of the body and that of the surrounding medium. However, if the temperature of the medium surrounding the body of frozen soil is above the freezing point, involving the thaw-

ing of the frozen soil, the increase of the temperature of the frozen body is delayed by the latent heat of fusion of the ice. The latent heat of fusion is the amount of heat, in thermal units per unit of weight of water, which is consumed while the ice melts. In the cgs system it amounts to about 80 calories per gram of water. The same amount of heat is liberated when one gram of water is frozen.

Freezing and Thawing

Freezing of Water.

If the temperature of free water drops below the freezing point the water turns into ice and its volume increases. Both the freezing point and the coefficient of volume expansion depend on the pressure in the water. At a pressure of one atmosphere the freezing point is 0°C, at 600 atm. about —5°C and at 1,100 atm. —10°C. The corresponding coefficient of volume expansion increases from 0.09 at one atmosphere to 0.102 at 600 atm. and 0.112 at 1,100 atm. (Birch et al., 1942).

Ice Formation in Soils.

If the temperature of a mass of clean sand or gravel in a moist or saturated state is lowered below the freezing point the water contained in the voids of the mass freezes in situ. The freezing is associated with a volume expansion of the water by almost ten per cent. However, this expansion does not necessarily lead to a ten per cent increase in the volume of the voids of a saturated sand because part of the liquid water may be expelled while freezing proceeds.

If thick layers or large pockets of ice are encountered in strata consisting of fairly clean sand or gravel, it is almost certain that the ice was formed by the freezing of a pool, or was moved to its present location and subsequently buried.

If saturated silt or silty sand is exposed to freezing temperatures, the effects of freezing depend on the rate at which the temperature is lowered. Rapid cooling of a saturated specimen in the laboratory causes the water to freeze in situ as it does in sand, but if the temperature is lowered gradually, the major part of the frozen water accumulates in the form of lawers of clear ice oriented parallel to the surface exposed to the freezing temperature. As a consequence the frozen soil consists of a series of layers of frozen soil separated from each other by layers of clear ice.

Under natural conditions ice layers with a thickness of several inches are also formed in silty soils located within the zone of seasonal frost penetration. In frozen silt located in a permafrost region beneath the zone of seasonal temperature variations ice was found to occur also in the form of pockets or lenses with a height of many feet.

Bodies of clear ice could not possibly be formed in a body of soil unless the water migrates through the voids of the soil towards the centers of freezing. The water may come out of the soil which freezes or it may be drawn out of an aquifer located below the zone of freezing. These possibilities are illustrated by Fig. 4.

Figure 4 represents three cylindrical specimens of a fine satu-

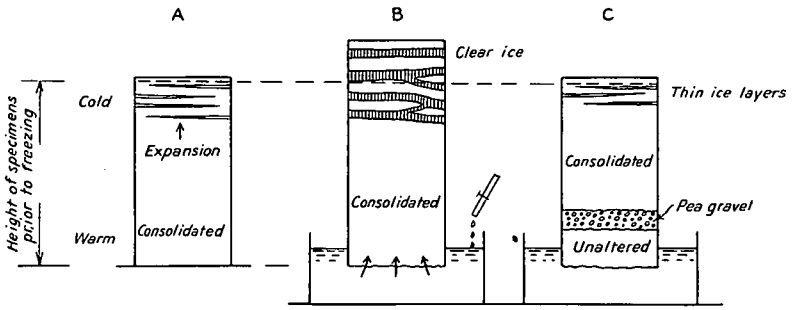

FIG. 4.—ICE LAYER FORMATION IN CLOSED SYSTEM A AND OPEN SYSTEM B. LAYER OF PEA GRAVEL IN SPECIMEN C CHANGES UPPER PART OF SPECIMEN INTO CLOSED SYSTEM.

rated silt. Specimen *A* rests on a solid base, whereas the lower ends of specimens *B* and *C* are immersed in water. The temperature of the upper end of each specimen is kept below the freezing point. In *A* the water which enters the ice layers is drawn out of the lower part of the specimen. As a consequence, the lower part consolidates in the same manner as if the water were pulled by capillarity toward a surface of evaporation at the upper end. The growth of the ice layers probably continues until the water content of the lower part is reduced to the shrinkage limit, provided the temperature is sufficiently low. Since all the water entering the ice layers comes from within the specimen, the sample is referred to as a *closed system*. The volume increase associated with the freezing of a closed system does not exceed the volume increase of the water contained in the system. It commonly ranges between 3 and 5 per cent of the total volume.

In *B* the water required for the initial growth of the ice layers is also drawn out of the specimen, whereupon the lower part of the sample consolidates. However, as the consolidation progresses, more and more water is drawn from the pool of free water located below the specimen. Finally, both the rate of flow toward the zone of freezing and the water content of the unfrozen zone through which the water percolates become constant. Such a sample constitutes an *open system*. Experience in permafrost regions shows that the total thickness of the ice lenses contained in such a sytem can increase to many feet.

The open system represented by sample *B* can be transformed into a closed system by inserting a layer of coarse-grained material between the zone of freezing temperature and the water table, as shown by *C*. Since the water cannot rise by capillary through the coarse layer, the upper part of the sample represented in *C* constitutes a closed system. The lower part of the closed system is subject to drainage by frost action.

During the last decades the formation of ice layers in silt and silty soils has received increasing attention on account of its detrimental effects on highways and airports. An annotated bibliography on frost action in soils, covering 283 items, has been published by the Highway Research Board (1948). It has been found that ice layers do not develop as a result of freezing, unless the soil contains at least several per cent of soil particles passing the 200-mesh sieve. However, the effect of frost depends also on the degree of uniformity, the relative density and the type of stratification. The influence of these factors on ice segregation is not yet completely known.

Ice layers have also been obtained by slowly cooling cylindrical clay specimens in the laboratory. Owing to the low permeability of clay the migration of water towards the centers of freezing proceeds very slowly.

Information regarding the effect of freezing temperatures on clay in the field is still inadequate. An example for what appears to be anomalous behavior of clay at low temperatures was described by Mr. L. E. Wood[1] as follows:

[1]Personal communication of March 14, 1951. Boring records in the files of the Alaska District of the Corps of Engineers, U. S. Army, indicate that the base of the clay stratum was located at a depth of about 60 ft. The clay was described as "soft plastic" and it contained numerous, thin ice lenses and layers. The ground temperature was measured by means of thermocouples. It increased from —11.5°C at a depth of 6 ft to —9.0°C at 54 ft. The water content of a somewhat firmer clay encountered in an adjacent testboring ranged

"At Barter Island (on the coast of the Arctic Ocean, about 70 miles west of the boundary between Alaska and Canada), in depths from 25 to 35 ft, under a layer of gravel, the clay encountered consisted of unfrozen, soft plastic material, separated by a layer of ice one to three inches thick. Soil temperature was about —15°C. Within several hours after the core was removed in an air temperature slightly warmer than —15°C, the clay turned hard, definitely frozen. Drillers said that the core visibly expanded when removing it from the sampling sleeve".

Frost Effects on Level Ground.

If the water freezes in a void of a soil under a moderate overburden pressure it acts like a jack which pushes the soil grains apart and thus increases the void space. If the freezing occurs in the voids of a coarse-grained soil such as sand or gravel or in a closed system, the corresponding rise of the ground surface does not exceed about

$$h = 0.1\ nH \tag{8}$$

wherein n is the average porosity and H the thickness of the layer subject to freezing. On the other hand, if the freezing takes place in an open system susceptible to the formation of ice layers, h can assume values which are many times greater than those determined by Equation (8). The corresponding rise of the ground surface is known as *frost heave*. If a structure is located above the seat of ice formation, it goes up together with the surface of the ground.

Conclusive information regarding the maximum load which can be lifted by a growing ice layer is not yet available. The greatest pressure which can possibly be exerted by freezing water is about 2,000 tons per sq ft, because under higher pressures the water crystallizes into ice VI which does not involve a volume increase.

The surface settlements due to the thawing of frozen layers of soil will be discussed in the last part of this paper.

In Arctic regions the processes of alternate freezing and thawing are sometimes associated with the migration of soil particles in radial directions from points scattered at random over the ground surface. The migration is associated with segregation according to particle size and the terrain affected by the process is known as

between 24.3 and 34.6 per cent. Practically all the soil particles passed the 200 mesh screen. Almost fifty per cent were smaller than 0.005 mm and the percentage of particles smaller than 0.002 mm ranged between 25 and 33 per cent.

polygonal ground. The most conspicuous manifestations of this process are the *stone rings,* which develop in silty soils containing large rock fragments. The rings on Kingsbay in Spitzbergen have an inner diameter of about five feet. They consist of buried, ring-shaped walls with a height of about two feet and a thickness up to two feet. The walls are composed of stones which have been expelled from the space surrounded by the wall. The mechanism of the stone-ring formation is not yet satisfactorily explained.

Frost Creep on Slopes and Solifluction.

On slopes the soil movements due to freezing and thawing take place primarily in a down hill direction. Under given climatic conditions the character and the rate of the movement depends on the slope angle and the grainsize characteristics of the material.

In coarse-grained soils the slope movements appear to be chiefly due to frost wedging and to the formation of needle ice. The growing ice needles displace the rock particles in a direction at right angles to the slope and when the ice goes out of the ground, the particles descend vertically downward. The soil movements due to ice action combine with those due to thermal expansion and contraction. The resulting creep proceeds very slowly. Yet the ultimate effect can be very striking as shown by some *"rock glaciers"* which were produced entirely by frost and temperature creep.

On slopes underlaid by silty material, soil movements appear to be chiefly due to the formation of pockets of trapped water produced by the melting of ice lenses. The soil located above the water pockets disintegrates and the soil movement assumes the character of a mud-flow. This process is called *solifluction.* It is sometimes associated with the segregation of the soil according to particle size, leading to the formation of *stone strips* following lines of maximum surface gradient or *stone-banked terraces* with stone accumulations along contour lines.

The importance of slope movements due to freezing and thawing increase from the southern boundary of seasonal frost towards the southern boundary of the permafrost region. Between the margin of the permafrost zone and a line which follows very roughly the 5°C mean annual temperature isotherm, solifluction is one of the most important causes of slope movements. At the foot of the slopes subject to solifluction the soil strata may be intricately folded and faulted to a depth of more than ten feet.

During the Pleistocene glaciation the southern boundary of the permafrost region was located far beyond its present position. As a consequence the southern boundary of the area affected by solifluctions is by no means identical with that of the area where solifluction still takes place. In many regions it is even located south of the line to which the ice advanced during the Pleistocene period.

Physics of Permafrost

Definitions.

The term *permafrost* refers to ground which remains frozen throughout the year. Increase of the thickness of the permafrost layer is commonly referred to as *permafrost aggradation* and a shrinkage of the body of permafrost is called *permafrost degradation.*

The region located between the ground surface and the lower boundary of seasonal freezing and thawing is called the *active layer.* Under stationary conditions the base of the active layer is identical with the top surface of the permafrost. However, in zones of progressive or recent permafrost degradation the two zones can be separated from each other by a thick layer of ground which remains unfrozen throughout the year.

Prerequisites for Permafrost.

The only essential difference between the active layer and the permafrost resides in the length of time during which the frozen state persists. In the active zone the frozen condition prevails over a period of less than one year whereas in the permafrost areas located along the coast of the Arctic Ocean the ground may have been frozen for more than one million years.

Permafrost starts to grow wherever and whenever the temperature of the ground ceases for many consecutive years to reach a temperature above the freezing point during the summer months. At a short distance below the base of the active layer the ground temperature remains throughout the year equal to the mean annual air temperature at the surface. Therefore the southern boundary of the permafrost zone coincides very roughly with the 0°C mean annual temperature isotherm. The local deviations between these two lines are due to secular variations in the mean annual temperature combined with the lag between a change in the mean temperature and the corresponding change of the thickness of the permafrost layer. The

lag is caused by the absorption of heat during the process of melting and the liberation of heat during freezing.

In the southern hemisphere there is no continental territory, other than ice-covered Antarctica and the highest parts of the Andes, with a mean annual temperature below 0°C. Hence south of the equator and beyond the confines of Antarctica, permafrost is a local phenomenon. On the other hand, in the northern hemisphere vast continental areas, including the northernmost parts of North America and Asia are located within the 0°C isotherm which coincides very roughly with latitude 55°N. Therefore practically all land surfaces on these continents north of about 55°N are underlain by sheets of permafrost.

The base of the permafrost layer is exposed to the heat which radiates from the interior of the earth. The thickness of the permafrost layer is determined by the geothermal gradient and by the mean annual surface temperature.

Geothermal Gradient.

Volcanic phenomena and other geological evidence have led to the conclusion that the temperature of the interior of the earth is high above the melting point of rocks under low pressure. The rapid increase of the temperature with depth is disclosed by the high temperatures which prevail in deep mines. Therefore the interior of the earth represents a heat reservoir. Since the temperature at the surface is relatively very low there is a steady flow of heat from the interior of the earth towards the surface, involving a continuous increase of the temperature with depth. The temperature increase per unit of depth is called the *geothermal gradient* i_g(°C $\times$ cm^{-1} in the cgs system). If A is an area oriented at right angles to the direction of the flow of heat, the quantity of heat Q_h which flows through this area per unit of time is equal to

$$Q_h = Ak_h i_g \tag{9}$$

wherein k_h is the thermal conductivity (Fig. 3). This equation corresponds to the equation which determines the rate of flow of water through the voids of a fine-grained soil.

Since k_h in Equation (9) is different for different layers of the earth's crust, the geothermal gradient varies along the lines of flow of heat. It is greatest in strata with the lowest heat conductivity

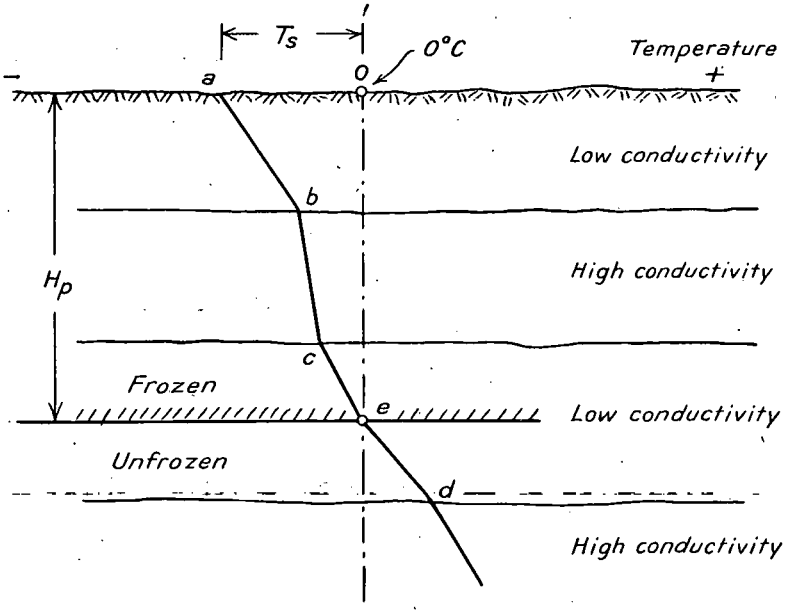

FIG. 5.—INFLUENCE OF VARIATIONS OF HEAT CONDUCTIVITY WITH DEPTH ON GEOTHERMAL GRADIENT (SLOPE OF LINE a b c e d WITH REFERENCE TO VERTICAL LINE O e)

and vice versa as shown in Fig. 5. The figure represents a vertical section through the top layer of the ground in a fictitious region in which the temperature T_s of the atmosphere is constant and below the freezing point. On account of the flow of heat from the interior of the earth the temperature of the ground increases with depth as indicated by the broken line ad. Since the thermal conductivity of the rocks which form the earth's crust varies with depth, the geothermal gradient also varies. Further variations are produced by sources of heat, such as bodies of hot igneous rock, located within the earth crust. At any depth the geothermal gradient is equal to the slope of the geothermal line ad with reference to the vertical. To a depth H_p the temperature of the ground remains permanently below the freezing point. Hence to depth H_p the ground is permanently frozen.

Experience has shown that the rate of flow of heat, Q_h/A (Equation 9) varies from region to region. It may range between 2×10^{-6} cal sec^{-1} cm^{-2} in volcanic regions and less than 1×10^{-6} cal sec^{-1} cm^{-2} in nonvolcanic ones. Therefore the geothermal gradient depends not only on the thermal conductivity of the rocks or sediments

located beneath the ground surface but also on the rate of heat supply. During the construction of the Mt. Cenis tunnel in Switzerland it was found that the temperature increased about 1°C for every 50 meters of depth. In some regions of South Africa the geothermal gradient is even smaller than 1°C per 100 meters. On the other hand, in coal-bearing formations, where the chemical changes in the coal beds represent a supplementary source of heat, the temperature may increase by 1°C for every 12 m of depth. Finally A. E. Benfield (1949) has shown that the temperature gradient can be significantly affected by geological surface processes. If a region is subject to denudation, the geothermal gradient increases because of the heat release associated with the cooling of the rocks or sediments which were originally located at greater depth. On the other hand, if aggradation takes place, the gradient is lowered because of the heat absorbed in the sediments which have been deposited at surface temperature. Hence the geothermal gradient is by no means constant. At an average, the temperature increase by 1°C for every 30 m of depth, corresponding to an average thermal gradient of 0.033 $(^\circ C)m^{-1}$.

Relations between Surface and Ground Temperatures.

At the surface, the ground temperature is slightly lower than the temperature of the adjacent air (shade temperature). This is due to the heat which is absorbed in the process of surface evaporation. Therefore the mean annual ground temperature at the surface is lower than the mean annual air temperature. The difference is about 0.5°C, but in arid regions it may be as high as 5°C. Furthermore the temperature of the atmosphere undergoes diurnal, annual and secular variations involving similar variations of the ground temperature. To demonstrate the fundamental relations which determine the effects of such variations let us assume that the temperature of the atmosphere referred to in Fig. 6 is suddenly increased from T_0 to $T_0 + \Delta T_0$. To simplify the investigation it is further assumed that the initial temperature of the ground is uniform and equal to T_0.

The effects of the temperature increase are shown by the plain curve *b c* in Fig. 6. As soon as the temperature of the atmosphere rises, the temperature of the ground starts to increase and the heat penetrates the ground from the surface in a downward direction.

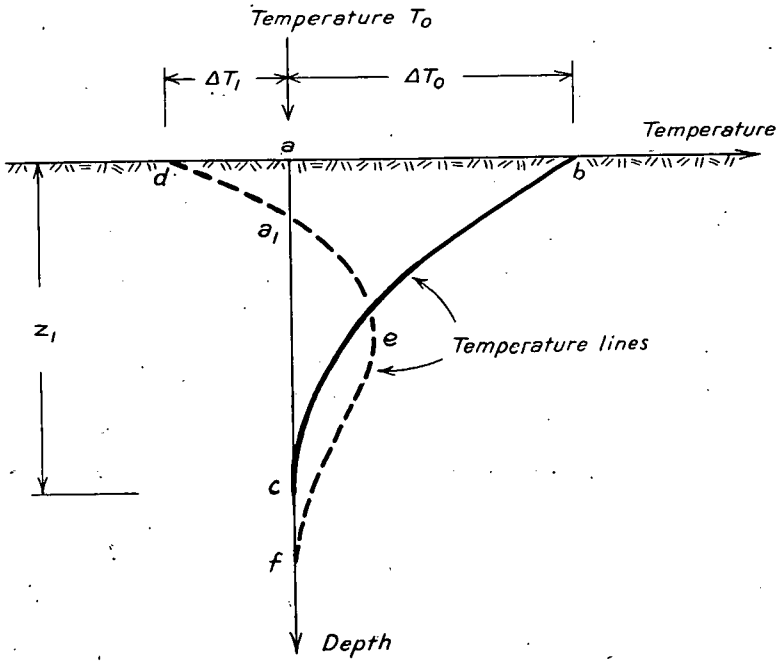

FIG. 6.—DIAGRAM ILLUSTRATING DISTRIBUTION OF TEMPERATURE IN SEMI-INFINITE MASS WITH UNIFORM INITIAL TEMPERATURE T_0 AT TIME t_1 AFTER SURFACE TEMPERATURE WAS INCREASED BY ΔT_0 (PLAIN CURVE b c f), AND AT TIME t_2 AFTER THE SURFACE TEMPERATURE WAS REDUCED FROM $T_0 + \Delta T_0$ TO $T_0 - \Delta T_1$.

According to the theory of the nonstationary flow of heat through solids the distribution of the temperature in homogeneous ground is roughly parabolic at any time t_1 after the surface temperature has been raised from T_0 to $T_0 + \Delta T_0$.

In the simplified theory of the flow of heat into the ground, given in Appendix *A*, it is assumed that the temperature distribution is strictly parabolic at any time t_1 after the surface temperature has increased and that the temperature of the ground below point *c* in Fig. 6 is unchanged. On these assumptions the time t_1 at which point *c* arrives at depth z_1 is

$$t_1 = \frac{z_1^2}{12a} \tag{10}$$

wherein a is the heat diffusivity, Equation (7). If the ground con-

sists of quartzite, $a = 45 \times 10^{-3}$ cm^{2}sec^{-1}, time t_1 at which point *c* arrives at a depth of $z_1 = 50$ feet $= 1{,}520$ cm is

$$t_1 = \frac{1{,}520^2}{12 \times 45 \times 10^{-3}} = 43 \times 10^5 \text{ sec} = 50 \text{ days.}$$

If the ground consists of granite, $a = 15 \times 10^{-3}$ cm^{2}sec^{-1}, t_1 is equal to 150 days. For soft clay, $a = 4 \times 10^{-3}$ cm^2 sec^{-1}, $t_1 = 562$ days. These figures show the decisive influence of the diffusivity on the rate of propagation of temperature in the ground.

The depth z_1 within which the temperature of the ground has perceptibly increased during a given time t_1 is

$$z_1 = \sqrt{12 a t_1} \tag{11}$$

If, at some time t_1, the temperature of the atmosphere is suddenly reduced from $T_0 + \Delta T_0$ to $T_0 - \Delta T_1$, the cooling, like the preceding heating, proceeds from the surface of the ground in a downward direction. In the earlier stages of the process of cooling the distribution of the temperature will resemble that indicated by the dash line *d e f*, Fig. 6. The position of this curve, with reference to the vertical line a f (initial temperature line) shows that the temperature of a layer located between the elevations of points a_1 and *f* will be higher than the temperature in the ground immediately above and below the layer.

In connection with periodic changes of the surface temperature, such as those associated with the sequence of day and night, one-half of the difference between the minimum and the maximum temperature is called the *amplitude* of the temperature wave and the lapse of time between two successive temperature maxima is the *period* of the wave. The theory of periodic flow of heat shows that every temperature wave on the surface produces a similar temperature variation in the ground. However, with increasing depth, the amplitude of the temperature wave decreases and the time lag between the temperature maximum at depth with reference to that at the surface increases. The level below which the amplitude of the temperature variations becomes imperceptible is the *level of zero amplitude.*

The greater the period of the surface wave the greater is the depth to which the temperature of the ground varies perceptibly. Figure 7 shows the diurnal variations of the surface temperature.

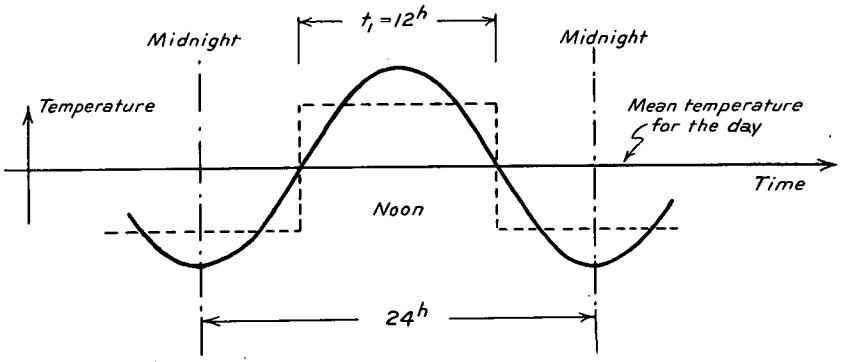

FIG. 7.—DIAGRAM ILLUSTRATING DAILY VARIATIONS OF TEMPERATURE AT GROUND SURFACE (PLAIN SINUSOIDAL CURVE).

The period of these variatians is 24 hours. For 12 hours the temperature is above and for 12 hours below the average for the day. A rough estimate of the position of the level of zero amplitude of the temperature variations produced by the periodic change of the surface temperature can be made on the assumption that the change of the surface temperature occurs abruptly as indicated in Fig. 7 by a dash line. On this assumption the time $t_1 = 12$ hours corresponds to t_1 in equation (11). According to this equation the depth H_0 to which a sudden rise of the surface temperature increases the ground temperature in 12 hours is

$$H_0 \text{ (cm)} = \sqrt{12a \times 12 \times 3{,}600} = 720\sqrt{a} \qquad (12)$$

wherein a is the average diffusivity of the ground in cm^2sec^{-1}. As soon as the cooling has proceeded to depth H_0 the temperature again becomes higher than the average. Therefore H_0 represents the depth to which the diurnal variations of the surface temperature produce perceptible changes of the sub-surface temperature. In order to compute this depth for wet sand, we introduce into the preceding equation $a = 8 \times 10^{-3}$ cm^2sec^{-1} and obtain

$$H_0 = 720\sqrt{8 \times 10^{-3}} = 62 \text{ cm}$$

or about 2 feet. Hence the diurnal temperature variations do not extend beyond a depth of a few feet. For the seasonal variations, with a period of 365 days ($t_1 = 365/2 = 182$ days) we obtain values of H_0 of the order of magnitude of 70 feet. Finally, the effects of secular cold waves like that which produced the last period of glacia-

tion extend to a depth of many thousand feet. The rigorous method for computing changes in the ground temperature due to periodic changes of the surface temperature can be found in any textbook on heat conduction (see for instance Ingersoll, Zobel and Ingersoll, 1948, pp. 45-57 or H. S. Carslaw and J. C. Jaeger, 1947, pp. 43-51).

The effect of the seasonal variations of the surface temperature on ground frost conditions is illustrated by Fig. 8 *a* and *b*. Figure 8 *a* represents the temperature conditions in the subsoil of a region with a temperate climate and Fig. 8 *b* refers to a similar region with an

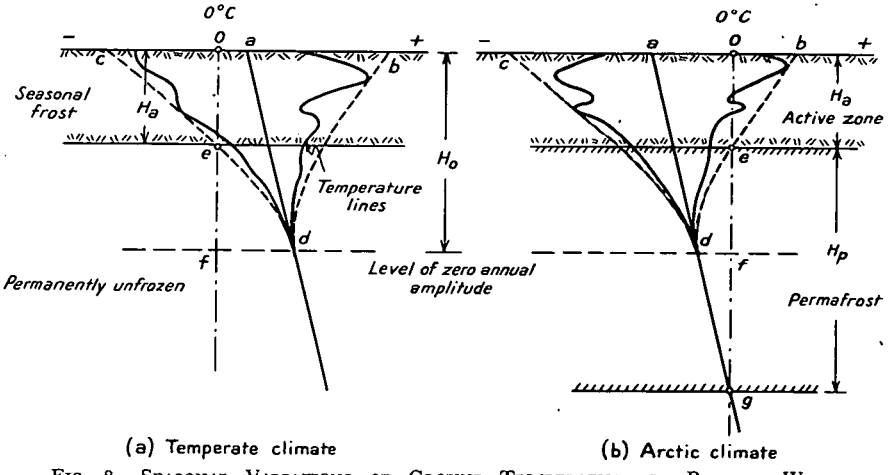

FIG. 8.—SEASONAL VARIATIONS OF GROUND TEMPERATURE IN REGIONS WITH (a) TEMPERATE AND (b) ARCTIC CLIMATE.

arctic climate. The mean annual temperature *aO* in the temperate region is above the freezing point and that of the arctic region is well below this point. In both regions the level of zero annual amplitude is assumed to be located at the same depth. Within this depth the temperature conditions change continuously. On account of the diurnal variations of the surface temperature and the time lag between the maximum surface and the corresponding subsurface temperature, every temperature line representing the ground temperature at a given moment can have one or more points of inflection. However in both regions all the temperature lines are located within a roughly triangular space *b c d*. The two sides *b d* and *c d* of this triangle determine the maximum range of the ground temperature for every depth.

The two diagrams, (*a*) and (*b*) lead to the following conclusion regarding the ground frost conditions. In the temperate region (*a*) the ground will freeze in every winter to a depth equal to or somewhat smaller than H_a. This is the depth of the active layer. Below the depth H_a the soil moisture is permanently liquid. In region (*b*) the frost penetrates to depth $H_a + H_p$. To depth H_a it thaws every summer, whereas between depth H_a and $H_a + H_p$ it is permanently frozen. In exceptionally mild winters after a hot summer, the frost may not penetrate to the full depth H_a. In such winters the frozen ground in the active zone will be separated from the permafrost by a layer of unfrozen ground or *talik*. Wherever the layer of permafrost is in a state of progressive degradation this condition is normal. On the other hand, if a cold summer is followed by a cold winter a frozen layer may develop at the base of the active zone which remains unthawed during one or more summers. Such layers are known as *Pereletok*.

Fig. 8 *b* also discloses the existence of a simple relation between the thickness of the active and the permafrost zone. The thickness H_a of the active zone is equal to the ordinate of point *e* at which the freezing line *O f* intersects the lateral boundaries *b c d* of the temperature triangle. In a temperate zone, Fig. 8 *a*, point *e* is located on the boundary *c d* representing the minimum ground temperatures. If the mean annual surface temperature *O a* decreases at unaltered range of annual temperature variations, point *e* moves down, until the freezing line *O f* becomes tangent to the line *b d* representing the maximum ground temperatures. A further decrease of the surface temperature is associated with the formation of permafrost, but, at the same time, point *e* moves up and the depth of the active zone decreases. Hence, in a general way, an increase of the thickness of the permafrost zone must be associated with a decrease of that of the active layer.

Thickness and Continuity of Permafrost.

On account of the physical relationship between surface temperature and the temperature conditions in the subsoil the geographic distribution of permafrost reflects in a general way the existing climatic conditions. This can be seen in Fig. 9 representing the position of the southern boundary of the arctic permafrost with reference to the 0°C mean annual temperature isotherm. In some areas the south-

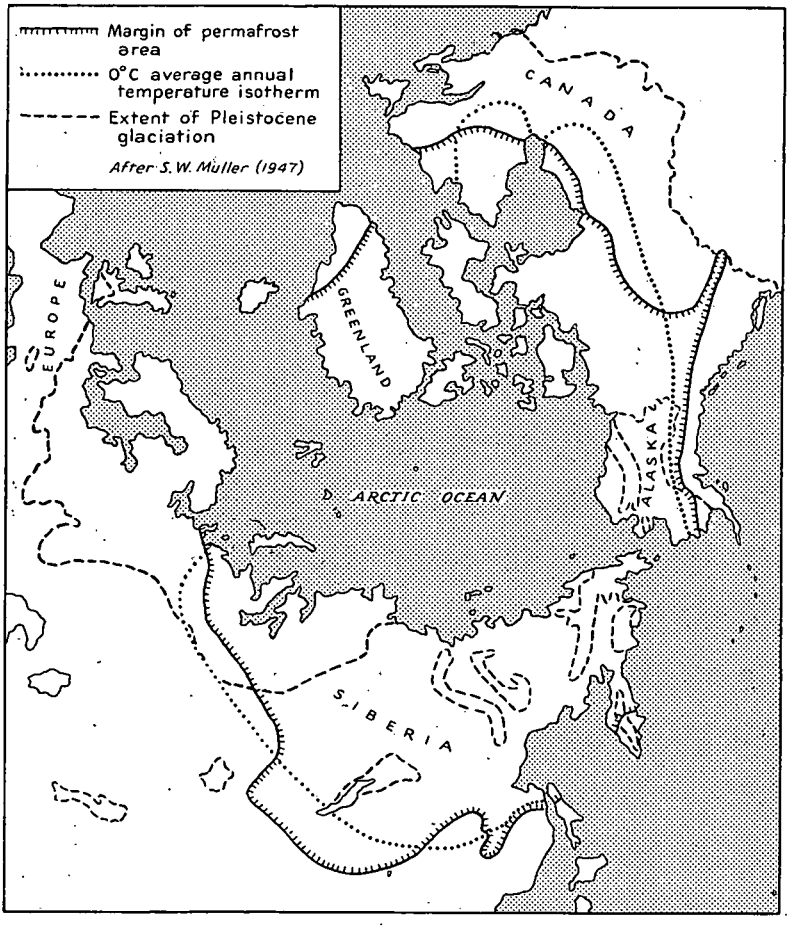

FIG. 9.—APPROXIMATE PRESENT LOCATION OF SOUTH BOUNDARY OF PERMAFROST AND OF 0°C ISOTHERM OF MEAN ANNUAL TEMPERATURE WITH REFERENCE TO EXTREME BOUNDARIES OF PLEISTOCENE ICE SHEET ON NORTHERN HEMISPHERE.

ern boundary is located south and in other parts north of the isotherm, but nowhere is the mean annual temperature at the southern boundary of the permafrost area beyond the narrow range of about —2° and +2°C. Changes in the mean annual temperature of ±2° (C) may occur within less than a century.

Fig. 9 also shows the boundaries of the Pleistocene ice caps. Ice caps can develop only in areas with heavy snow fall. Hence even

during the Pleistocene glaciation large portions of the present permafrost areas were not glaciated.

Wherever the mean annual temperature T (°C) has not risen above the freezing point (0°C) within the last few thousand years, the thickness H_p of the permafrost layer is approximately determined by the equation

$$H_p = -\frac{T}{i_g} \tag{13}$$

wherein i_g denotes the geothermal gradient. Assuming i_g equal to 1°C per 30 meters,

$$H_p \text{ (meters)} = -30\ T\ (^\circ C) \tag{13a}$$

Since the temperature decreases towards the north the thickness of the permafrost layer should also increase towards the north. Furthermore, as indicated in Fig. 8, the increase of the permafrost layer should be associated in a general way with a decrease of the thickness of the active zone. Both conclusions are confirmed by experience.

Along the coast of the Arctic Ocean (north coast of Siberia, Victoria and South Hampton Island in Canada) the mean annual surface temperature is somewhat below —10°C. Hence, according to Equation (13a) the thickness of the permafrost layer should be about 300 meters. According to Muller (1947, Fig. 3), the permafrost layer at the north coast of Siberia has a depth of 200 to 400 m. Muller also reports that the thickness of the active zone decreases from 0.7 to 4.0 meters at the southern boundary of the permafrost territory to 0.2 to 1.6 meters at the coast of the Arctic Ocean. The great difference between the extreme values of the thickness of the active layers for the same mean annual surface temperature is due to the strong influence of the type of surface cover, the soil moisture conditions and various other factors on the depth of seasonal thawing.

Wherever the mean annual surface temperature is below the freezing point the upper boundary of the permafrost is identical with the base of the active layer. Hence, if the mean annual temperature increases or decreases by ΔT without reaching the melting point, the upper boundary of the permafrost layer moves together with the lower boundary of the active zone and the displacement is relatively unimportant. On the other hand, if the mean annual temperature rises, even temporarily, above the freezing point, the upper boundary of the permafrost goes down and the lower boundary goes up. Such

conditions are encountered along the southern boundary of the permafrost zone. Along this boundary it is by no means uncommon to strike permafrost at a depth of 10 or 15 meters below the surface, which is well below the active layer and the base of the permafrost layer may be located at a depth of 20 or 25 meters. Such layers are obviously the remnants of permafrost layers which extended from the base of the active layer at a depth of 2 or 3 meters to a depth of 30 or 40 meters.

In those parts of the permafrost areas where the thickness of the permafrost is close to its maximum, gaps in the permafrost have been found only beneath big river valleys. The valley of the Jenisei at Ust-Prom in northern Siberia is an example. (Muller 1947, Fig. 14). These major gaps are probably due to the fact that big rivers are flowing throughout the year. They constitute large veins of heat supply. Over the area covered by the rivers and the perennial part of the ground water stream associated with the rivers, the mean annual surface temperature is above the freezing point whereas on both sides of the valley it may be —10°C.

Along the southern margin of the permafrost territory the permafrost becomes discontinuous. The permafrost zone surrounds patches of unfrozen ground (talik) and vice versa. Some of these gaps in the continuity of the permafrost are due to the melting action of ground water streams or to the heat which radiates from open water courses. Gaps beneath sloughs may be remnants of the gaps which existed at a time when the sloughs were still occupied by flowing water

Gaps can also be explained by variations in the average heat diffusivity of the top layer of the ground in horizontal directions, as shown in Fig. 10. At points O_1 and O_2, located at the margin of the permafrost territory, the maximum, minimum, and mean annual temperatures are assumed to be the same, but the average diffusivity of the ground is different.

According to Equation (11) the depth H_0 at which the level of zero annual amplitude is located increases in direct proportion to the square root of the diffusivity, α (Equation 7). In Fig. 10b, showing conditions in an area located above strata with low diffusivity, the line *b d* representing the maximum ground temperature intersects the freezing line O_1g whereas in the region represented by Fig. 10b the maximum ground temperature (line *b d*) are above freezing

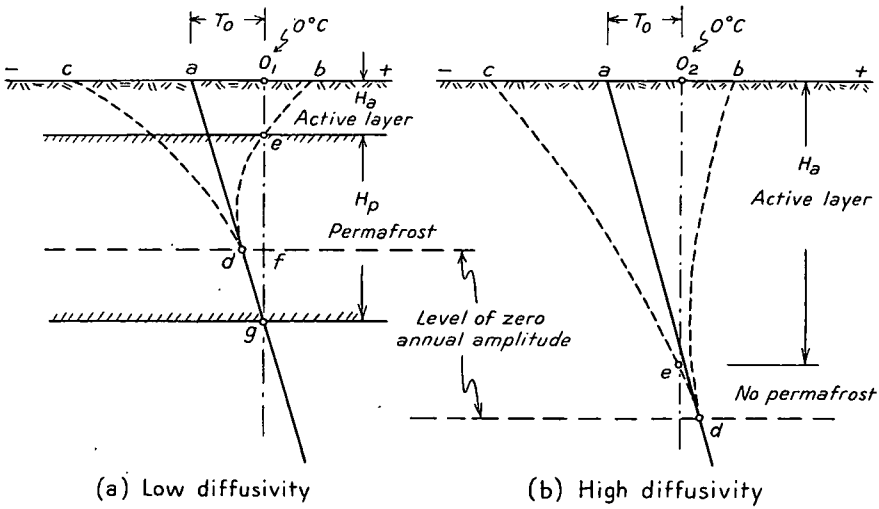

FIG. 10.—DIAGRAM ILLUSTRATING INFLUENCE OF DIFFUSIVITY ON PERMAFROST CONDITIONS AT GIVEN MEAN ANNUAL TEMPERATURE T_0 AT GROUND SURFACE.

Hence beneath point O_1 there will be a layer of permafrost whereas beneath point O_2 no permafrost can exist. Yet the surface temperature conditions at the two points are perfectly identical.

Gaps in the permafrost layer can also be produced by installing and continuously operating heating systems in buildings located above permafrost. If a heating system maintains in a building (Fig. 11) an almost constant temperature, the temperature conditions in the subsoil will gradually approach a stationary state represented by a temperature line similar to *m n r*. The position of this line can be computed on the basis of the laws of the stationary flow of heat through solids. If this line, like *m n r* in Fig. 11, does not intersect the freezing line *O f*, the heat emanating from the building will gradually thaw a hole through the layer of permafrost. Otherwise the permafrost located below the building will merely shrink, whereby the base of the permafrost layer will rise and its surface will descend. A method for estimating the rate of thawing will be described in the last part of this paper.

Aggradation and Degradation of Permafrost Due to Natural Causes.

In connection with aggradation and degradation of permafrost due to natural causes distinction must be made between the central

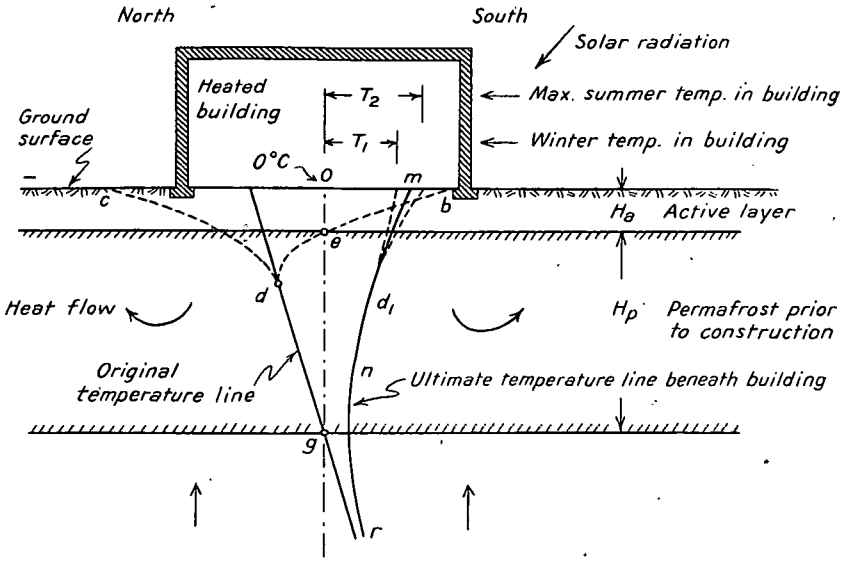

FIG. 11.—EFFECT OF OPERATING A HEATING SYSTEM ON THE TEMPERATURE CONDITIONS IN THE SUBSOIL OF THE HEATED BUILDING.

part of the permafrost area where the mean annual surface temperature has not reached the melting point for many thousands of years and the marginal parts where it has fluctuated between values above and below the freezing point.

In the central part the upper boundary of the permafrost remains identical with the base of the active layer. The corresponding vertical displacements of the upper boundary are relatively unimportant and the major changes take place at the base of the permafrost layer. The rate at which these changes take place are determined by the rate q_i of heat radiation from the interior of the earth, which is about 40 cal cm^{-2} per year and by the heat of fusion of the ice which is 80 cal/gm^{-1} or about 70 cal cm^{-3}.

If the ice occupies $n = 30\%$ of the total volume of a frozen sediment, every ccm of sediment contains 0.3 ccm of ice and the quantity of heat required to melt it is

$$q_f = 0.3 \times 70 = 21 \text{ calories.}$$

Hence the rate at which the lower boundary of the permafrost recedes in an upward direction cannot exceed the value

$$q_i/q_f = \frac{40}{21} = 2 \text{ cm/year.}$$

The real rate is somewhat smaller on account of the heat required for increasing the average temperature of the permafrost layer and of the sediments located below it. The time required for the melting of a permafrost layer with a thickness H from below is

$$t = \frac{H \text{ (centimeters)}}{2 \text{ (centimeters/year)}} \tag{14}$$

If H = 200 meters, t = 10,000 years.

In the marginal area where the mean annual temperature is alternately above and below the freezing point the permafrost layer degrades simultaneously from below and above. The rate of degradation from below is determined by Equation (14). Concerning the rate of degradation from above, trial computations based on the theory of permafrost degradation presented in the last part of this paper have led to the following conclusions. If the mean annual surface temperature had suddenly risen by 2°C from —1°C to +1°C, the upper boundary of the permafrost layer would have descended within less than one hundred years from the base of the active layer to a depth of 15 meters below the surface and during the same period the lower boundary would have risen by about two meters.

These estimates demonstrate that the position of the southern boundary of the permafrost area is determined by fluctuations of the mean annual surface temperature with a period of a few hundred years and not of thousands of years. This accounts for the fact that the southern boundary of the permafrost area follows fairly closely the 0°C mean annual temperature isotherm and it justifies the statement that the geographic distribution of the permafrost area reflects existing and not Pleistocene climatic conditions.

The successive stages in the formation and subsequent degradation of permafrost are illustrated by Fig. 12. In all stages the slope of the temperature line T, representing the increase of the temperature with depth remains practically unaltered whereas the abscissa of the upper end of the line, equal to the mean annual surface temperature, varies.

Stage a corresponds to sub-tropical climatic conditions similar to those which prevailed for instance in Greenland and in Spitzbergen during the Tertiary period. Following stage a the temperature decreased. As soon as the minimum surface temperature reached the

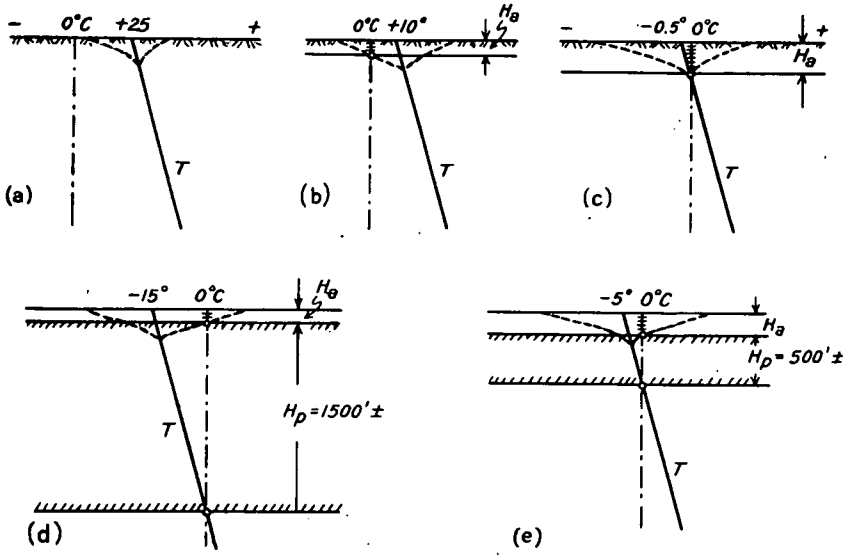

FIG. 12.—DIAGRAMS ILLUSTRATING THE EFFECT OF A GRADUAL TRANSITION FROM A SUBTROPICAL CLIMATE (a) TO A MODERATE (b, c) AND AN ICE-AGE CLIMATE (d) TO A RECENT ARCTIC CLIMATE (e) ON GROUND FROST CONDITIONS.

freezing point the formation of an active layer started (stage *b*). The thickness of the active layer became a maximum in stage *c* when the curve of maximum temperatures touched the vertical line representing zero °C. A further decrease of the mean annual temperature led to the formation of permafrost, stage *d*, associated with a decrease of the thickness of the active layer. This condition still prevails in arctic regions, with no change other than occasional increase or decrease of the thickness of the permafrost layer, associated with a decrease or increase of that of the active layer, due to changes of the mean annual temperature, for instance from —10°C (stage *d*) to —5°C (stage *e*).

In accordance with the laws of heat conduction the change of the thickness of the layer of permafrost lags behind the change of the mean annual temperature and the lag increases approximately with the square of the thickness of the layer. The lag due to the time required for the temperature of the ground to adapt itself to a change in the surface temperature must be added to the lag due to the heat of fusion of the ice. On account of the heat which is liberated at freezing and absorbed during thawing (about 80 calories per gram

of water) the rise of the temperature in permafrost to values above 0°C proceeds as if each body of permafrost were surrounded by a skin with a very low heat conductivity. The insulating effect of the heat of fusion will be realized if the following is considered. Suppose the porosity of a stratum of completely saturated and frozen silt stratum is 50 per cent. It takes less than two calories per cubic centimeter to increase the temperature of the silt-water system from —1°C to +1°C, but the thawing of the ice contained in the voids of the silt requires an additional inflow of heat of about 40 calories per cubic centimeter.

On account of the time lag between a change of the mean annual surface temperature and the corresponding change of the ground temperature it is possible that the geothermal gradient between the ground surface and the center of the permafrost stratum is temporarily reversed, involving a decrease of the temperature with increasing depth below the level of zero annual amplitude. Such a case has been reported by Sumgin (Muller 1947, Fig. 7, p. 15).

The most conspicuous consequence of the time lag between the surface and the ground temperature is the occasional occurrence of one or more layers of unfrozen ground or talik between layers of permafrost. (*Layered Talik,* Muller 1947, p. 10). To demonstrate the prerequisites for the formation of layered talik two layers of permafrost, Fig. 13, will be considered, separated by a layer of talik. The surface of the lower permafrost layer is located far below the bottom of the present active zone. This fact indicates that the lower layer represents the results of the progressive degradation of a very much thicker layer. Progressive degradation of a permafrost layer can be started only by an increase of the mean annual surface temperature. The presence of the upper permafrost layer indicates that the increase of the temperature was followed by a decrease. Since cooling, like heating, proceeds from the surface in a downward direction, the secular cold wave started a new permafrost layer above the original, degrading one. Since the degrading layer is trapped between two unfrozen layers, its degradation inevitably continues as long as the talik layer exists, while the upper permafrost stratum aggrades. Hence the presence of a layer of talik between two layers of permafrost is a transitory phenomenon caused by important secular variations of the mean annual surface temperature.

Fig. 13 shows the temperature conditions which prevail in a two-

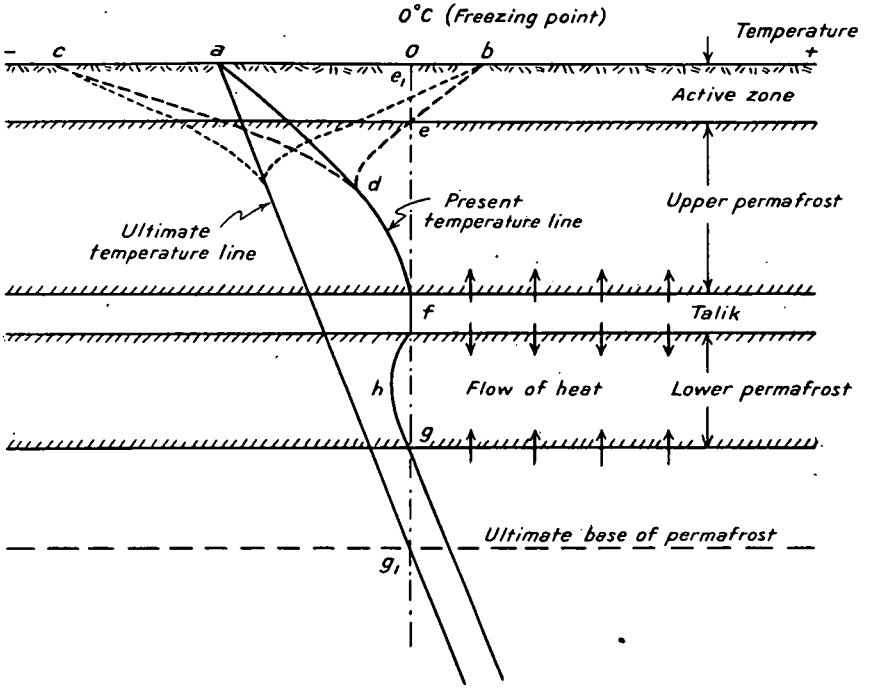

FIG. 13.—DIAGRAM ILLUSTRATING THE FORMATION OF A TALIK AS A RESULT OF A DROP OF THE MEAN ANNUAL TEMPERATURE TO A VALUE BELOW THE FREEZING POINT AFTER A SHORT PERIOD OF PERMAFROST DEGRADATION DUE TO A TEMPORARY RISE OF THE MEAN ANNUAL TEMPERATURE TO A VALUE ABOVE THE FREEZING POINT. THE TEMPERATURE CONDITIONS WHICH PREVAILED DURING THE PERIOD OF PERMAFROST DEGRADATION ARE NOT SHOWN.

layer sheet of permafrost. As time goes on the temperature line *a f h g* moves into the ultimate position *a* g_1 provided the mean annual surface temperature retains its low value long enough. The last stage of the transition may be associated with a downward movement of the base of the permafrost sheet as indicated in the figure, but this movement cannot possibly start before the layer of talik is completely frozen.

PERMAFROST INVESTIGATIONS FOR ENGINEERING PURPOSES

Practical Implications of Permafrost.

A general review of the engineering problems associated with permafrost has been published by S. W. Muller (1947). Most of them grow out of the consequences of permafrost degradation or ag-

gradation induced by man-made changes on the surface such as the construction of buildings and fills or the replacement of the natural soil cover by artificial pavements. Any operation of this kind changes the temperature regime in the subsoil which in turn causes the upper boundary of the permafrost layer to advance towards the ground surface or to recede. Since frozen, saturated layers are impermeable a local rise of the permafrost surface may cause undesirable changes in the subsurface conditions, whereas the melting of a permafrost layer may cause important differential settlement.

On account of the influence of the aggradation or degradation of permafrost layers on the performance of building foundations and lines of communication it is necessary in permafrost regions to supplement the conventional methods of subsurface exploration by others furnishing information on the location, texture and structure of the layer or layers of permafrost. The methods of exploration include soundings, borings, sampling, testing and subsurface temperature observations. The observational data furnish the basis for estimating the effect of the proposed modifications in the surface conditions on the position of the upper boundary of the permafrost layer, the rate at which this boundary will approach its final position and the amount of settlement which will be associated with the recession of the permafrost boundary.

Subsoil Exploration in Permafrost Regions.

Selection of the site for a structure should be preceded by a general survey of the area, including a study of the topographical features, preferably by aerial photography, of the climatological conditions, the drainage conditions and the distribution of the different species of the most common plants. The botanical studies should be made by a plant ecologist who is familiar with permafrost regions. Subsoil exploration should not be started until all the information to be derived from surface evidence has been secured. The technique of the subsoil exploration depends to a large extent on the degree of saturation of the layers of permafrost.

If the permafrost layer consists of sediments which were completely saturated before they froze, it is so hard that the upper boundary of the permafrost layer can be determined reliably by driving steel rails into the ground by means of a pile driver. At the permafrost boundary the steel rails meet refusal. This procedure has been

successfully used in a permafrost investigation at the site for a large hangar at Mile 26 on the Tanana River east of Fairbanks in Alaska. The subsoil consisted of layers of sand with a variable admixture of gravel. Beneath a slough the ground was frozen, forming a belt of permafrost with a width of about 1,200 feet. Under the area covered by the slough the top surface of the permafrost was located at a depth of a few feet and it was identical with the base of the active layer. At the edges of the permafrost belt the depth to permafrost increased to a maximum of about 35 ft. The base of the permafrost layer was at a depth of about 60 ft over the entire width of the belt.

In order to determine the location and the width of the permafrost belt, steel rails were driven into the ground. Within the ground the blows per foot, down to the maximum depth of driving, which was about 60 ft, nowhere exceeded 15. Fig. 14 shows a typical rail penetration record for a test rail which did not strike any permafrost. However, if the test rail struck frozen ground the number of blows per foot increased abruptly to more than 60. Hence the rail penetration diagrams left no doubt concerning the position of the upper boundary of the permafrost layer.

Entirely different conditions have been encountered at a site in the proximity of Ladd Field although this site is located on sediments similar to those at Mile 26. Within the unfrozen layer the number of blows per foot of penetration (70 lb rails, 2,200 lbs hammer and 10 ft drop) varied between 1 and 15 whereas in the frozen ground it ranged between 10 and 90. Hence if the number of blows was between 10 and 15 it was impossible to decide whether the rail had penetrated frozen or unfrozen ground. Such large variations of the resistance against the penetration of the rails into frozen ground are hardly conceivable unless the percentage of void space occupied by ice also varies between wide limits. Hence in spite of the similarity of the geological conditions at the two sites, the physical properties of the frozen ground were extremely different. Furthermore the top surface of the frozen ground at Mile 26 had the character of that of a gentle ridge, whereas at Ladd Field it was very uneven, involving differences in elevation of 20 ft over a distance of not more than 40 ft. The causes of the differences remained unexplained.

If rail penetration tests fail to furnish reliable information con-

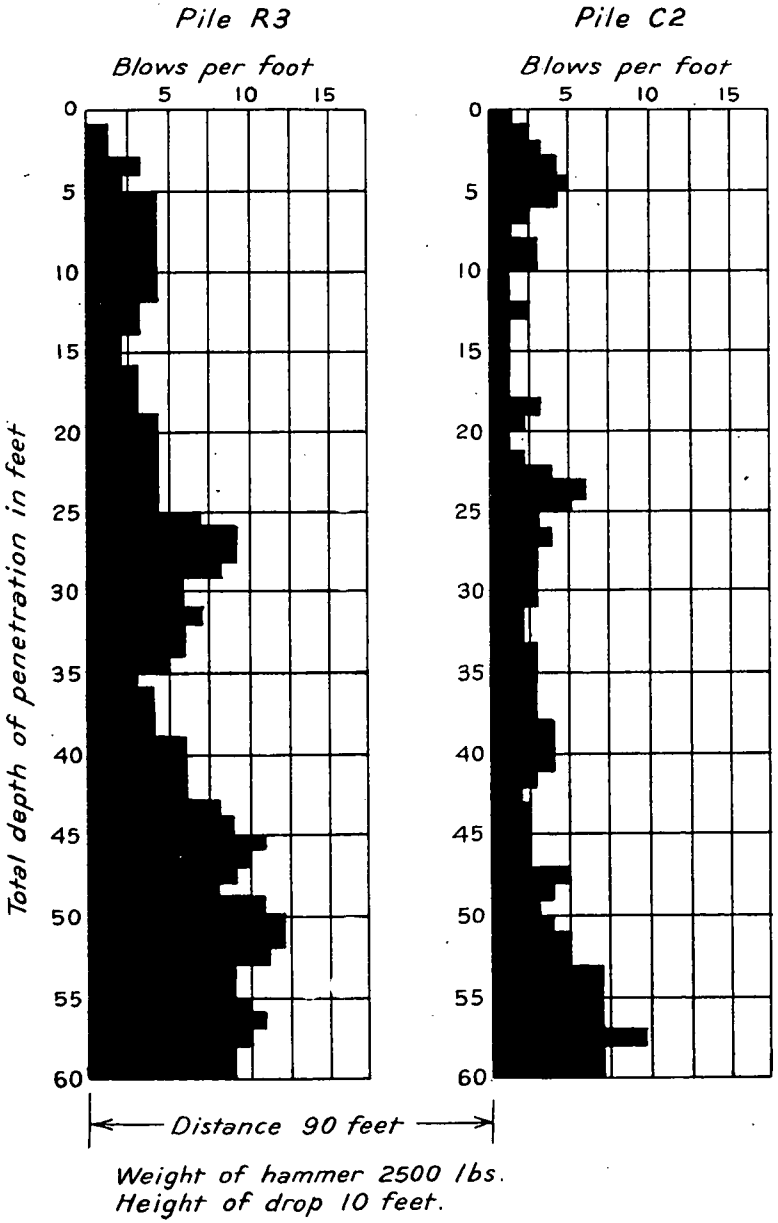

FIG. 14.—RELATION RETWEEN TOTAL DEPTH OF PENETRATION OF STEEL RAILS AND BLOWS PER FOOT OF PENETRATION INTO UNFROZEN SAND AND GRAVEL AT THE SITE OF A HANGAR AT MILE 26 IN ALASKA.

cerning the position of the top surface of the permafrost layer, it is necessary to supplement them by sampling operations.

Very satisfactory samples of frozen silt and sand, with a diameter of about six inches, have been obtained by the U. S. Army Engineers with rotary drills equipped with a sawtooth cutter. However, when the equipment was used in frozen sand and gravel, difficulties were encountered. The outer layer of the core melted on account of the heat developed by the rotating tool and the core slipped out of the barrel. Better results were obtained when the washwater was mixed with alcohol and cooled to about 32°F.

The samples should be examined immediately after recovery with a powerful hand lense to find out whether the voids are completely or partially filled with ice. The strength of the frozen material should be appraised by trying to crush the fragments of the frozen core between the fingers. The hardness of the fragments should be described as very high (like that of a dry, fat clay), high, medium, low, very low (like that of a dried silty sand). These tests are very important, because the degree of saturation of the frozen material with ice has a decisive influence on the rate at which the body of permafrost will degrade at a given supply of heat and on the cost of thawing the layer.

On routine jobs, drillholes in frozen ground are still made with the chopping bit which breaks the material up into angular fragments and rock powder. The fragments are recovered with a bailer or other suitable tools. By the time the fragments arrive at the surface, the ice may already have melted. Therefore the drilling foreman cannot always be sure whether he has encountered frozen ground or a dense and slightly cemented layer of gravel which also delays progress. Furthermore, if two layers of permafrost are separated by a layer of unfrozen ground or talik, the presence of the talik may easily escape the foreman's attention. Hence the customary technique of sampling in frozen ground still leaves a wide margin for improvement.

If drillhole samples do not provide conclusive information concerning the position of the upper boundary of the permafrost layer, the boundary can be located by means of temperature measurements, because within the permafrost body the temperature of the ground is at or below the freezing point. In connection with permafrost investigations the most convenient instrument for temperature measurements is the thermocouple.

A *thermocouple* consists of two wires known as *leads*. The wires

are made out of different metals and their two ends are interconnected. The lower junction is located below the ground surface in a drillhole and the upper one at an accessible, convenient place above ground. A difference in temperature at the lower and upper junction between the wires produces an electric potential difference between these points and the difference increases in simple proportion to the difference in temperature. Hence if the temperature at the upper junction and the thermo-electric force are measured, the temperature at the lower junction can be computed.

It may also be possible to determine the position of the upper boundary of permafrost layers by means of *geophysical methods*. These include the *electric potential* and the *seismic method*. Both methods have successfully been used for determining the position of the boundary between unconsolidated sediments and rock. However the application of geophysical methods to the problem of determining the upper boundary of permafrost layers is still in an experimental stage.

Rate of Thawing Beneath Heated Buildings

If a building located above a permafrost layer is heated throughout the winter, the heat which flows from the heated rooms into the ground increases the ground temperature whereby the 0°C isothermal surface of ground temperature and the upper boundary of the permafrost layer recede beneath the source of heat to greater depth. If the process of thawing reaches the base of the permafrost layer, the layer is punctured and the entire subsoil of the heated structure will ultimately be in an unfrozen state.

The initial rate at which the upper boundary of the permafrost layer recedes beneath the heated structure can be estimated on the simplifying assumption that the heat escapes from the structure in vertical downward directions only. It is also assumed that the floor of the basement of the structure rests directly on the frozen ground and the heat required for increasing the average temperature of the permafrost layer is disregarded. Let

T_1 = temperature in the basement in (°C),
n = porosity of the frozen ground,
$\gamma = 2.65$ gm cm^{-3} average unit weight of the soil particles,
$\gamma_w = 1.0$ gm cm^{-3}, unit weight of water,

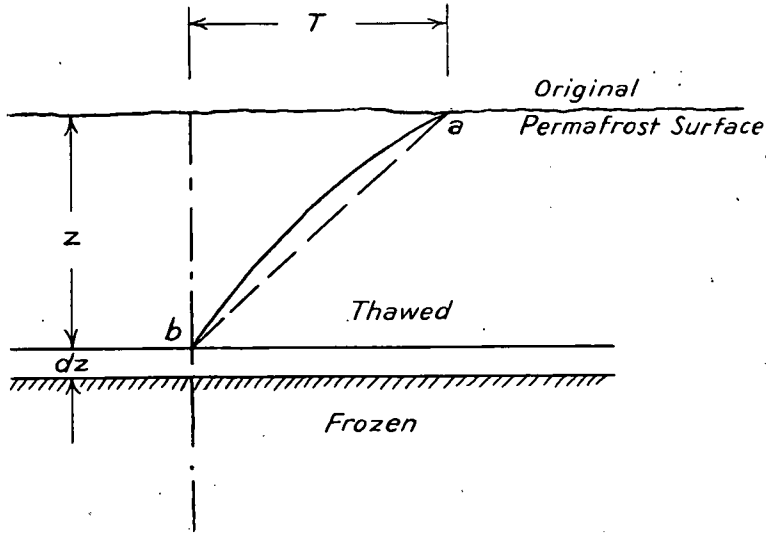

FIG. 15.—DIAGRAM ILLUSTRATING PROGRESS OF THAWING BENEATH HEATED BASEMENT ON PERMAFROST.

$\gamma_i = 0.9$ gm cm^{-3}, unit weight of ice,
$c_h = 0.2$ cal gm^{-1}(°C)$^{-1}$, average heat capacity of the soil particles,
$c_{hw} = 1$ cal gm^{-1}(°C)$^{-1}$, heat capacity of liquid water,
$h_f = 80$ cal gm^{-1}, heat of fusion of ice, and
$k_h = 6 \times 10^{-3}$ cal cm^{-1} sec^{-1} (°C)$^{-1}$, the average thermal conductivity of the soil located between the base of the structure and the top of the permafrost layer.

Fig. 15 illustrates the progress of thawing of the permafrost layer located beneath the heated building.

If the voids of the frozen ground are completely filled with ice, one cc of frozen ground contains 2.65 (1-n) gms of mineral substance and 0.9 n gms of ice. The heat required to melt the ice in the voids of 1 cc of frozen ground is $\gamma_f = h_f\, n\, \gamma_i = 80 \times n \times 0.9 = 72\, n$ cal cm^{-3} and the quantity of heat required to increase the temperature of 1 cc of thawed, but saturated ground by 1°C is

$$q_t = \gamma(1\text{-}n)\, c_h + \gamma_w\, n\, c_{hw} = 2.65\,(1\text{-}n) \times 0.2 + 1 \times n \times 1.0$$
$$= 0.53 + 0.47\, n.$$

At the boundaries of the frozen ground the temperature is approximately 0°C and the average temperature in the soil between basement and permafrost is $T_1/2$. By the time the surface of the layer of permafrost has receded from the bottom of the floor of the basement to a depth z below the bottom, the total quantity of heat which has entered the subsoil per unit of area of the basement floor is

$$Q_h = z\left(q_f + \frac{T_1}{2} q_t\right) = z\left[72\ n + \frac{T_1}{2}(0.53 + 0.47\ n)\right] = Az$$

While the heat flows from the basement towards the permafrost layer, part of the heat is consumed in raising the temperature of the layer of thawed soil through which it flows. As a consequence the rate of heat-flow through horizontal sections decreases with increasing distance between the section and the basement floor and the corresponding thermal gradient, represented by the cotangents of the slope angle of the plain curve in *a b* in Fig. 15, also decreases.

To simplify the computation it will be assumed that the thermal gradient is constant and equal to the cotangents T_1/z of the slope angle of the straight line *a b* in Fig. 15. On this condition the average rate q of the flow of heat towards the permafrost zone is

$$q = \frac{T_1}{z}\ (\text{thermal gradient}) \times k_h\ (\text{heat conductivity})$$

The quantity q must be equal to the rate of increase of the amount of heat which has entered the ground,

$$q = \frac{dQ_h}{dt} = A\frac{dz}{dt}$$

whence

$$\frac{T_1}{A} k_h\, dt = z\, dz$$

and

$$z = \sqrt{\frac{2t\, T_1 k_h}{A}} \qquad (15)$$

wherein $A = 72\ n + \frac{T_1}{2}(0.53 + 0.47\ n)$.

The approximate validity of Equation (15) was demonstrated by

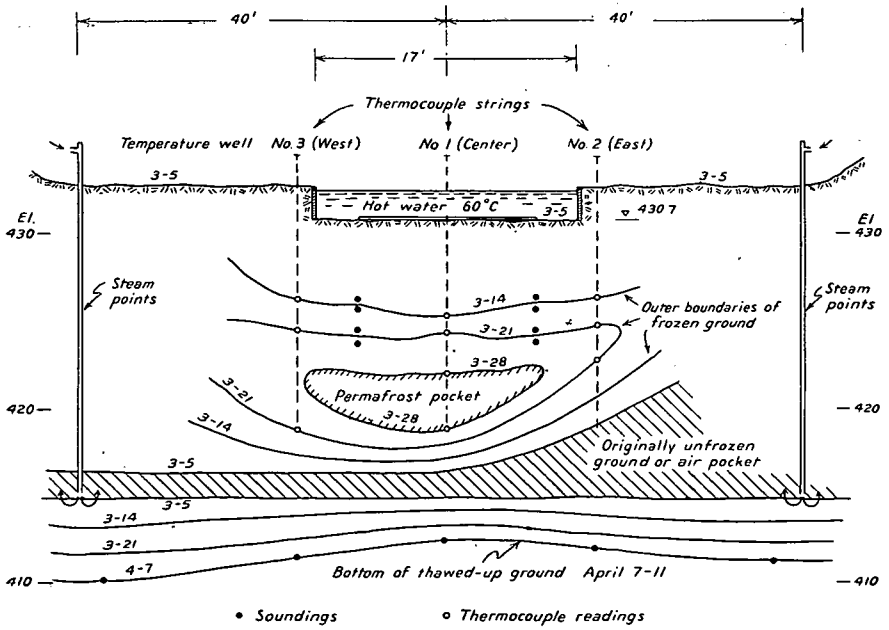

FIG. 16.—PROGRESS OF THAWING DURING LARGE-SCALE THAWING TEST AT LADD FIELD, ALASKA. THE FIRST ONE OF THE NUMERALS (FOR INSTANCE 3-14) INSCRIBED AT THE PERMAFROST CONTOURS INDICATES THE MONTH (3—MARCH) AND THE SECOND ONE THE DAY AT WHICH THE POSITION OF THE PERMAFROST BOUNDARY WAS DETERMINED.

the results of the thawing test shown in Fig. 16.[2] A square pool covering an area of 17 by 17 ft was established on the bottom of an open excavation with a diameter of about 100 ft. The bottom of the pool was located on permafrost and the temperature of the water in the pool was kept at 60°C. During the test the progress of thawing was observed by soundings, supplemented by temperature readings on three strings of thermocouples, No. 1, 2 and 3 in Fig. 16. The porosity n of the subsoil ranged between 15 and 46%, and averaged 25. The results of the observations are shown in the same figure. During the first 16 days after the flooding of the bottom of the pool with hot water the surface of the permafrost had receded to a depth of 6.8 ft below the bottom of the pool.

Introducing into Equation (15) the numerical values $T_1 = 60°C$,

[2]The experiment was made in the spring of 1949 on Ladd Field near Fairbanks, Alaska, by Fay, Spofford and Thorndike, Consulting Engineers, in accordance with specifications prepared by the writer.

$t = 16$ days $= 14 \times 70^5$ seconds, $n = 0.25$ and $k_h = 6 \times 10^{-3}$ cal cm^{-1} sec^{-1} (°C)$^{-1}$, the values

$$A = 37.5 \text{ cal cm}^{-3}$$

and

$$z = 5.4 \text{ ft}$$

were obtained. The real value of 6.8 ft exceeds the computed one by 26%. In other words, the test result demonstrated the approximate validity of Equation (15). On account of the inevitable errors involved in the evaluation of n and k_h and of the arbitrary assumption that the voids of the soil are completely filled with ice more refined methods of computation would hardly be justified.

In order to accelerate the process of thawing at the test shown in Fig. 16, sixteen steampoints were introduced into the ground on a circle about the center of the pool with a radius of 40 ft. If the frozen ground had been homogenous and of the same character as the ground beneath the pool, the thawing produced by the steampoints would have advanced towards the vertical centerline of the pool in radial directions. At the end of a period of 16 days the inner boundary of the thawed-up space could not have been located at a distance of more than a few feet from the cylindrical surface occupied by the steampoints.

In contrast to this theoretical conclusion the heat radiating from the steampoints rapidly advanced within a thin layer, located at about el. 415, towards the vertical centerline of the pool and started degradation of the permafrost located above and below the "permeable" zone as shown in Fig. 16. This observation indicates that the body of frozen soil contained, at about el. 415, a thin stratum which was either unfrozen to start with or else very incompletely saturated with ice. The existence of such layers had already been suggested by the excessive variation of the resistance of the permafrost layer against the penetration of steel rails and it indicates the complexity of the permafrost conditions at the margin of permafrost areas. Within a distance of 100 ft from the center of the test area the original surface of the permafrost layer descended without any visible reason from about el. 432 to about el. 412.

As the process of thawing beneath the base of a heated structure proceeds, more and more of the heat which radiates from the base of the structure escapes through the unfrozen ground towards the surface of the terrain surrounding the structure. Hence if the

thickness of the layer of permafrost is great compared to the width of the area covered by the structure a state may be reached in which the further increase of z becomes imperceptible. The computation of this limiting depth is beyond the scope of this paper.

Settlement Due to Permafrost Degradation.

The settlement phenomena produced by the degradation of permafrost layers differ only in scale from those which take place in spring on the surface of the ground due to the melting of the ice in the active zone. In both cases the importance of the settlement depends primarily on whether or not the freezing of the ground was associated with the formation of layers or pockets of clear ice.

The prerequisites for the formation of ice lenses in permafrost zones appear to be identical with those for the formation of ice lenses in the active zone. However, in the active zone the length of the period of transition from the frozen into the unfrozen state and vice versa is measured in months whereas in the permafrost zone the period amounts to centuries or even to thousands of years. As a result of the rapid rate of aggradation combined with the shallow depth of the zone involved in freezing and thawing the thickness of the ice layers in the active zone is extremely small compared to that of the bodies of ice which are encountered in permafrost layers.

If the bodies of clear ice contained in a permafrost layer melt due to a change in climatic conditions or man-made changes at the surface, the roof of the cavities produced by the melting subsides or collapses. The funnel-shaped depressions, which appear at the surface above the seats of the roof collapse, are similar to the sinkholes in limestone terraines. Therefore the pock-marked topography of the surface of the ground above degrading layers of permafrost with ice inclusions has been referred to as *"Thermo-Karst"* (Muller 1947, p. 83).

If the melting has been produced by a source of heat operating in a structure located above a layer of permafrost with ice inclusions, the melting of the ice may—and repeatedly did—cause destructive unequal settlement of the foundations of the structure. Settlement of this kind can be avoided by either one of two procedures. One of them consists in providing an insulating layer or a ventilated space between the floors of the heated rooms and the ground surface. The other one consists in thawing the ground to the maximum depth

to which thawing may proceed. The foundations are then supported on piles driven through the thawed layer into permanently stable material.

While the roof of the cavities formed by the melting of the ice subside or collapse, the ground located above the cavities is badly disturbed, like the soil above melted ice lenses in the active zone and it may take many years until it consolidates under its own weight. Therefore foundation piles surrounded by the damaged ground are acted upon by "negative skin friction".

The choice between the two procedures for preventing damage due to the melting of bodies of clear ice in permafrost layers is commonly governed by economic considerations.

If the grainsize characteristics of the frozen ground, like those of clean sand or gravel, exclude the possibility of large scale ice segregation, bodies of ice encountered in the permafrost layer must have existed before they were buried by the accumulating sediments. Such bodies might consist either of remnants of a glacier or of frozen pools. However, the writer has not yet seen any conclusive evidence that such bodies exist except in the proximity of the edge of existing glaciers.

If a frozen layer of coarse grained sediments does not contain buried bodies of ice, the settlement of the surface due to melting cannot exceed the settlement due to the volume decrease associated with the melting of the ice contained in the voids, which is roughly equal to 10% of the volume of the ice. Hence the upper limiting value for the settlement of the surface due to the melting of the ice in a coarse grained permafrost layer with porosity n is

$$s_t = 0.1\ n \tag{16}$$

per unit of thickness of the melted part of the layer. If $n = 0.3$, $s_t = 3\%$ However the real value of s_t cannot exceed the increase of the porosity of the sediment caused by the preceding process of freezing. While the water in the voids freezes it displaces the major part of the liquid water and the porosity of the sediment remains practically unaltered except in the proximity of the ground surface where the overburden pressure resisting the wedge-action of the ice is small. Hence one should expect that the real value of s_t would decrease rapidly with increasing depth below the surface.

Conclusive information concerning the settlement due to the thawing of coarse grained layers of permafrost can be obtained only

by large scale thawing tests in the field combined with settlement observations. Test data of this kind are still very scarce.

At one test performed at Mile 26 on the Tanana River in Alaska[3] the frozen ground consisted of frozen sand and gravel with an average porosity of about 25%. The interstices between the grains were completely filled with ice. The settlement benchmark was established on the top of a disk which rested on the surface of the permafrost layer at a depth of about 3 ft below the original ground surface. At the surface of the permafrost layer the measured value of s_t was about 2.5%, but it decreased rapidly with increasing depth. The upper limiting value of s_t, Equation (16), is 2.5%.

In the test illustrated by Figs. 16 and 17. the bottom of the heated pool was located on the surface of the permafrost, at a depth of about 13 ft below the original ground surface. The average porosity of the sediment was 25%. According to Equation (16) the upper limiting value for s_t is 2.5%. The measured value decreased from 1.8% at a depth of 13 ft below the original ground surface to very small values at a depth of 30 ft.

These observations are still inconclusive. However they show that the settlement of the ground surface due to the melting of coarse-grained permafrost is not necessarily negligible.

Conclusions.

1. Our knowledge of the physical aspects of permafrost is still expanding. So far not much is known concerning the effect of relative density and degree of saturation on the thermal conductivity of coarse-grained sediments, the effect of freezing temperatures on the strength of saturated clay in an undisturbed state, the relation between stress, strain and time for pure ice, and various other fundamental relationships. Yet in spite of the gaps in our knowledge of the details, the causes of permafrost and its salient features are already reliably established.

2. The heat conductivity of both frozen and unfrozen ground depends not only on the porosity and the water content but also on the average mineral composition of the grains and various other factors. Therefore the real conductivity values may deviate ±25% or more from the average values represented in Fig. 3. However for most practical purposes the average values are accurate enough.

[3]By the engineers of Fay, Spofford and Thorndike, under the writer's supervision.

3. The geographic distribution of permafrost is essentially determined by the present climatic conditions. The time lag between a change in the mean annual temperature and the corresponding displacement of the southern boundary of the permafrost area is measured in decades or centuries and not in thousands of years.

4. If the voids of a coarse-grained soil are completely filled with ice the upper boundary of the permafrost layer can be determined by means of rail penetration tests and the rate of permafrost degradation due to radiation from heated structures can be estimated by means of the procedure described in this paper. However if the permafrost layer contains layers or lenses of porous or spongy material, both methods become unreliable. Such conditions have been encountered at the margin of the permafrost area in Alaska. The available methods for distinguishing between compact unfrozen and porous frozen soil are still unsatisfactory.

5. Lenses or layers of clear ice are formed during the freezing of fine grained soils only. Coarse grained sediments may conceivably contain bodies of clear ice which were formed or deposited on the surface and subsequently buried, but so far no such relics have been encountered.

6. The upper limiting value for the settlement of the surface due to the thawing of frozen coarse grained sediments with a porosity n is 0.1 n per unit of depth of thawing. The few observational data which are available indicate that the real value of the shrinkage due to melting decreases from the upper limiting value close to the ground surface to almost zero at a depth of a few tens of feet. Nevertheless the settlement of the surface is by no means necessarily negligible.

Acknowledgments.

The writer wishes to express his gratitude to Fay, Spofford and Thorndike, consulting engineers in Boston, who gave him an opportunity to coöperate with them on some of their permafrost problems in Alaska and to utilize the data obtained by the field observations in the preparation of this paper. Most of the figures accompanying this paper have been prepared by the writer in 1949 for his contribution on "Physics of Permafrost" to the Encyclopedia Arctica and the writer is indebted to Mr. Hjalmar Stefansson for the permission to reproduce them. He is also indebted to Mr. L. E. Wood, Administrative Assistant, St. Paul District, Corps of Engineers, U. S. Army, for valuable comments and suggestions.

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Appendix A

Approximate Computation of Temperature Distribution in Semi-Infinite Solid After a Temperature Change at Surface

Fig. A-1 represents a section through a semi-infinite solid at right angles to its surface. Prior to time $t = 0$ the temperature of the solid is uniform and equal to T_0. At time $t = 0$ the temperature at the surface increases from T_0 to $T_0 + \Delta T_0$ and then remains constant. According to the rigorous theory of heat conduction the distribution of the temperature in the solid at any time is roughly parabolic. Beyond a certain depth z_1 below the surface, the temperature change is imperceptible. As time goes on, z_1 increases and finally it becomes equal to infinity.

To simplify the computation, it is assumed that the distribution of the temperature is strictly parabolic as shown in Fig. A-1. Below depth z_1 the change in temperature is zero. Let

k_h = coefficient of thermal conductivity
c_h = heat capacity
w_t = unit weight

$$a = \frac{k_h}{c_h w_t} = \text{diffusivity.}$$

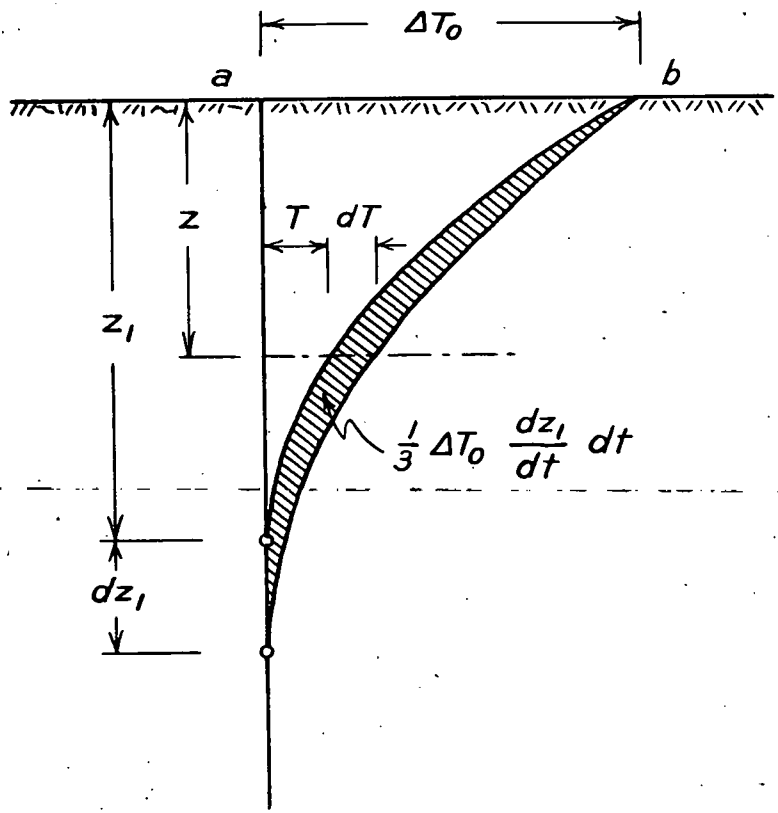


FIG. A-1.—DIAGRAM ILLUSTRATING APPROXIMATE METHOD FOR COMPUTING FLOW OF HEAT INTO SEMI-INFINITE SOLID AFTER THE TEMPERATURE AT THE SURFACE WAS INCREASED BY ΔT_0

At any depth z smaller than z_1 the temperature change T at any time t is

$$T = \Delta T_0 \frac{(z_1 - z)^2}{z_1^2},$$

the thermal gradient at that depth is

$$\frac{dT}{dz} = -\frac{2\,\Delta T_0}{z_1^2}(z_1 - z)$$

and the quantity of heat which flows through the section per unit of time is

$$q = -k_h \frac{dT}{dz} = k_h \frac{2\,\Delta T_0}{z_1^2}(z_1 - z)$$

At the surface $z = 0$ and the quantity of heat which enters the solid through the surface per unit of time is

$$q_1 = k_h \frac{2\,\Delta T_0}{z_1} \tag{1}$$

The total quantity of heat Q which entered the solid between $t = 0$ and time t is equal to the area $a\ b\ c$ in Fig. A-1 multiplied by the heat capacity c_h and the unit weight w_t of the solid.

$$Q = \frac{1}{3}\,\Delta T_0 z_1 \times w_t\, c_h$$

and it increases per unit of time by

$$q_1 = \frac{dQ}{dt} = \frac{1}{3}\, w_t\, c_h\, \Delta T_0\, \frac{dz_1}{dt}$$

This quantity must be equal to the quantity of heat, q_1 (Equation 1) which entered the solid per unit of time, whence

$$\frac{1}{3}\, w_t\, c_h\, \Delta T_0\, \frac{dz_1}{dt} = k_h\, \frac{2\Delta T_0}{z_1}$$

The time t_1 at which the change in temperature has advanced to depth z_1 is

$$t_1 = \frac{1}{12}\, \frac{w_t\, c_h}{k_h}\, z_1^2 = \frac{z_1^2}{12a} \tag{2}$$

This equation is identical with the approximate equation for the consolidation of a semi-infinite mass of ideal clay. The diffusivity a corresponds to the coefficient of consolidation, which has the same dimension, $cm^2\ sec^{-1}$. It has been shown[4] that the errors involved in the approximate method of computation do not exceed a few per cent of the accurate values.

[4]Terzaghi, K. and Fröhlich, O. K., Theorie der Setzung von Tonschichten, Wien 1936, p. 105.

ELECTRO-OSMOTIC STABILIZATION OF SOILS

By Dr. Leo Casagrande, Member

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on January 10, 1951.)

Synopsis

A review of the principles and practical applications of electro-osmosis with particular attention to the stabilization of natural and excavation slopes in fine-grained soils, consisting of three parts: the first part discussing the principles of electro-osmotic flow through capillaries, the second part describing various phenomena which result when applying electro-osmosis to compressible, fine-grained materials, and the third part presenting practical applications in earthwork and foundation engineering.

Principles of Electro-Osmotic Flow Through Capillaries

The main purpose of this paper is to discuss the possible applications of a new method for stabilizing fine-grained soils in foundation construction by means of electro-osmosis. To explain the principle of this method it is necessary that I review briefly certain phenomena of physical chemistry. Since they are far afield from civil engineering, I will simplify the explanations as much as possible.

In 1807 Reuss (1)[1] discovered that if an electric potential is applied to a porous diaphragm, the water moves through the capillaries towards the cathode; and that as soon as the electric current is switched off, the flow of water stops immediately. In 1879 Helmholtz (2) explained this phenomenon and provided a mathematical basis which, after slight improvement by others, is still accepted today by most investigators as being basically correct.

In a cylindrical capillary tube filled with water we have to distinguish, according to Holmholtz, between free water and a boundary film of water immediately adjacent to the wall of the capillary as shown in Fig. 1(a). He called this boundary film "Double Layer" because he assumed the existence of opposite electric charges on two layers forming together the double layer. One part of the double

[1]Arabic numbers in parentheses refer to Bibliography at end of paper.

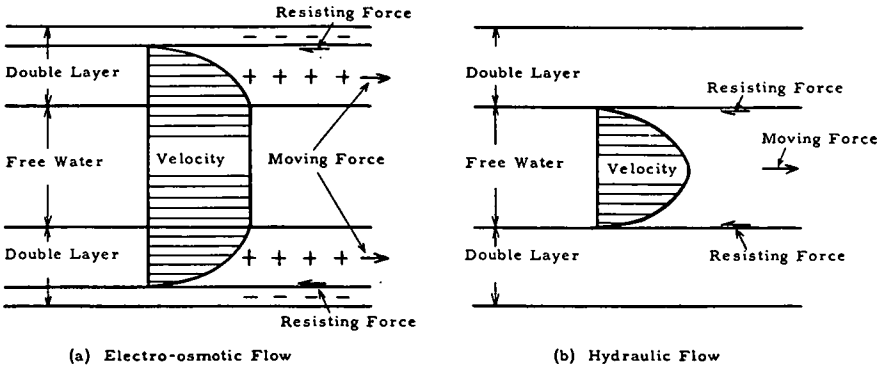


FIG. 1.—COMPARISON OF ELECTRO-OSMOTIC FLOW WITH HYDRAULIC FLOW IN A SINGLE CAPILLARY.

layer, which is very thin as compared to the total thickness of the double layer, and which normally carries negative charges, is rigidly attached to the wall. The much thicker inner part of the double layer which carries the positive charges was termed by Helmholtz the movable part; movable, because if an electric potential is applied to the capillary, the positive charges move toward the negative pole. In doing so they drag along the water molecules of the movable part of the double layer. In exceptional cases the charges are positive on the wall of the capillary and negative in the movable part of the double layer, and then the water flows in opposite direction, i.e., toward the anode.

The cylinder of free water inside the capillary, which is completely enclosed by the double layer, is dragged along by the movable part of the double layer. If we assume that no other forces are acting upon this cylinder of free water, its rate of flow will be constant and the velocity distribution will be as shown in Fig. 1(a). The phenomenon just described is known in the literature as electro-osmotic flow.

In comparison with electro-osmotic flow of water, ordinary hydraulic flow of water through fine capillaries is laminar flow with zero velocity along the wall and a parabolic velocity distribution. When referring to a zero velocity along the wall, we mean strictly speaking along the inside surface of the double layer. Since this layer, which is held immobile by molecular forces, is extremely thin in comparison to the diameter of capillaries even for very fine-grained

soils, it is permissible to say that for practical purposes the velocity is zero along the wall of the tube.

In soil mechanics, the existence of a very thin immobile layer of water next to the wall, which is held by molecular forces to the wall and which does not participate in the movement of water under a hydraulic gradient, was already recognized by Terzaghi in 1925 (3).

With these considerations in mind, the velocity distribution for hydraulic flow through a capillary is illustrated in Fig. 1(b). Strictly speaking, even this distribution is not quite correct because as a result of the movement of the water a flow of electricity is generated in the double layer, and for this reason the actual velocity distribution is modified and more complicated. However, when considering from a quantitative viewpoint hydraulic seepage through *coarse capillaries* one may well assume zero velocity within the entire double layer.

The thickness of the double layer varies with the concentration of the electrolyte. A number of investigations have been performed to determine its magnitude and I should like to mention as an example the results obtained by the French scientist Gouy (4) for various concentrations of sodium chloride.

Solution	Thickness of movable part of Double Layer		
0.1 Normal sodium chloride	0.96	$\mu\mu$ =	2 molecules of H_2O
0.001 Normal sodium chloride	9.6	$\mu\mu$ =	20 molecules of H_2O
Distilled water	one thousand	$\mu\mu$ =	2,000 molecules of H_2O

Note· One $\mu\mu$ = one millionth mm = one thousandth micron.

Nutting (5) found that the thickness of the water layer adhering to air-dried quartz and other silicates at normal atmospheric humidity amounts to approximately 40 $\mu\mu$. For glass and other silicates Parks (6) determined somewhat larger values. He found magnitudes of between 70 and 100 $\mu\mu$, depending on the test conditions.

Terzaghi (7) estimated from the interference colors the thickness of the waterfilm adhering to glass in air dried condition to be 100 $\mu\mu$.

A few simple permeability tests on rock flour convinced me of the importance of the double layer in soil mechanics. The permeability of the rock flour mixed with distilled water and using distilled water for the test, ranged between 0.2 and 0.3 $\times$ 10^{-4} cm/sec. When the

tests were repeated with a small concentration of sodium chloride, the coefficient of permeability ranged between 0.9 and 1.5×10^{-4} cm/sec, i.e., about five times as large.

Though the pore diameter of clays and colloidal materials may vary considerably depending on the type of grain structure, it is reasonable to assume that a major portion of the pores in such soils has a diameter of the order of magnitude of 100 $\mu\mu$, in other words of the order of magnitude of the thickness of the double layer.

The quantity of liquid moved in a unit of time by electro-osmosis in a single capillary can be computed from the improved Helmholtz Equation (8) as follows:

$$q_e = \frac{EDr^2\zeta}{4\eta L} \tag{1}$$

where all dimensions are in cm-g-sec system and electrostatic units, and

E—the electric potential
D—the dielectric constant of the liquid
r—the radius of the capillary
ζ—the electrokinetic potential, or zeta potential, which is defined as the potential existing between the rigid and the movable parts of the double layer
η—the viscosity of the liquid
L—the length of the capillary between the electrodes.

In analogy with the hydraulic flow we introduce for $\frac{E}{L} = i_e$ the potential gradient, and for $\frac{D\zeta}{4\pi\eta} = c_1$ (constant), and finally for $r^2\pi = a$ = cross-sectional area of capillary. Thus we obtain for electro-osmotic flow

$$\left.\begin{aligned} q_e &= c_1.i_e.a \\ v_e &= c_1.i_e \end{aligned}\right\} \tag{2}$$

as compared to

$$\left.\begin{aligned} q_h &= c_2.i_h.a^2 \\ v_h &= c_2.i_h.a \end{aligned}\right\} \tag{3}$$

in Poiseuille's law governing laminar flow.

For a bundle of N straight capillaries in the total cross-sectional

area A, and with the void ratio e, the rate of hydraulic flow, is therefore,

$$Q_h = N \cdot q_h = \frac{A}{a} \cdot \frac{e}{1+e} \cdot c_2 \cdot i_h \cdot a^2$$

$$= (a \cdot \frac{e}{1+e} \cdot c_2) \cdot i_h \cdot A = k_h \cdot i_h \cdot A \qquad (4)$$

In the final form this is the law which Darcy has established and which is known under his name. It can be seen in Equation (4) that k_h = constant x a.

The rate of electro-osmotic flow in the same bundle of straight capillaries would be

$$Q_e = N \cdot q_e = \frac{A}{a} \cdot \frac{e}{1+e} \cdot c_1 \cdot i_e\, a$$

$$= \left(\frac{e}{1+e} \cdot c_1\right) \cdot i_e \cdot A = k_e \cdot i_e \cdot A \qquad (5)$$

From this equation it follows that the electro-osmotic coefficient of permeability k_e is independent of the size of the capillaries.

The dimensions of the electro-osmotic coefficient of permeability k_e are the dimensions of $\frac{Q}{A} \cdot \frac{1}{i_e}$, i.e. $\frac{cm^3}{sec.cm^2} \cdot \frac{cm}{volt} = \frac{cm^2}{sec.volt}$.

Because the dimensions of the hydraulic coefficient of permeability k_h are $\frac{cm}{sec}$, it is more convenient to define k_e by the dimensions $\frac{cm}{sec}$ for the constant potential gradient of one volt per cm.

The following important difference between hydraulic and electro-osmotic flow through porous systems should be emphasized. While the coefficient of permeability k_h, and therefore the rate of hydraulic (laminar) flow, is proportional to the first power of the cross-sectional area *a* of the individual capillaries, the coefficient k_e and the rate of flow for electro-osmotic flow are independent of the size of the capillaries. This follows also directly from equations (2) and (3) which show that the electro-osmotic velocity in a single capillary is independent of the size of the capillary, whereas for laminar hydraulic flow the velocity is proportional to the cross-sectional area *a* in the first power.

Thus, based on the Helmholtz theory, one should expect k_e to be essentially independent of the pore size of soils, i.e., about of the same magnitude for either sands, silts or clays, provided that the zeta potential is about the same for the major portion of mineral matter in these soils. From tests on various materials I concluded already in 1946 that this paradox does actually apply to many soils possessing widely different hydraulic permeabilities. Tests conducted in subsequent years have confirmed this result for materials ranging from sands to fat clays.

As to variations in the zeta potential, it is known that it does not vary much within the ordinary range of concentration of electrolytes as found in soils. However, for high concentrations of electrolytes the zeta potential can drop to zero so that electro-osmotic flow will stop; or the charges in the double layer may become reversed so that the direction of the flow will also be reversed. The latter applies to soils and rocks consisting chiefly of calcium carbonates and to some industrial wastes. E.g., a potential gradient applied to chalk causes water to flow from the anode to the cathode.

When plotting the electro osmotic velocity against potential gradients, the results lie within a fairly narrow band as shown in Fig. 2. As can be seen in the following table, the results for clays and silts lie within an even narrower band. For all practical purposes most soils may be assumed to have a constant coefficient of electro-osmotic permeability of 0.5×10^{-4} cm/sec for a gradient of one volt per cm.

Although the technique for testing the electro-osmotic permeability of soils is far from well developed (particularly the accuracy of the above results for highly compressible materials may be open to question), it is worth noting that the tests on quartz powder which were conducted independently in three different countries, gave approximately the same results even though the crushed quartz which was used had widely different grain size distributions. The quartz powder used by Schaad and Haefeli (9) in Switzerland had a hydraulic permeability of 0.037×10^{-4} cm/sec, whereas the crushed quartz used by Bernatzik (10) in his tests in Sweden had a hydraulic permeability of 9.3×10^{-4} cm/sec, i.e. about a 250 times greater value, and nevertheless the electro-osmotic permeabilities of these two materials were about the same. The quartz powder which I used in England had a value of $k_h = 0.79 \times 10^{-4}$ cm/sec, thus falling in between the above extremes.

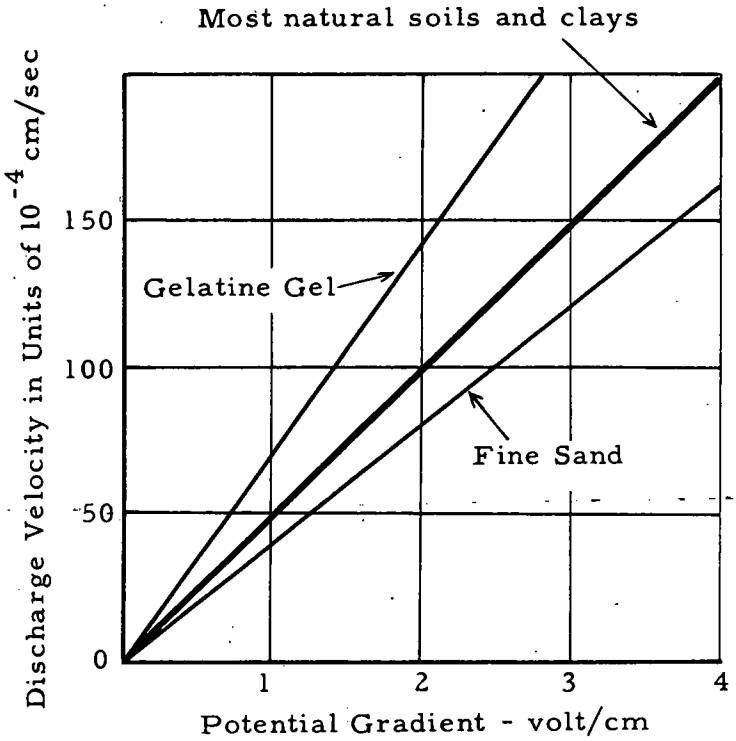


FIG. 2.—RELATION BETWEEN ELECTRO-OSMOTIC FLOW AND POTENTIAL GRADIENT FOR VARIOUS MATERIALS.

Material	Moisture Content in %	k_e in units of 10^{-4} cm/sec for one volt per cm
London Clay	52.3	0.58
Boston Blue Clay	50.8	0.51
Commercial Kaolin	67.7	0.57
Clayey Silt (England)	31.7	0.50
Rock Flour (Hartwick, N. Y.)	27.2	0.45
Red Marl (Scotland)	18.4 to 29.1	0.07 to 0.26
Na-Bentonite	170.0	0.20
Na-Bentonite	2,000.0	1.20
Mica Powder	49.7	0.69
Fine Sand	26.0	0.41
Quartz Powder (L.C.)	23.5	0.43
Quartz Powder (Schaad and Haefeli)	?	0.45
Quartz Powder (Bernatzik)	?	0.68

Let us now ask to what extent such electro-osmotic effects could be useful in foundation engineering. If the magnitude of $k_e = 0.5 \times 10^{-4}$ cm/sec for one volt/cm is compared with hydraulic permeabilities of fine-grained soils, the answer becomes immediately clear. There are excellent possibilities of moving and removing pore water in silts and clays by this method, but no advantageous results can be expected if electro-osmosis is applied to coarser materials. E.g., the application of electro-osmosis to a fine sand with a hydraulic coefficient of permeability of $k_h = 50 \times 10^{-4}$ cm/sec will not be advantageous. A discharge velocity of 0.5×10^{-4} cm/sec, as induced by a potential gradient of one volt per cm, will of course not interfere with a velocity of 50×10^{-4} cm/sec that would result from an existing hydraulic gradient of unity. This picture changes quite radically if electro-osmosis is applied to a clay which has a hydraulic coefficient of permeability of the order of 10×10^{-9} cm/sec. To compute for this clay the hydraulic gradient which would cause the same rate of flow as a potential gradient of $\bar{i}_e$ = one volt per cm, we set $\bar{i}_h \cdot k_h = \bar{i}_e \cdot k_e$ from which

$$\bar{i}_h = \bar{i}_e \cdot \frac{k_e}{k_h} \tag{6}$$

$$= \frac{0.5 \times 10^{-4}}{10 \times 10^{-9}} = 5{,}000,$$ i.e. a head of about one mile for a distance of about one foot.

Electro-Osmotic Effects Upon Compressible Materials

According to the Helmholtz theory of electro-osmotic flow through cylindrical capillaries which are open at both ends, the plug of free water is dragged along by the movable part of the double layer in such a manner that practically no stresses are developed in the free water. From considerations of the electro-kinetic forces between the positive and negative charges I was at first led to the conclusion that this flow should not develop stresses in the wall of the capillary. However, it seems that the latter question has never been specifically investigated or discussed in literature. We shall see presently that electro-osmosis does create in compressible fine-grained soils tension in the pore water, and compressive forces and a corresponding amount of consolidation in the grain skeleton, and also other phenomena which cannot be explained by the Helmholtz theory or at

least not by the simple form as it is presented and interpreted in most references on this subject.

Some investigators have pointed out the possibility that the Helmholtz theory might not be correct. Manegold and Solf (11) have made the following comment—translated: ". . . tests by various investigators on macro- and microscopic capillaries and using simple liquids, yield no disclosures which would permit a decision for or against the Helmholtz theory. On the other hand, tests on ultramicroscopic capillaries (e.g. on Collodium membranes) which should be considered particularly pertinent, have as yet hardly been evaluated because it has not been possible to eliminate the influence of the structure of the capillaries upon the electro-osmosis."

In an attempt to study the effects of electro-osmosis on compressible, fine-grained materials, I have used the following types of tests:

Type (1). The electrodes are completely surrounded by the material to be tested, or are in contact with the ends of the specimen which have no access to free water. This type of test is comparable to the conditions which apply in practical applications of electro-osmosis in earth work and foundation engineering. Such systems of flow will be referred to as *Closed Systems.*

Type (2). The electrodes are in contact with the ends of the sample to be tested, with free water accessible through the electrodes to the end faces of the sample, and with external pressure applied to the sample through the electrodes, thus combining electro-osmosis with an ordinary consolidation test.

Type (3). The electrodes are in contact with the ends of the sample to be tested, with free water accessible through the electrodes to the end faces of the sample, but without application of external pressure.

Type (4). The electrodes are separated from the end faces of the sample by space filled with water so that the specimen has entirely free ends. Therefore, this will be termed an *Open System.*

From the standpoint of practical application type (1), the *closed system,* is most important. In addition, type (4) the *open system,* is particularly interesting because of certain phenomena which are revealing in regard to the mechanism of electro-osmosis in clays. Therefore, I shall describe briefly typical results of tests of types (1) and (4).

Observations When Testing Closed Systems. A typical example of the principal effect of electro-osmosis when applied to an internally closed system of clay, is the successful consolidation of soft clay in a railway embankment in Scotland to facilitate driving a tunnel through the embankment which I will describe later more in detail. The main effect is illustrated in Fig. 19 which shows how the original water content of the clay was reduced by electro-osmosis from about 19% to an average of about 16%. Because of the two-dimensional (radial) flow condition, the fact that the water content halfway between the electrodes is higher than at the electrodes does not necessarily mean that the process of consolidation was still progressing when the water content distribution was measured. Even if it should be proven by laboratory tests using a closed system that the final equilibrium water content for one-dimensional flow approaches a constant value, it would follow theoretically that in the two-dimensional system of Fig. 19 the water content in the equilibrium condition must approach a non-uniform distribution similar to that actually measured.

Systematic laboratory tests on clay specimens, with their ends in contact with pervious electrodes, but without free water available at the electrodes, i.e. a closed system, have resulted in the following observations:

(1) Progressive decrease in water content and corresponding progressive consolidation of the clay starting at the cathode; "shrinkage cracks" and regular transverse fissures starting at the cathode. The intensity of the shrinkage cracks and the frequency of the transverse fissures (parallel to the cathodic face) increase with the potential gradient. For high potential gradients the clay develops near the cathode an intensely laminated structure. In some tests the water content at the cathodic face dropped to values as low as the shrinkage limit of the material within a short period after the start of the test, and then rose to considerably higher values.

(2) The rate of discharge of water at the cathode continuous for many weeks at an almost constant rate and finally drops off gradually, with a corresponding decrease in the electric current.

(3) For a very short period after the start of the test water discharges at the anodic face. The water content of the specimen also drops progressively near the anode, but the progress of consolidation toward the cathode is at a much smaller rate than the progress of

consolidation from the cathode toward the anode. Usually one observes a short distance from the anode, and for a limited period after the start of the test, actually an increase in water content of the clay. Irregular fissures develop at the anode which tend to be more pronounced for higher potential gradients.

(4) Tension in the pore water develops which parallels throughout the specimen approximately the progress of the decrease in water content. Unfortunately it is very difficult to measure tension in pore water in excess of about 0.9 atm. However, indirectly it can be concluded that electro-osmosis creates very high tension forces in the pore water of clays, and particularly in highly colloidal materials such as bentonite and gelatine.

(5) During a test, while the voltage is kept constant, the current at first increases slightly above the starting value, and then stays approximately constant for many weeks. Finally, the current drops off gradually. The smallest final value reached in any test was about one-fourth of the original current.

(6) Immediately at the start of the test the potential gradient is found to be fairly uniform from the anode to the cathode, but quickly it changes in such a manner that it becomes somewhat steeper near the anode and very steep near the cathode, with a flatter gradient extending over the middle portion of the specimen.

(7) Particularly striking are the phenomena observed when applying electro-osmosis to a specimen of soft gelatine. Immediately after the start of the test one observes at the cathodic end intense consolidation which becomes plainly visible by the large decrease in volume and corresponding discharge of water, and the formation of narrow folds on the surface parallel to the cathodic face which appear to correspond to the fissures observed when testing clays. After several days of continuous testing, the cross section of the surface between the anode and cathode has the appearance as shown in Fig. 3. The consolidated portion with the intensely folded surface has moved from the cathode some distance toward the anode, while the gelatine near the cathode has experienced substantial swelling and the former folding has almost disappeared. At the anodic face some consolidation has developed and a tension crack has formed a short distance from the anode, parallel to the anodic face. Some distance further toward the cathode the surface of the gelatine is raised slightly, indicating swelling. Still further toward the cathode, a portion of the

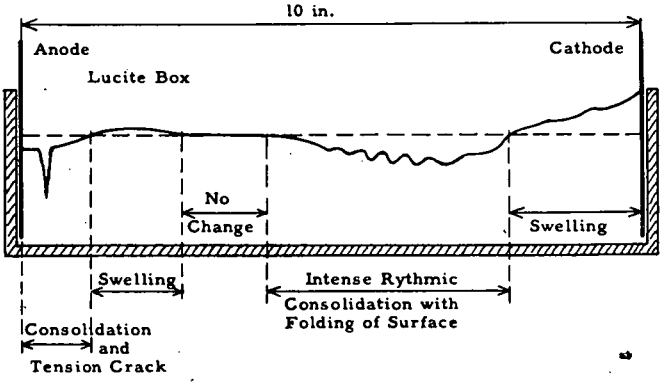


Fig. 3 Condition of Gelatine Specimen after Several Days of Testing in Closed System

surface shows no change and remains at the same elevation as at the start of the test.

(8) In contrast to the results obtained when testing clays and highly colloidal materials, such as bentonite and gelatine, no tension in the pore water could be measured when testing fine, clean sands.

Observations When Testing Open Systems. When the ends of a clay sample are free and separated from the electrodes by a space filled with water, then the phenomena that start at the cathode are very similar to those described for the closed system. However, at the anodic face, depending on the magnitude of the potential gradient, the clay either swells intensively or disintegrates by progressive slaking. The flakes of clay which drop off the face disperse completely into a cloudy clay suspension. In contrast to the instability at the anodic face, the cathodic face remains perfectly stable and the strength of the clay at that end increases because of progressive consolidation.

When testing non-cohesive materials like fine sand or rock flour, the anodic face sloughs off until it finally reaches equilibrium at a very flat slope, whereas in striking contrast the cathodic face remains standing at the vertical slope at which it was temporarily supported before applying the current. However, just as soon as the current is switched off, the cathodic face also collapses.

Working Hypothesis. Although the phenomena just described

are far from fully explored by tests and theoretical considerations, it is possible to visualize at least to some extent the reasons why compressible, very fine-grained materials must be subjected to consolidation when exposed to electro-osmosis.

The forces upon the double layer resulting from electro-osmotic flow through a capillary in an open system, as shown in Fig. 1(a), are theoretically in equilibrium. However, the open system tests on soil specimens show that the soil grains at the two faces are subject to forces which are not fully in equilibrium, such that at the anode there is a tendency for swelling or progressive dispersion of the grains, whereas at the cathodic face a force is holding the grains in place which thus has a stabilizing effect. However, these forces are believed to be too small to account for the intense consolidation and large tension forces in the pore water.

In Fig. 4(a) is shown a capillary tube which is closed at the anode with a flexible membrane. In such an arrangement there is a return flow of water in the middle of the tube which follows Poiseuille's law, and which requires a hydraulic gradient from the cathode toward the anode. Assuming at the free cathodic end the water to be at atmospheric pressure, the deviation from this pressure inside

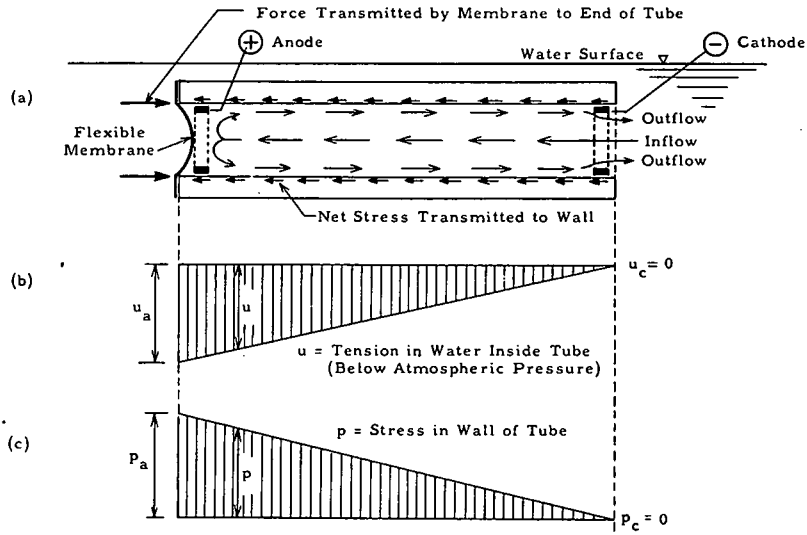


Fig. 4 Electro-Osmotic Flow and Stresses in Capillary Tube Closed at Anode (Closed System)

the tube is shown by the line in Fig. 4(b). This line starts at the cathode with the stress u_c = zero and it reaches the maximum value of u_a at the anode. The membrane at the anode is subjected to the difference in pressure between the two sides of the membrane which is u_a. The stress u_a multiplied with the cross-sectional area of the tube is transmitted through the membrane as a compressive (axial) force upon the wall of the tube at its anodic end.

If a similar tube is filled with water but is not submerged and is kept open at the anode, and if the concave meniscus which would form as a result of the tension in the water, is potentially capable of developing sufficient capillary tension to resist being pulled into the interior, the stress conditions would be the same as described above. In that case the water would actually circulate also at the cathode, whereas with the tube immersed there is probably new water being drawn in from the outside of the tube while the annular flow toward the cathode is discharging into the reservoir rather than turning around.

The force which is acting upon the wall of the tube at the anodic end through the membrane, or the meniscus, would tend to move the tube to the right in Fig. 4(a), unless there is a balancing force transmitted to the tube. Since there is no possibility of developing a resisting force at the cathodic end, because that end is entirely free, and since tests have shown that there is no indication of longitudinal movement of the capillary toward the cathode (by suspension of the tube on long threads which would reveal the slightest force), it follows that the resisting force must be transmitted to the wall through the double layer by a force which is distributed over the inside surface of the tube. That such a force should exist, follows from the following consideration.

In Fig. 5(a) is shown the velocity distribution in an open capillary which is submerged and subjected to electro-osmosis. The friction s_1 which is transmitted from the movable part of the double layer to the fixed part of the double layer is proportional to the first power of the velocity gradient, tan α_1, at that boundary. The friction s_1 is theoretically balanced by the stress p acting in the rigid part of the double layer in opposite direction. In Fig. 5(b) is shown the velocity distribution in the same capillary if the anodic end is closed, resulting in the return flow within the tube as illustrated in Fig. 4(a). This velocity distribution is the result of the superposi-

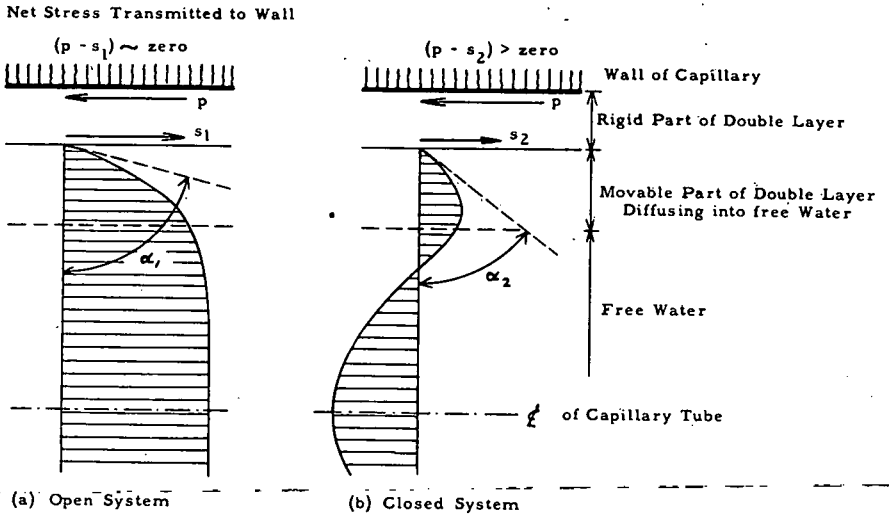


Fig. 5 Velocity Distribution in Capillary Tube

tion of the distribution shown in Fig. 5(a) and of a parabolic distribution of laminar flow in opposite direction. In this case the velocity gradient, tan α_2, at the boundary between the rigid and the movable parts of the double layer is much smaller than in Fig. 5(a), even though the same potential gradients are applied in both cases. The force p, however, is not changed in the closed system because it is only dependent on the potential gradient. Hence, the net force upon the wall (considered as a unit with the rigid part of the double layer) is equal to the difference ($p - s_2$), a stress acting in the direction toward the anode. It is not difficult to demonstrate mathematically that the total force resulting from integration of this stress over the entire inner surface of the tube must equal the force which is applied in Fig. 4(a) by the membrane, or the meniscus, to the end of the wall. The resulting distribution of compressive stress in the wall of the tube will, therefore, be as shown in Fig. 4(c).

In order to explain the progressive consolidation in the compressible capillaries of a fine-grained soil we must take into consideration that the latter differ from coarse, cylindrical and incompressible capillaries as follows:

(1) The very small average diameter of the capillaries permits the formation of very large tension in the water, and as a further consequence microscopic gas bubbles will develop by the liberation of gases dissolved in the pore water. It should be noted that in contrast to the tension that may reach many tens of atmospheres in the pore water of clays, electro-osmosis will cause in a coarse glass capillary, or in clean sand, a maximum tension u_a (Fig. 4) which is so small that it could hardly be measured.

(2) The capillaries are very irregular in shape.

(3) The walls of the capillaries are compressible and they change their length and volume corresponding to changes in the effective stresses.

(4) The empirical fact that very soon after the start of electro-osmosis the distribution of the potential gradient $\frac{dE}{dL}$ becomes non-uniform, with a maximum near the cathodic end of the test specimen, is of particular importance.

In Fig. 6(a) is demonstrated that compressive stresses can be produced by means of electro-osmosis in the walls of a microscopic capillary even without the assistance of a membrane or a meniscus. By arranging the anode halfway between the ends of the capillary, and the cathodes at the ends, the tension in the water and the compressive force in the wall of the tube will be distributed as shown in Fig. 6(b). If the anode were placed closer to one end, then for this shorter section a steeper potential gradient, and therefore steeper slopes of the stress distribution lines would result. There would be an abrupt change in the intensity of the stresses at the anode, such that the areas, i.e. the total forces, will still remain equal on both sides of the anode.

If there is gas dissolved in the water, bubbles may develop in the tube without, however, interrupting the continuity of the double layer and the movement of the electric charges.

Each section A of water between bubbles would develop its own return flow, as indicated in Fig. 6(c). The stresses in the water and in the wall along such a section will still be the same as shown in the corresponding section A in Fig. 6(b). However, within the microscopic distances of the wall where the two menisci of a bubble meet the wall, the wall itself will be subjected to tension. If the wall could not stand tension, it would crack open.

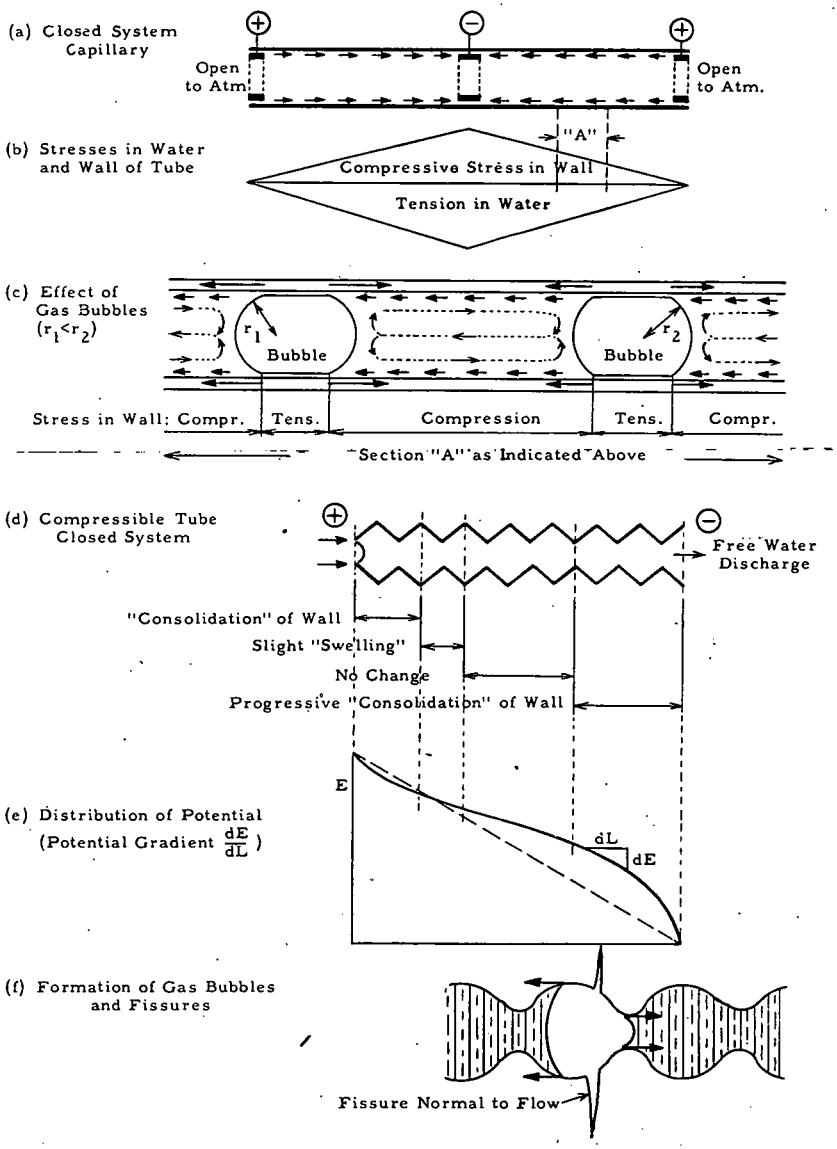


FIG. 6.—EXPLANATION OF PROGRESSIVE CONSOLIDATION IN COMPRESSIBLE CAPILLARIES.

If we now introduce into our consideration a microscopic capillary having a compressible wall, and a non-uniform distribution of the potential with the maximum gradient near the cathode, as shown in Fig. 6(d) and (e), we create a close imitation of the conditions in clay. If there is no supply of water at the anode, a concave meniscus will be formed at the anodic end because of the electro-osmotic forces which tend to move the water toward the cathode. The compressive force transmitted by the meniscus will initiate a shortening or "consolidation" of the wall of the tube in the vicinity of the anode. If the tube is very long, one may find a short distance further a portion of the tube in which there is an accumulation of water that may produce a lengthening of that portion of the wall or "swelling". Still further toward the cathode there may be a substantial section of tube where the wall will not have changed in length.

Because of the possibility of unhindered discharge of water at the cathodic end, and particularly because of the steep potential gradient near the cathode, there will be large tension forces developing in the water near the cathode which would lead to the formation of a gas bubble near the cathode, as indicated in Fig. 6(f). Once such a bubble is formed, the section of tube between the cathode and the meniscus will be subjected to substantial "consolidation". There will also be a tendency of cracking of the wall at the location of the bubble, which in the case of clay would correspond to the formation of a fissure. Very soon the concave meniscus on the left side of the fissure, which now forms the cathodic end of that portion of the tube to the left, will disappear because water will be pushed toward that face. This in turn will create sufficient tension in the water at a short distance further toward the anode to produce another bubble, and the whole process just described will be repeated.

In the case of clay, this "rythmic" repetition of fissuring parallel to the cathodic face, leads also to shrinkage of one face and swelling of the other face of the fissure, corresponding to the swelling and consolidation of the ends of a specimen subjected to electro-osmosis in an open system. Usually one observes also some water dicharging through the cracks. This rythmic development of fissures is probably responsible for the laminated structure which is observed in clay that has been subjected to electro-osmosis, particularly of the clay in the vicinity of the cathode (12). In the case of the test on gelatine, illustrated in Fig. 3, the rhythmic application of stresses to the skele-

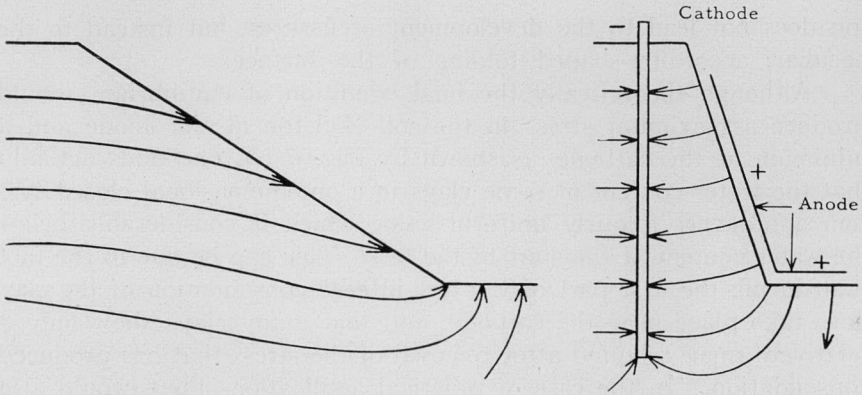
ton does not lead to the development of fissures, but instead to the peculiar, accordion-shaped folding of the surface.

Although theoretically the final condition of equilibrium should produce a maximum stress in the soil skeleton at the anode and a minimum at the cathode, as shown by Fig. 6(b), one finds actually that the water content of some clays in a one-dimensional closed system approaches a fairly uniform value which is considerably below the water content at the start of the test. This can be due to the fact that during the first part of the test intense consolidation of the clay does take place near the cathode, and that many clays show only a relatively small rebound after removal of the stress that has produced consolidation. In the case of practical applications there would also remain the stress due to the overburden which would further reduce the tendency toward swelling. And last but not least, electro-osmosis in clays cannot be carried out without getting at least some base exchange which, particularly with the use of aluminum anodes, leads to the phenomenon of irreversible consolidation (13).

Description of Practical Applications

From a practical point of view it is of great interest that by means of electro-osmosis one can move water in the pores of fine-grained soils much more effectively than by gravity, and that compressible fine-grained soils can be consolidated. Suppose a cut is made in silt or silty clay with a high water content. Even a small quantity of ground water seepage toward the cut, as shown in Fig. 7(a), may result in serious instability, even at flat slopes. By applying electro-osmosis as indicated in Fig. 7(b) one can quickly increase the effective stresses in the soil and increase its shear strength to such a degree that even a steep cut will remain stable.

Electro-osmotic control of unstable soil conditions was applied for the first time in 1939 in the excavation of a long railroad cut at Salzgitter, Germany (14) and (15). Had it not been for desperate conditions in this exceptionally long cut of clayey silt which had already been excavated to a depth of approximately 6 ft, and where a test well installation had proved that because of the fine texture of the soil wellpoints would not be effective in stabilizing the slopes of the cut, I would have hardly dared to suggest a full-scale trial section based on the small-scale experiments which I had been conducting in the preceding three years. Flow slides as shown in Fig.



(a) Hydraulic Flow (b) Electro-osmotic Flow

FIG. 7.—PORE WATER FLOW TOWARD EXCAVATION CAN BE REVERSED BY ELECTRO OSMOSIS AS INDICATED IN (b)

8, and an oozy state of the bottom of the excavation, had practically stopped further progress of work. The 300 ft long trial section was installed in a great hurry. As cathodes, 4-inch steel pipes were slotted and driven 22.5 ft into the ground at 30 ft intervals along the top of both sides of the excavation. Lengths of one-half-inch gas

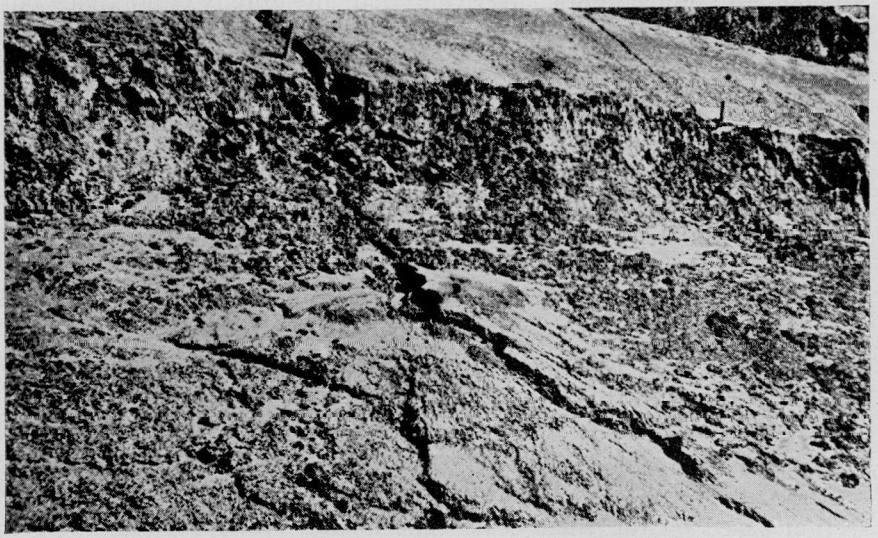


FIG. 8.—FLOW SLIDE OF SLOPES IN RAILROAD CUT AT SALZGITTER, GERMANY.

pipe were used for anodes which were driven halfway between the cathodes. The installation of the electrodes in the test section is shown in Fig. 9.

Within a few hours after application of a potential of 180 volts, the condition of the slopes improved to such an extent that excavation work could be resumed; and one or two days later even the power shovels were able to work from inside the cut as shown in Fig. 10. In this trial section slopes could be cut to any degree of steepness without the slightest indication of instability.

FIG. 9.—INSTALLATION OF ELECTRODES SPACED AT 30-FT. INTERVALS, ON BOTH SIDES OF THE CUT.

Based on the success of this trial section, electro-osmotic stabilization was applied to the entire length of this cut. For final execution of the work the potential was decreased to 90 volts. The power used amounted to approximately 1.7 kw per cathode in the beginning and dropped gradually to 1.2 kw in the course of a few weeks. Total consumption of energy per cu yd of excavation amounted to approximately one kilowatt hour.

After completion of the railway cut the bottom and the sides of

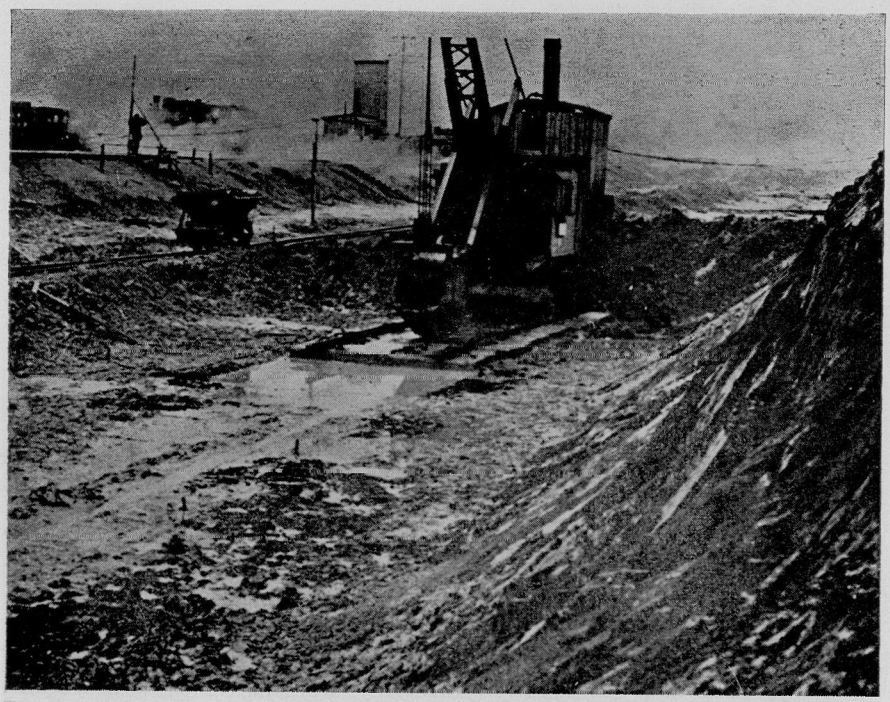


FIG. 10.—POWER SHOVEL WORKING IN CUT DURING APPLICATION OF ELECTRO-OSMOSIS

the cut were covered with a sand filter (Fig. 11) and a drainage pipe was installed along both sides of the cut so as to provide protection against damage by seepage after electro-osmosis was discontinued.

Encouraged by the favorable results on this project, the contractor decided to make use of electro-osmosis also for the foundations of an overpass bridge which had to be built across the railway. From the fact that in the trial cut the slopes stood up practically vertical to a height of 23 ft, it was concluded that excavations 20 ft in depth in similar material would stand up without sheetpiling if electro-osmosis were applied. Electrodes of similar dimensions as used for the railway cut were driven at approximate intervals of 13 ft to a depth of 25 ft, as indicated in Fig. 12, and a potential of 90 volts applied for approximately one week before the excavation was started.

The foundations were built without occurrence of any noticeable movement or flow in the sides of the excavation. The power and energy consumption were approximately 1.4 kw per cathode and 8 kwh per cu yd of excavation.

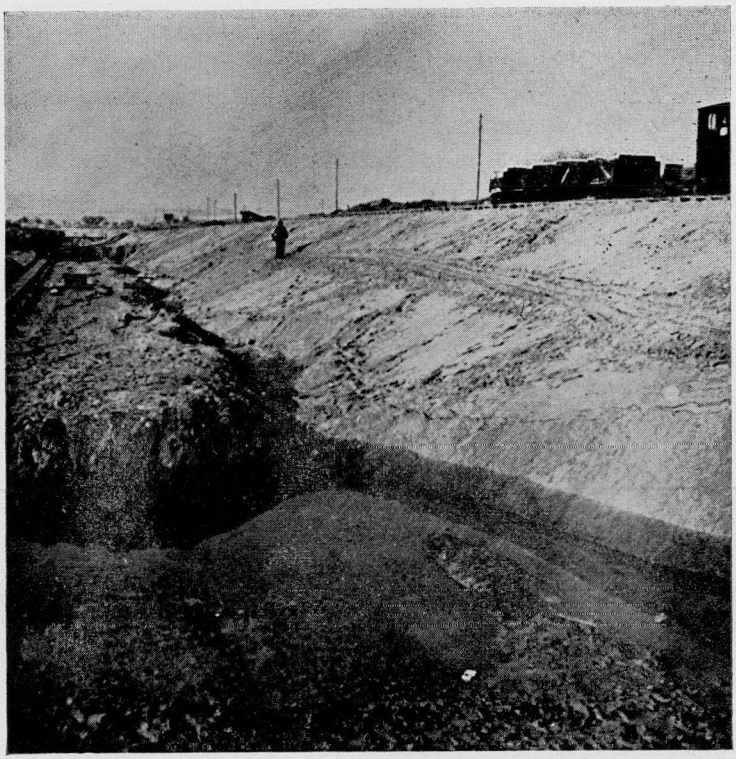


FIG. 11.—FINISHED SECTION OF CUT AFTER APPLICATION OF PROTECTIVE SAND COVER.

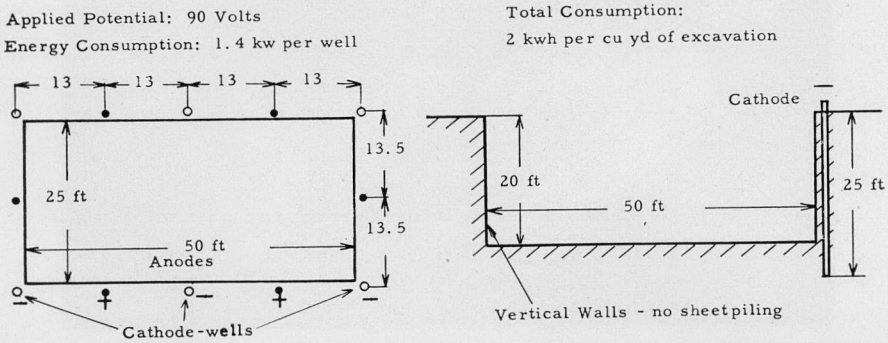


FIG. 12.—ARRANGEMENT OF ELECTRODES FOR BRIDGE FOUNDATION AT SALZGITTER.

Another large excavation which led to considerable difficulties was for the construction of a U-boat pen in Norway during the last war (15). The subsoil consisted of uniform, very silty clay to several hundred feet in depth. In undisturbed state the material was soft, and in a remolded state it became almost liquid. An aerial view of the layout of this project is seen in Fig. 13. Because of the presence of the open sea immediately adjoining the site, it was decided to surround the entire site by two rows of steel sheetpiling with embedded length of 65 ft. In addition, two berms were provided

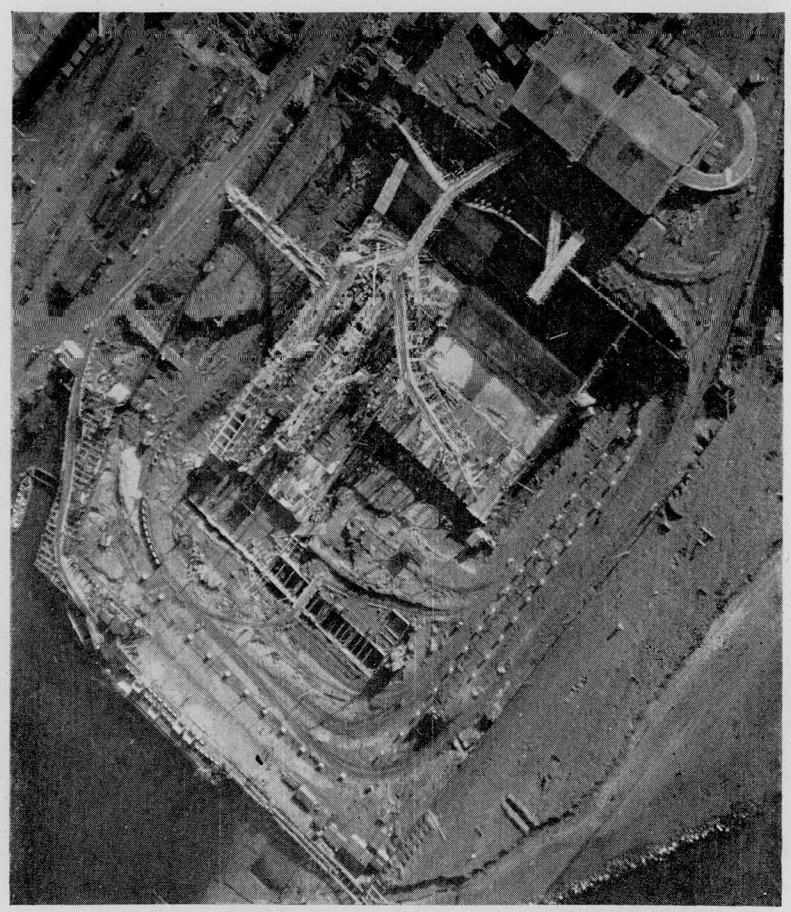


FIG. 13.—AERIAL VIEW OF EXCAVATION FOR U-BOAT PEN IN NORWAY.

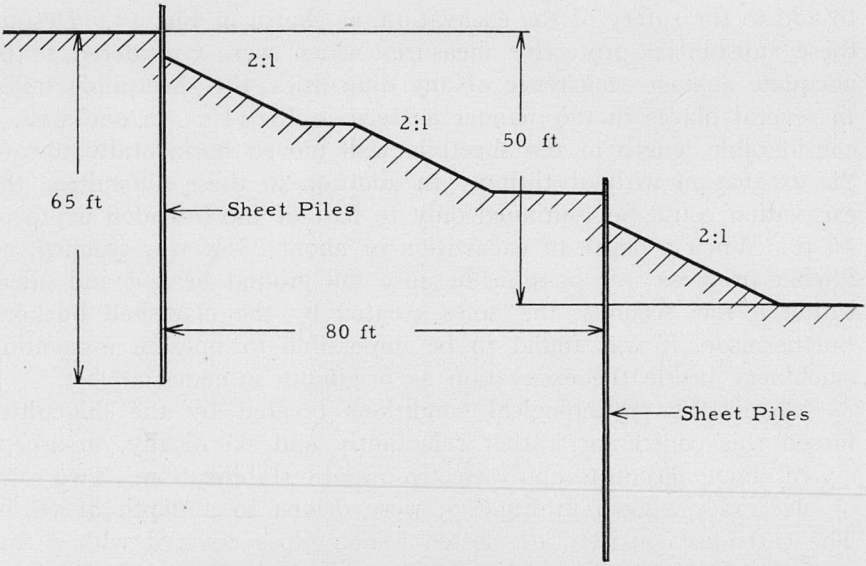


FIG. 14.—SECTION THROUGH SLOPE OF EXCAVATION FOR U-BOAT PEN IN NORWAY.

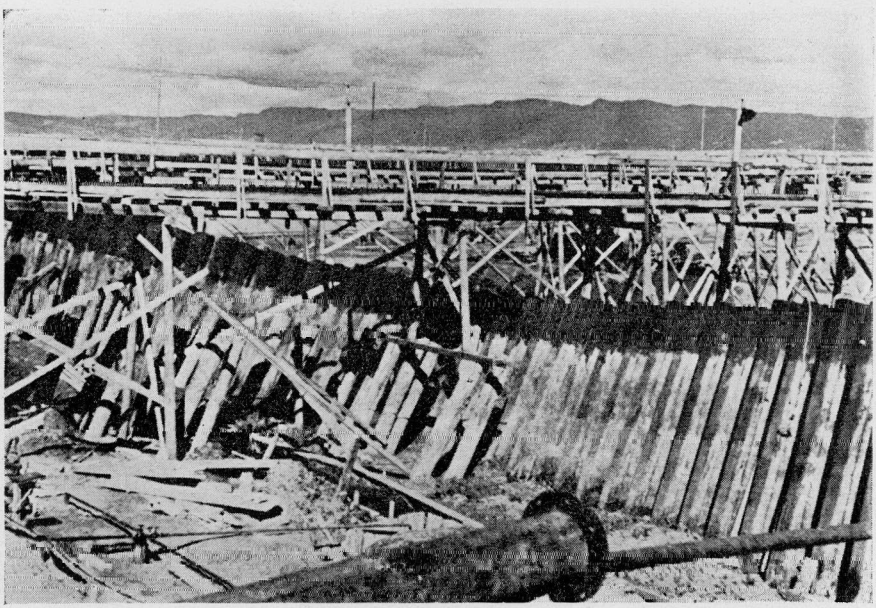


FIG. 15.—FAILURE OF SHEET PILING IN SPITE OF 65-FT. EMBEDMENT (SEE FIG. 14).

to add to the safety of the excavation, as shown in Fig. 14. Despite these substantial protective measures which were considered to be adequate against occurrence of any difficulties, the sheetpiling failed in several places in the manner as seen in Fig. 15. In one area, a considerable length of the sheetpile wall moved horizontally toward the excavation without tilting. In addition to these difficulties, the excavation could be continued only to half of the intended depth of 46 ft. When a depth of excavation of about 23 ft was reached, no further progress was possible because the ground heaved and filled, within a few seconds, the holes created by the clamshell buckets. Furthermore, it was found to be impossible to operate excavation machinery inside the excavation as originally intended.

Again the psychological conditions created by the difficulties forced this contractor, rather reluctantly and skeptically, to accept my recommendation to apply electro-osmotic stabilization. Two rows of electrodes, shown in Fig. 16, were driven to a depth of 60 ft. The cathodes consisted of slotted 8-inch pipes covered with a fine copper mesh and were spaced 30 ft apart. Common gas pipe was used for the anodes arranged halfway between the cathodes.

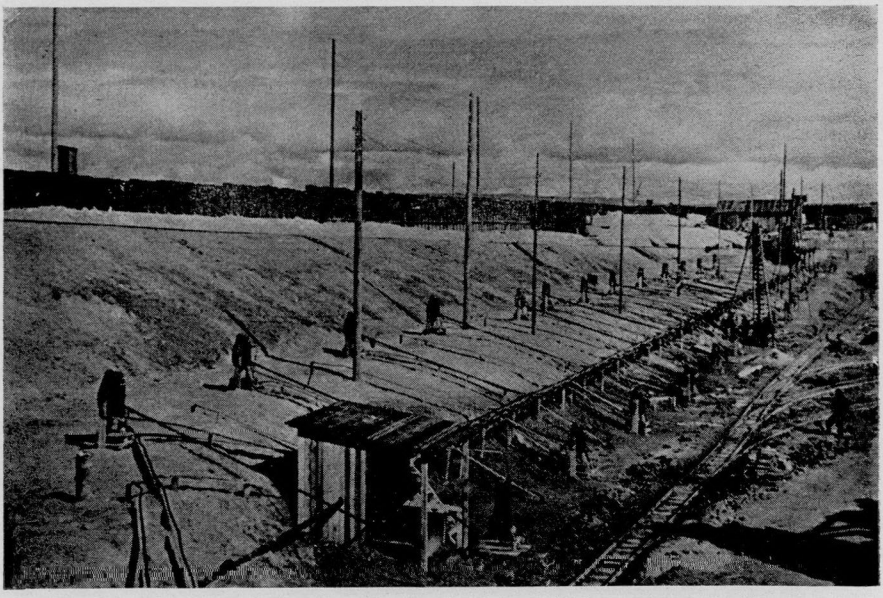


Fig. 16.—View Showing Part of Installation for Electro-Osmotic Stabilization of Excavation for U-Boat Pen in Norway.

Because the soil contained a high concentration of salts, the initial potential of 90 volts had to be decreased to 40 volts so as not to exceed the available capacity of the generator.

Two days after the potential was applied to the electrodes, it was possible to continue the excavation to full depth without any difficulties, and the excavation machinery was able to work in the excavation as shown in Fig. 17.

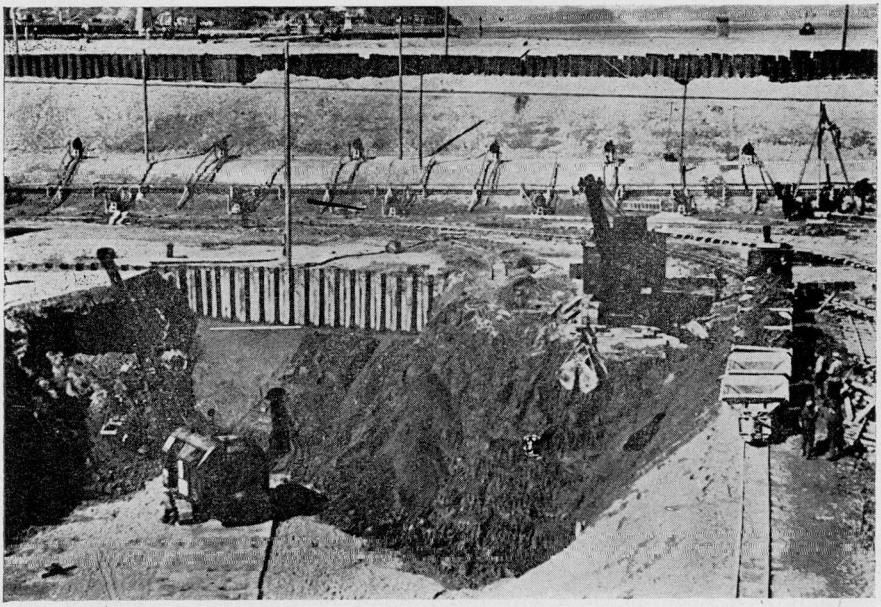
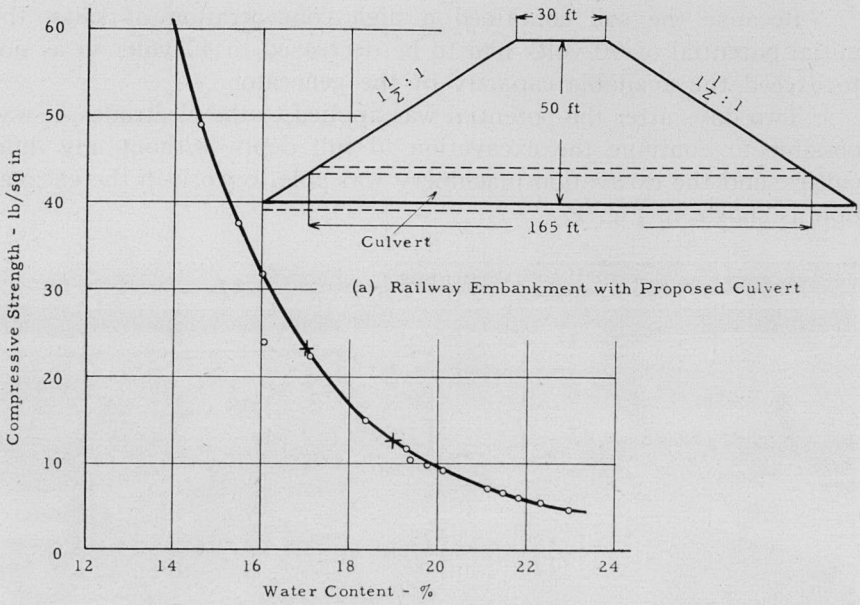


FIG. 17.—EXCAVATION CARRIED TO PROPOSED DEPTH AFTER ELECTRO-OSMOSIS WAS STARTED.

Since the total volume of excavation was large, the average total consumption of energy per cu yd amounted only to 0.4 kilowatt hour. It is of particular interest to note that during the period of approximately 6 months during which electro-osmosis was kept in operation on this project, the average water content of the affected soil decreased only about one-half per cent.

Although electro-osmosis could not be regarded as an economical proposition if applied with the intention of decreasing moisture content of a material to any considerable extent, it should be emphasized that such undertaking may still be justified in exceptional cases.



(b) Relation between Water Content and Compressive Strength

FIG. 18.—(*a*) SECTION THROUGH RAILROAD EMBANKMENT. (*b*) RELATION BETWEEN STRENGTH AND WATER CONTENT OF FILL MATERIAL.

A few years ago I was consulted by the chief engineer of the Scottish Railways who suggested to apply electro-osmosis in connection with a 50-ft railway embankment (16) as shown in Fig. 18(a) which had been washed out by a flood in form of a steep V, and replaced in great haste by very soft material. The material consisted of a red marl, a clayey material containing sand, gravel and boulders, with a liquid limit of 29 and a plastic limit of 15 for the minus No. 40 mesh material. Laboratory investigations performed by the Building Research Station at Watford, England, shown in Fig. 18(b), demonstrated that a relatively small decrease of the existing water content would increase the compressive strength considerably and thus render the embankment safe for train operation. An additional purpose of the proposed decrease in moisture content of the new fill material was to be able to tunnel through the base of the embankment without special precautions for the construction of a proposed large culvert.

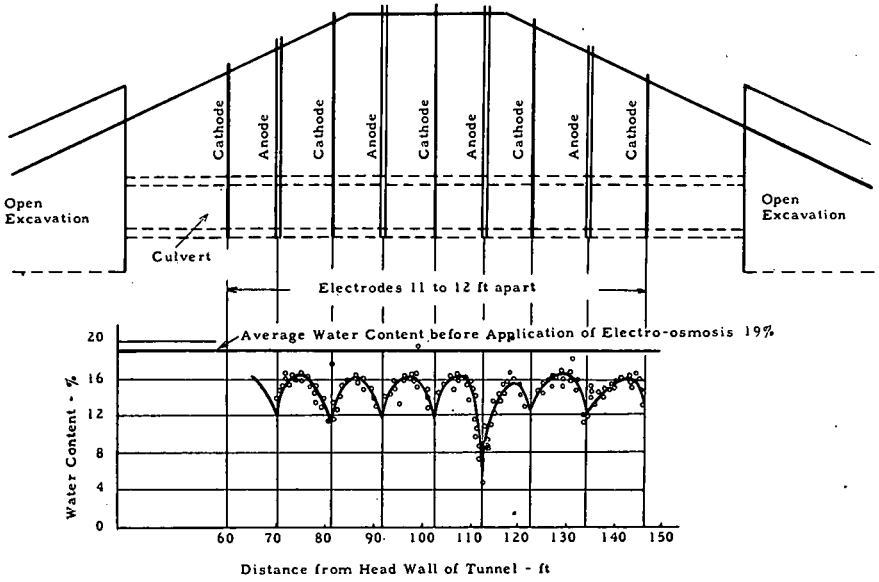


FIG. 19.—DISTRIBUTION OF WATER CONTENT BEFORE AND AFTER APPLICATION OF ELECTRO-OSMOSIS IN RELATION TO POSITION OF ELECTRODES.

Because of the presence of a considerable quantity of coarse material, the electro-osmotic permeability of this soil was below normal as shown in the preceding table for the red marl.

As indicated in Fig. 19, one row of electrodes was driven with a spacing of 11 ft, the cathodes consisting of wellpoints and the anodes of scrap sheetpiling. A potential of 90 volts was applied for a period of 6 months with a total consumption of energy of 61,000 kwh. A total quantity of 9,000 gallons of water was removed by pumping from the wellpoints. However, from the measured reduction in water content shown in Fig. 19, it was computed that the total volume of water removed must have been of the order of 30,000 gallons. The difference between these quantities is probably due to the fact that the embankment is underlain by pervious materials.

As can be seen from a comparison of the water content in the treated section of the embankment before and after 6 months of treatment, as shown in Fig. 19, the water content decreased most in the vicinity of the electrodes and least in the center between the electrodes. Incidentally, this dome-shaped distribution of the water

content between the electrodes in this large scale application agrees very well with radial flow laboratory tests which had been carried out on various materials.

While the tunnel work was in progress I inspected the site, and I had to agree with the tunnel foreman that the soil in the tunnel was much harder than would have been necessary to assure adequate safety of the construction, in contrast to the original condition of this material which would have necessitated the use of the shield method.

Electrical Installations. For anodes any scrap iron, such as old rails, may be used. If pumping of the discharging water is not contemplated, any type of scrap iron would also be suitable to serve as cathodes. In many cases ordinary pipe into which small holes are drilled, may be used for the cathodes. Usually a portion of the water finds its way to the ground surface along the outside of the pipe and the remainder discharges through the pipe itself. In most cases ordinary wellpoints, as they are used in this country, make satisfactory cathode wells.

The proper spacing of the electrodes will depend mainly on the electric potential available at the site. Spacing of between 12 and 16.5 ft and potentials varying between 30 and 180 volts, which were actually used in the few practical applications of electro-osmosis for construction purposes, have proven to be both effective and economical. Potential gradients of more than one-half volt per cm should, as a rule, not be exceeded for long term applications because higher gradients lead to energy losses in form of considerable heating of the ground. However, it might be of advantage to use potential gradients of one to two volts per cm during the first few hours, which will assure a much faster buildup of tension in the pore water, and to decrease later the gradient to a much smaller value. Furthermore, a recent study by the Bureau of Reclamation (17) has led to the suggestion of a possible economy in power consumption by intermittent operation.

Combination of Electrical Hardening and Electro-Osmosis for Stabilization of a Slide Area. Twenty years ago I discovered that it is possible to solidify irreversibly a clay by electrical treatment with aluminum anodes. I patented this invention in several countries including the United States (13). A successful application of this method, combined with electro-osmosis, was carried out on an unstable slope of sandy clay in connection with a hydro-electric develop-

ment in northern Italy (18). The electrical stabilization was achieved by treatment of the bank in two stages. First the surface of the slope was stabilized to a depth of about 8 ft by means of short aluminum electrodes and then the effect was extended to greater depth by means of an additional cathode which was placed in a bore hole to a depth of 80 ft. The average distance between the group of anodes and the cathode, consisting of iron plate, was about 90 ft. The applied electric potential was 40 volts during the first stage of treatment, and it was increased to 300 volts in the second stage of stabilization. The total energy consumption amounted to about 9,200 kilowatt hours or about 1.1 kilowatt hour per sq ft of the treated surface of the bank.

Several load tests were performed on the treated slope and on two neighbouring sections of the bank, one of which had no treatment whereas the other had a system of French drains installed in the slope. The total settlements at 5 ton/sq ft were 8.3 mm (⅜ in.) in the treated section, 21.5 mm (⅞ in.) in the section with the French drains, and 77.5 mm (3 in.) on the untreated portion of the bank. It is of interest to note that these load tests were made during a rainy period.

Closing Remarks

When I made my first experiment twenty-two years ago with passing electric current through a sample of clay, I did not suspect that this would turn into a life-time adventure which would lead me to extensive studies in a branch of physical chemistry.

Although the main purpose of this paper is to review principles and practical applications of electro-osmosis, with particular attention to the stabilization of excavations and natural slopes, I hope that the brief mention of some of the fascinating and far from clearly understood phenomena which one can observe when transmitting a direct current through compressible, fine-grained soils will stimulate colloid chemists and physicists to become interested in these phenomena. I feel certain that more systematic research in this field would result not only in worthwhile scientific progress, but would also yield rich rewards through practical applications in many branches of technology.

In passing, let me indulge for a moment in a speculation on the disputed subject of the divining rod. Schaad and Haefeli (19)

called attention to the possibility that the concentrated seepage through the voids of a soil might create a field of electric currents which might be felt by some human beings. I consider it not impossible that supersensitive electronic instruments could be developed which would detect the differences in the electric fields over an aquifer and the adjacent areas where seepage is much less or non-existent. Based on empirical correlations this might permit a rough estimate of magnitude and depth of the ground water currents. As to the possibility that at least some of the successful applications of the divining rod might be due to an extraordinary acuteness of certain senses of some human beings, it might be pertinent to recall Galvani's classical experiments in which he activated a dead frog's leg muscles by a small electric current.

Acknowledgment

I cannot list here the names of all my colleagues who, during the past twenty years, have in some form or other assisted me in the investigations of the effects of applying direct current to fine-grained soils. In the earlier period I owed much encouragement and cooperation to the late Professor Kurt Endell (Technical University in Berlin), who is well known for his pioneer work on clay minerals.

I should like to express here in more detail my gratitude to those who have made possible my investigations during the past five years: to Dr. F. M. Lea, Director of the Building Research Station in Watford, England, and my colleagues at this Station, in particular Mr. L. F. Cooling and Mr. W. H. Ward, whose spirit of friendly helpfulness has done so much to render a valuable experience the period from 1946 to 1950 during which I worked at the Building Research Station; to Professor Gordon M. Fair for permission to continue my investigations in the soil mechanics laboratory of Harvard University, and to my brother Arthur and to Professor S. D. Wilson of the Harvard University Engineering Department for their constructive critical review of this paper.

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ENGINEERING IN CONNECTION WITH TRANSPORTATION FACILITIES OF THE METROPOLITAN TRANSIT AUTHORITY, BOSTON, MASS.

By E. B. Myott*

(Presented at a meeting of the Boston Society of Civil Engineers, held on November 19, 1951.)

Transportation Services of the M.T.A.

The Metropolitan Transit Authority, a political subdivision of Massachusetts, was created by Legislative Acts to operate the facilities formerly belonging to the Boston Elevated Railway Company for furnishing transportation of fourteen municipalities of Metropolitan Boston.

The 135 square miles served directly by the Authority consist of highly developed areas having a population of 1,529,000, one-third of the total population of the Commonwealth. However, a much greater population uses the services of the Authority, including residents of twenty municipalities in the outer limits of the Metropolitan area and also transients. The services provided by the Authority are consequently of great importance to the Commonwealth, and as equally important as services provided by other state facilities, such as highway arteries, ports, airports, and metropolitan water supply, sewerage and drainage systems.

The Authority operates all types of modern transit lines over routes extremely irregular and with many complex intersections. The most important are the rapid transit lines, consisting of 10.13 miles of elevated structure, 9.43 miles of subways for train operation, 5.0 miles of subways for trolley car operation, 2.6 miles of surface lines for train operation, and 3.7 miles of surface lines including viaducts for high speed trolley car operation, a total of 30.86 miles.

The transportation system at one time included 471 miles of surface car tracks which mileage has now been reduced to 184 miles. About 29 miles of trolley car tracks, on important feeder lines are in reservations. In effect they provide rapid transit service as cars in reservations are not seriously affected by dense vehicular traffic as is the case of cars operating in roadways. Surface car lines operated in

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reservations and through subways undoubtedly will continue to provide efficient transit service for many years. It has been more economical to discontinue surface car operation in favor of buses and trackless trolleys on many surface lines which are not operated through subways.

The Authority now operates round trip distances of 402 miles of bus lines and 162 miles of trackless trolley lines. These lines, together with the surface track lines, provide complete and well distributed feeder service throughout the entire system.

The combined transportation system comprises a total mileage equivalent to single track or oneway operation of 820 miles and 463 rapid transit cars, 790 trolley cars, 370 trackless trolleys and 579 buses are now required throughout the system.

The Authority now operates about 50 million revenue vehicle miles and carries about 308 million revenue passengers annually but the facilities have a far greater capacity. The traffic varies considerably from time to time depending upon general economic conditions and in general has increased during the life of the system in proportion to the increase of population. The traffic has been seriously affected by the greatest competitor of the transit industry, the passenger automobile.

Revenue passengers during World War I were carried at the peak rate of about 380 million annually. After World War I there was a sharp decline in traffic, an increase during the following period of prosperity, then a much greater decline during the depression. During World War II, the revenue passengers carried annually reached a peak of about 433 million, and since then has declined rapidly to the present rate of about 308 million.

Since the last war there has been an extremely sharp increase in the registration of passenger automobiles and a marked decrease in the revenue passengers carried. At the present time construction is under way of needed express and other main highways which may increase the number of people traveling to and from Boston in passenger automobiles, and may result in a further decrease in the number of passengers carried by the Authority.

Considering the effect the use of passenger automobiles has on transit traffic, it is interesting to note that one line of rapid transit trains can carry easily 30,000 people per hour in one direction, while a modern two-lane express highway in a metropolitan area may carry about 7,500 people per hour in automobiles.

In addition to the large mileage of tracks, subways, tunnels and elevated structures, the Authority maintains many appurtenant facilities throughout the system, including carhouses, trackless trolley service buildings, garages, bridges, repair and other shop buildings, signal, telephone and fire alarm systems, power transmission and distribution lines, power stations, sub-stations, heating plants, subway and tunnel pumping and ventilating equipment, fare collection equipment and station structures.

Maintenance Engineering

A large force is engaged steadily on maintenance of property. The property now in use originally cost about 210 million dollars. It has been maintained in good condition and consequently long useful life still remains and the physical value of the plant is high.

Maintenance work involves many engineering problems relating to design, construction and economics. The economical amount of maintenance and repair work required to maintain the plant in good physical condition and always safe for operation, is based on detail inspections, records of operation and long range programs. It is important that maintenance policies and programs result in full protection of the capital invested in the plant. Careful consideration is always given to the functional value of every item of plant in preparing maintenance programs and to its need to meet anticipated future requirements.

A large amount of the maintenance work on elevated structures and in subways has to be done at night during a working period of 3 to 4 hours after trains have ceased operating and at a very high labor cost. It is particularly important, therefore, that engineering designs and kinds of material used for construction and maintenance be such that only a minimum amount of work be required to maintain them in operation.

Continuous engineering studies are made to improve designs, including the use of new materials, in order to increase efficiency of maintenance work and consequently reduce the cost of operation. Many of the structures including the elevated structure are about 50 years old and are reaching the age where maintenance work will consist to a considerable extent of reconstruction. Some of the buildings are large, of antiquated design for their present function, and are difficult to maintain.

As a result of lack of standardization of design and use of materials in the construction of the older parts of the system, it is necessary to store large stocks of materials for emergency replacements. When many of the tracks were first built engineers had no hesitancy in designing a special piece of track work for each individual layout.

It is necessary, where this applies on rapid transit lines, to have available for immediate use, spare parts for each special location in order to maintain continuous service. Standardization of design and use of materials is now being carried out by engineers of the Authority as rapidly as conditions warrant, giving consideration to the amount of stock required for emergency repairs, designs which are practical for simplicity of maintenance and the economical life of the structures.

Engineering for New Projects

In addition to the engineering work required for maintaining the plant in good physical and safe operating condition, a large amount of engineering work is done by forces of the Authority in connection with projects to meet changing requirements and for plant extensions.

The functions of the former Transit Department of the City of Boston, concerning transit facilities, were transferred to the Authority in August, 1949. Since then the the Authority has been responsible for the design and construction of all engineering works required including subways and tunnels.

At the present time construction is under way of the East Boston Rapid Transit Extension, which work was started by the former Transit Department.

Preliminary engineering designs are practically completed of two large subway projects and construction plans are now being prepared for another large project.

East Boston Rapid Transit Extension

The East Boston Extension, which is expected to be completed and ready for operation shortly, together with trackless trolley and bus feeder lines, will provide rapid transit service for East Boston, Chelsea and Revere, serving an area of 14 square miles and a population of about 130,000. An additional area including Winthrop and other municipalities north of Revere will be served by this extension through feeder lines of other transportation systems.

The new line, extending from Maverick Station at the terminus

of the East Boston Tunnel to the vicinity of the Suffolk Downs Race Track, a length of about 2½ miles, will have three important stations —Airport, Day Square and Orient Heights. A fourth station will be built at Suffolk Downs to be used only during the racing season.

The route of the East Boston Extension follows for a considerable distance the Old Narrow Gauge Railroad right-of-way and the line can be further extended along the right-of-way to Beachmont, Crescent Beach and the Bath House in Revere, over land owned by the Authority. The extension includes two sections of subways and inclines having a total length of 3,000 feet and 9,800 feet of surface construction. One of the subway sections extends from Maverick Square to the Airport Station and the other section underpasses a Boston and Albany Railroad siding which serves the Airport. The construction work also included high retaining walls; bridges; station structures; an interlocking tower; an electric sub-station; car inspection and repair shop; power distribution and overhead catenary feeders; lighting and signal systems; 26,000 feet of main line tracks; and a modern rapid transit storage and operating yard consisting of 22,000 feet of track. Two automibile parking areas have been built, one at Orient Heights Station having a capacity of 200 automobiles and one at Day Square Station having a capacity of 340 automobiles.

The Airport Station is situated along the elevated express highway which will extend from the McLellan Highway to Maverick Square and near elevated connections from the express highway to the Airport. It has an elevated busway joining the elevated highway connections for buses operating at the Airport. The station was designed primarily to serve the Airport but a roadway and walks are provided for serving people in the immediate vicinity.

The station will have two levels; the lobby level which is at the same elevation as the busway and the train platforms which are approximately at the elevation of ground surface. The lobby will contain fare collection equipment, service rooms and two stairways on each side extending from the lobby to the inbound and outbound platforms below. The structure has been designed so othat escalators may be installed later if the amount of traffic at the station warrants the necessary expenditure.

The Day Square Station, the most important on the Extension, is located near the crossing of Bennington Street and the express highway extending from the existing McLellan Highway to the Air-

port and Sumner Tunnel. This station also consists of two levels; the lower level with rapid transit platforms at ground surface and the upper level of two parts; one for unloading and one for loading trackless trolleys and buses. The station will have space for 12 berths for both unloading and loading vehicles. During rush hours it is expected that trackless trolleys and buses will enter and leave this station at the rate of about 65 vehicles per hour. At the southerly end of the station there is an elevated loop for turning around lanes extends from the upper level to Bennington Street. At the beginning of operation six major and other intermediate feeder lines will transfer their passengers to the rapid transit trains below at the station.

The Orient Heights Station is located on Bennington Street near the intersection of Bennington Street and Saratoga Street. Feeder lines from Revere Beach section of Revere and from Bennington Street southerly of the station, will transfer their passengers at street level immediately adjacent to Bennington Street to the inbound rapid transit platform. A busway has been provided on the easterly side for the interchange of traffic from Winthrop. The station consists of two covered rapid transit platforms with an enclosed bridge over the tracks connecting the platforms. The platforms are divided into two sections for unloading and loading in separate areas. On the Bennington Street side of the station there is space for seven berths for both unloading and loading vehicles. This station will be the present terminus of the East Boston Extension except during the racing season at Suffolk Downs, when trains will be operated to the vicinity of the race track, where it is proposed to build a station designed particularly for handling traffic during short periods.

Just beyond the Orient Heights Station, marsh land has been filled to provide a yard for the storage, inspection and repair, of rapid transit cars. The facilities located here consist of an electric substation, an interlocking tower, inspection and car repair shop and a central heating plant.

The main line tracks are joined to the yard tracks by a double track wye connection with turnouts and crossings designed to provide maximum flexibility of operation. A relay track is located immediately adjacent to the main line and along the throat of the yard where a train will be held in readiness to go in service promptly. The throat of the yard has been laid out to prevent any possibility of a "bottleneck" forming which may occur in the throat of many such yards.

Immediately adjacent to the Orient Heights Station tracks have been installed for use in turning back trains rapidly to maintain schedules of operation.

The storage yard, having a capacity of 84 cars, is double ended, with single turnout ladder tracks. The yard has sufficient length for the storage of three 4-car trains. The southerly part of the yard consists of shop receiving tracks where trains may be stored temporarily prior to being moved into the inspection and repair shop.

There are tracks at the easterly end of the inspection and repair shop, connected through a double turnout ladder track and a loop track, permitting the movement of trains from the shop to the storage yard. Two running tracks are provided in the middle of the yard connecting to the ladder tracks and to the loop. A test track with a pit for testing trains is located immediately adjacent to the inspection and car repair shop.

The inspection and car repair shop is a steel frame structure 198 feet wide and 241 feet long, designed for accommodating four-car trains. It includes a wash room on the notherly side equipped with mechanical washers of the latest improved design for the rapid washing of trains. The middle section of the shop has four inspection tracks, with inspection pits, extending the entire length of the building. for heavy repairs and painting. Hydraulic car hoists have been installed for hoisting a unit of two vehicles for replacing trucks and general repairs. Electrically operated traveling overhead cranes together with a stationary jib crane will be installed for lifting and moving trucks. Oil rooms, stock rooms and a lobby have been provided in one part of the repair shop.

All main line tracks are built of new 85 A.S.C.E. rail on tie or guard plates securely screw spiked to oak ties ballasted with broken stone. All rails on main line tangents and on curves with long radii are thermit welded, using a full fusion weld. Standard bolted joints were used for rails on sharp curves. Tie pads were used on the main line tracks under all tie plates primarily to increase the life of ties, but also to act as a cushion to provide smooth and quiet operation of trains.

The minimum width of the right of way is 35 feet, each side of which is protected with chain link fence and barbed wire.

Special consideration as given to vertical and horizontal alignment of curves, particularly runoffs at each end of the curves. The

length of runoffs and horizontal easements was determined on the basis of providing an approximate rise on the outer rail of 1¼″ per second of train speed. On curves having particularly sharp radii, the length of easement and runoffs was determined on the basis of a rise of 1″ in 40 feet. The maximum allowable speed was established at 40 M.P.H., which required a superelevation of 6 inches and an easement length of 282 feet on one of the curves.

All curves on main line tracks have 85 lb. horizontal restraining rails along the inner rail of the curve for taking the centrifugal force of cars. The restraining rail is supported on every other tie by cast steel braces, with the bases extended to serve as a tie plate for the running rail. The braces are so designed that as the head of the restraining rail wears, shims may be added to maintain required width of flangeway.

Turnouts on the main line tracks were built in accordance with latest standards used on other rapid transit lines of the Authority. The turnouts have a radius of 250 feet and a lead of 48′ 6¼″. The frogs are of solid manganese construction. Switch points are 15 feet long, with a double guarded point on the inner curve and a specially designed housing for protecting and guarding the point. All switch points are equipped with electric heaters for melting snow and ice.

The turnouts used in the yard, excepting those in the throat of the yard which are controlled by interlocked switches, are standard A.R.E.A. #5 turnouts slightly modified. The turnouts have a lead of 40′ 7⅜″ and have bolted frogs, switch points 15 feet in length with a double guarded point on the inner curve. The inner rail of the turnout is equipped with a vertical guard rail and a compromise thermit welded joint connecting the vertical rail to the horizontal restraining rail on the curves extending from the frog.

All turnouts are equipped with rigid braces and cast steel separators on each tie excepting at switch points where adjustable rail braces are used.

The signal and interlocking system has been designed for a maximum operating capacity of 40 six-car trains per hour and to fully utilize the high performance characteristics of the new light weight rapid transit cars. The signal system is designed to prevent violations of speed reduction rules at grades, curves track switches, station approaches and other potential danger spots.

Control of the speed of the trains on grades and curves is ac-

complished by the display of a restrictive warning signal, followed by one or more stop signals. Approach timer relays automatically remove the warning aspect ahead of the train if speed is proper. If speed is excessive, the signals continue to display restrictive aspects and the speed of the train must be reduced or it will be stopped automatically.

Speed control signals are also used to avoid unnecessary stopping of trains approaching stations. When a station is occupied by a train, a following train may approach at reduced speed rather than be stopped at a considerable distance away.

Trains are protected to the rear by automatic train stop mechanisms at all signals, so that in case an operator passes a red signal the train will be stopped automatically.

All track switches are securely locked while trains are approaching or moving over them. Yard and adjacent main line tracks and switches have signals for controlling two way traffic and are operated by an all-relay interlocking machine of latest design.

Facilities for Trackless Trolleys in East Boston, Chelsea and Revere

While the East Boston Rapid Transit Extension has been under construction, facilities required for trackless trolley operation in East Boston, Chelsea and Revere have been built. The conversion to rubber-tired vehicles in this area will permit abandoning 31.8 miles of trolley track, a large part of which is now in need of reconstruction and the balance of which is costly to maintain. It will also permit the use of modern economical types of vehicles to replace old fully depreciated trolley cars. The construction work has consisted of the installation of many poles and the erection of many miles of overhead wire, the construction of a building for servicing the 75 trackless trolleys which will be required and other appurtenant work.

The service building is only 140 feet long, 70 feet wide, and consists of an inspection and repair section equipped with hydraulic hoists for servicing four vehicles simultaneously; an operating section, including rooms for mechanical equipment, tires, stock and a foreman's office and a washing section equipped with mechanical washing equipment.

Trackless trolleys can be stored outside throughout the year including winter months. They require a minimum amount of in-

spection and repair work and consequently a great saving is made in the cost of structures for servicing and storing them as compared with structures required for buses. It is not practical to store buses outside during winter months. Garages for storing buses cost $10,000 to $15,000 per vehicle.

New Subway Projects in Boston

The Legislature has directed the Authority to construct three large rapid transit line projects, including:

1. The construction of a subway and other facilities, extending from the Boylston Street Station to Forest Hills and the removal of the elevated structure on Washington Street.
2. The construction of a subway from Haymarket Square to Sullivan Square and the removal of the existing elevated structure.
3. The construction of a tunnel under Beacon Hill from Scollay Square to Park Street and the enlargement of the Park Street Station and the construction of new stations at Scollay Square.

The construction of subways to replace elevated rapid transit lines at each end of the Washington Street Tunnel, as provided in the first two projects, conforms to the city planning experience in New York City, where practically all elevated structures have been removed and also to experiences in Philadelphia and Chicago, where elevated lines have lost their importance in favor of subways.

Boylston Station to Forest Hills Subway Project

The proposed subway from the Boylston Station to Forest Hills will extend from the intersection of Washington Street and Kneeland Street, across private property to the intersection of Shawmut Avenue and Dover Street, thence in Shawmut Avenue and in Washington Street to a point near Marcella Street and thence across private property to a point near Columbus Avenue. Subway stations will be built at Union Park Street near the Cathedral, and at Massachusetts Avenue. A new subway station will be built near Dudley Street or a station will be built underneath the existing Dudley Street Station.

The construction of the subway will not present any unusual engineering problems except where the line will be under eight main

line tracks of the New York, New Haven and Hartford Railroad and the Boston & Albany Railroad where it is expected that designs now being made will indicate the advisability of constructing a tunnel.

The subway in Washington Street between Dudley Street Station and Marcella Street will be built under the elevated structure requiring the construction of temporary supports for all columns and the construction of new foundations under them for supporting the elevated structure, during the construction of the subway.

It is proposed to build a surface and elevated station just westerly of Columbus Avenue to provide for the interchange of traffic with surface vehicles, supplanting the existing Egleston Square Station.

The right of way of the N. Y., N. H. & H. Railroad, southerly of Columbus Avenue, is of ample width to permit the construction of rapid transit tracks without interfering with the operations of the railroad. It is proposed to extend the railroad fill and construct open surface tracks to carry the subway trains along the railroad to avoid costly subway construction in Washington Street. A station will be built at Green Street in the general location of the abandoned railroad station. The construction in this section of the work will be comparatively simple, with the exception of a section where the new fill will be placed over the existing Stony Brook conduit, which is about 17 feet wide and 15 feet high of brick arch construction. It is proposed to install continuous precast concrete beams spanning the conduit to prevent imposing additional load on the conduit.

A typical elevated structure will be built from the railroad at Williams Street, connecting to the existing elevated structure in Washington Street, northerly of the Forest Hills Station.

Haymarket Sq. to Sullivan Sq. Subway Project

The subway project from Haymarket Square to Sullivan Square has involved detail studies of two routes. One route will extend from Haymarket Square in Haverhill Street across Causeway Street at the end of the North Station and under the Charles River to Main Street in Charlestown, thence along Main Street, underneath the existing elevated structure to an incline connecting to the existing elevated structure, just southerly of Sullivan Square. This route has not been considered a suitable route because of the great cost of supporting the elevated structure and maintaining service during the long construction period required for building the subway.

It is believed that the proposed subway should be built along an alternate route at much less cost and with better alignment. The alternate route will also extend from Haymarket Square and the North Station buildings, under the Charles River to Rutherford Avenue westerly of City Square and thence along Lawrence Street and across private property to an incline, connecting with the existing elevated structure just southerly of Sullivan Square.

The construction of the subway will involve an intricate connection to the existing subway structure at Haymarket Square and a costly subway station at North Station. A part of the station will be built under the elevated structure and surface car facilities at Causeway Street. The subway station will have connections to the existing surface car station, the elevated station on the Lechmere line and passageways to the North Station of the Boston & Maine Railroad. The subway will pass between the North Station and the Boston & Maine Industrial Building, which will require a considerable amount of shoring of these buildings, as the floor of the subway is at a depth of about 35 feet below the ground surface and the distance between the buildings is only 39 feet.

A twin tube tunnel will be built under the Charles River and in the case of the most desirable route, the tunnel will be about 1,100 feet long, having both horizontal and vertical curves. The floor of the tunnel will be about 33 feet below M.L.W at shafts at each end of the tunnel and about 49 feet below M.L.W. in the middle of the river. To avoid steep grades at the ends of the tunnel, it will be necessary to keep the roof of the tunnel as close to the bed of the river as feasible, which may require blanketing of the bottom of the river in order to maintain air pressure during construction. Each tube will be 19 feet in outside diameter and will be built with cast iron tunnel lining plates faced with concrete.

The construction of the subway from the vicinity of City Square to Sullivan Square is largely across private property with no large or heavy buildings immediately adjacent to the route. The subway will be of normal depth and no serious construction problems will be involved. The connection to the existing elevated structure will be made without interfering with traffic by sliding three spans into place at night.

The existing subways and tunnels in Boston have been built of various sizes and shapes and in some cases they are too small to pro-

vide suitable clearance for the operation of large cars and room for maintenance inspectors during train operation.

Engineers should design subways of adequate size to meet all anticipated requirements for many years in the future, as subways are extremely costly to build and can be maintained for service almost indefinitely. It is important too, that the Engineer uses plenty of imagination in determining the anticipated future requirements to be used as a basis of design.

The Authority now operates cars only 8′-7″ wide and 46′ long on the Main Line from Sullivan Square to Forest Hills, through the Washington Street Tunnel, which is the largest size car which can be operated with proper clearance for maintenance inspectors. The size of car is limited by clearances in the tunnel. If the main line is extended in the future, undoubtedly larger cars will be desirable. In making preliminary designs for the new subways at each end of the Washington Street Tunnel, careful consideration has been given to the size of structure which should be built. It was decided that the tunnels and subways should be built sufficiently large, to provide suitable clearance for maintenance men, while operating the largest size car which could be moved through the existing Washington Street Tunnel, a car 9′-6″ wide and about 60′ long. While operating cars of this size through the tunnel, there will not be room at certain locations for maintenance inspectors. It may be necessary to enlarge the tunnel where critical clearances occur. Consideration was also given to vertical clearance required for the use of pantograph current collectors on cars, as it is believed, when other extensions of this rapid transit line are made, they will be on the surface requiring the use of such current collectors instead of third rails. A size of the new subways of 13′-6″ wide and 14′-4″ high on tangents was adopted which provides for operating a car 9′-6″ wide with clearances of 2 feet on each side of the car. In addition, niches will be located in the walls at frequent intervals for the use of maintenance men while trains are passing. The width of the subways will be increased on curves to compensate for the overhang of the vehicles. Each tube of the tunnel under the Charles River will have an inside radius to the clearance line of 8′-7″.

The construction of tracks, signal equipment and other appurtenances in the subways, will conform in general to the design of such facilities described for the East Boston Rapid Transit Extension.

Park Street to Scollay Square Subway Project

The third large subway project now being designed consists of the construction of a twin tube tunnel under Beacon Hill from Park Street to Scollay Square; the enlargement of the Park Street Station and construction of new stations at Scollay Square.

Park Street Station is the most important station on the surface car subway lines, because of its location in the center of the city, and connections which can be made at this location with rapid transit trains of the Cambridge-Dorchester rapid transit line. During the rush hours the platforms are congested to the limit of their capacity and the movement of cars is materially slowed down because of a serious "bottleneck" which exists in the track work system of the Tremont Street Subway. About 240 cars per hour, during periods of maximum density of traffic, pass through the station in each direction, carrying passengers from surface car lines operating between the south and north sides of the city. A large number of passengers using the surface car lines, originate or terminate their ride at this station and a large number transfer to the Cambridge-Dorchester rapid transit line and to other surface lines, requiring large platform areas for the interchange of traffic.

Most of these surface car lines also pass through the Scollay Square Station, which is grossly inadequate for handling the traffic originating and terminating at this location, as well as the passengers who transfer to the East Boston Tunnel trains.

The existing northbound platform of the Scollay Square Station only has a length sufficient for one 3-car subway train and the southbound platform has sufficient length for only two 3-car subway trains, both of which platforms are much to short, and therefore there is a great loss of time in operating cars through this station.

There are only two existing tracks for handling the large amount of traffic between Park Street and Scollay Square, but there are four existing tracks between Boylston Station and Park Street Station, two from the Boylston Street subway and two from the Tremont Street subway and also, there are four tracks between Scollay Square and North Station.

The new subway project provides for the construction of a twin tube tunnel, to carry two additional tracks between Park Street and Scollay Square, so that four tracks can be operated between the Boylston Street Station and North Station, thereby eliminating the

existing serious "bottleneck" in the track system. Four-track operation will provide maximum flexibility and speed of operation of the subway cars through the center of the city.

The Park Street Station will be enlarged to provide wider platforms laid out for right hand loading and unloading of the modern type of P.C.C. cars now in operation. The platforms will also be lengthened, to provide space for eight 3-car trains for loading and unloading on each side of the station, which will be nearly double the car capacity of the existing platforms.

The Boylston Street southbound platform also will be extended to provide additional loading and unloading capacity at this station.

The existing Scollay Square Station after reconstruction will be used by northbound cars only. Two platforms will be provided each of lengths sufficient for two 3-car subway trains. A new southbound station will be constructed between Scollay Square and Pemberton Square with two platforms of comparable length as those of the northbound station. Entrances and exits will be provided at the new southbound station, both at Scollay and Pemberton Squares. Escalators will be provided at Pemberton Square. The existing subway, between Hanover Street and Cornhill under Scollay Square, will be finished as a concourse extending between the southbound and northbound stations.

All new construction is being designed to permit the future operation of rapid transit trains through the tunnel and stations if such is deemed desirable in the future.

The proposed twin tube tunnel is the only part of this propect which will present serious and difficult construction problems, as it is located under heavy buildings and will have to be built in such a manner that no damage will occur to the buildings during construction. The construction of the tunnel and the Scollay Stations will require considerable shoring of the existing buildings.

The construction drawings are nearly completed and subject to the approval of the National Production Authority and the availability of material, construction of this project could be started in the near future.

BASIS OF STRUCTURAL DESIGN FOR THE PROPOSED SUBWAYS FOR THE M. T. A.

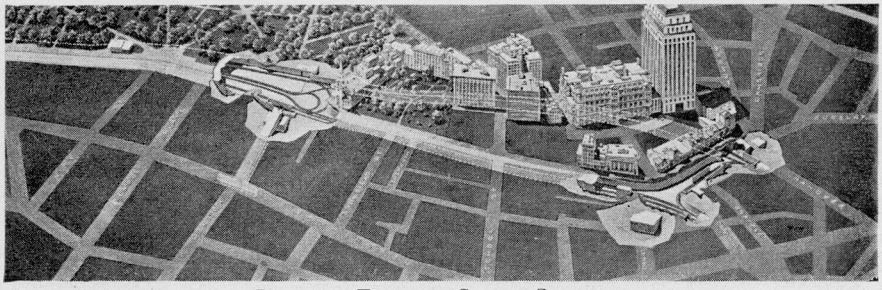
By E. H. Praeger*

(Presented at a meeting of the Boston Society of Civil Engineers, held on November 19, 1951.)

The modernization and extension of the Boston transit facilities involve a variety of interesting and important design and construction problems. In contrast with the recent subway construction in Toronto and Chicago and even those in Philadelphia and New York, where extensive new systems or parts of systems have been constructed, the work planned in Boston involves the modernization of existing old facilities using both trolleys and rapid transit trains with provision for substituting rapid transit for trolleys in the future.

The work with which our firm has been connected comprises two separate projects:

First, the improvement of the Tremont Street Subway from the Park Street Station to Scollay Square, eliminating the present bottleneck between these two locations. Trolleys now operate in this section but at some later date, conversion to rapid transit is contemplated.



Proposed Tremont Street Subway.

Second, the Boylston Street to Forest Hills project which involves the provision of a new construction to replace the Washington Street Elevated structure, partly above ground on an enbankment adjacent to the right of way of the N. Y. N. H. & H. R. R., and partly below ground as a subway in Washington and Shawmut Avenues, the total length of this section being about four and one half miles.

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This later project is somewhat similar in engineering requirements to the Haymarket Square to Sullivan Square section which was designed by the firm of Jackson & Moreland.

The Boylston Street to Forest Hills project, while presenting many interesting and somewhat difficult problems, is more or less conventional new construction. On the other hand, the Tremont Street project involves, in addition to a new shield driven tunnel, the addition of two new tracks in a subway structure which was constructed some fifty years ago and despite the fame of the New York system, the Tremont Street subway was the first constructed in this country. The widening of this old structure, planned to be done without too seriously interfering with railroad or street surface traffic, presents some very difficult problems.

For the Tremont Street project our contract called for the preparation of preliminary plans, specification and cost estimates, upon approval of which we were authorized to prepare final contract working drawings. The preliminary phase included a complete design, some two hundred drawings and a comprehensive report with outline specifications and detail cost estimates.

The first step was to secure all available plans of the original construction and with these as a basis, plans of the existing structure, the proposed alterations and the new construction were drawn with a reasonable degree of accuracy. While this work was in progress in the office, extensive field work was under way. A check survey of the existing structure was run, borings were made, existing subsurface utilities were investigated and plotted. You can well imagine the maze of underground facilities encountered under the surface and the problems which these cause in subway construction. A survey of all adjacent buildings and other structures was also made, plotted, loads computed and tabulated.

In the design office the existing structure was analyzed using both old and new design criteria. The new criteria were discussed, and agreed to by the M. T. A. and Jackson and Moreland. The new work and the alterations to the existing construction was then designed. The preliminary report included foundation and framing plans, sections and structural details, underpinning drawings, architectural drawings and record drawings of utilities and buildings. Also, adjacent property maps used for real estate appraisal, drawings of track work, signal system, power and lighting were prepared by M.T.A. engineers.

In the Tremont Street section particularly, the work had to be built mentally and on paper, step by step, from the beginning to the end, before the proposed layout could be accepted. It is, of course, of prime importance that there be no doubt that the construction can be accomplished without seriously interrupting both railway and street traffic.

For all engineering work certain standards of design are applicable. Therefore we first obtained and studied the standards used previously in Boston. The Board of Transportation in New York City is a large engineering organization which is the successor of others that have been building subways and other railroad structures for over fifty years and has developed and compiled comprehensive criteria and specifications for use of their own engineers. Then there are many other standard specifications, A.I.S.C., A.C.I. Joint Committee, A.W.S., A.R.E.A , A.A.S.H.O., A.S.T.M., City building codes and many others.

Many of these involve empirical or semi-theoretical principles, the origin of which is uncertain. In modern important work, advantage should be taken of recent research and field experience and this was done wherever possible.

An important question in subway design is whether the construction should be of structural steel and concrete or whether reinforced concrete alone should be used. In the Tremont Street section, a decision was not difficult to make in view of the fact that the existing construction, which must be joined to the new, is of steel and concrete jack arches. In new construction many factors enter into this problem in addition to a comparison of quantities. Generally, a reinforced concrete design will appear more economical but construction problems and construction costs often outweigh the apparent economy of the concrete design. In the Washington Street project, reinforced concrete was found to be more economical than steel, particularly in today's market.

In connection with steel constructions, comparative estimates of conventional riveted articulated framing and rigid frame type were made. Consideration was also given to alternates of bolted and welded connections.

While there is some apparent saving of materials with rigid frame construction, this disappears in the Tremont Street section because of lack of duplication. It would also be true in new construction such

as the Washington Street project. The same statement holds, in my opinion, for welded or high strength bolted versus riveted connections but I realize that proponents of these methods might not agree with this statement.

Unfortunately subway construction does not readily lend itself to aesthetics unless possibly we follow the example of Russia and line the stations with marble and hang murals on the walls. We need not go to these extremes but at least we should use materials which are pleasing to the eye and may be easily kept clean. Proper lighting and ventilation and reduction of noises are necessities rather than luxuries.

Provision for safely accommodating peak hour crowds on station platforms, station entrances and exists and through fare collection turnstiles is a problem that must be given thorough consideration.

Between stations adequate provisions must be made in case of accidents or fires. And, the problem of safe, rapid and adequate railroad operation is of prime importance not only to the public but also to the operator.

A few general criteria might be mentioned. Station platforms are 400 feet long. All platforms are on tangents and practically level grades. Maximum vertical grade 3%. Minimum radius of curvature 300 feet. Columns in stations, 5′-0″ from edge of platforms. Longitudinal spacing of columns in stations not less than 15′-0″. Minimum headroom in station mezzanines 8′-6″.

With reference to actual design our first consideration is that of loads, dead and various live loads.

The following are some notes copied from our preliminary report:

The dead load shall consist of the weight of the structure complete, including all material permanently fastened thereto or supported thereby.

Cover loads in these specifications is meant to be the combined weight of the roof construction (beams, slab, etc.) and the overlying earth.

Dead loads on tunnels are to be established according to local conditions. However, in no case should the tunnel be designed for less than 1,500 pounds per square foot.

Live load shall consist of any uniform or concentrated movable load which may be displaced either by reason of operation or any other cause.

The live load from sidewalks over subways shall be taken at 600 pounds per square foot.

The live load from roadways shall be computed by one of the following methods. Whichever method causes the higher stress shall be used.

a. For a cover of 2 feet the live load shall be taken as 1,300 pounds per square foot of roadway surface, decreasing by 100 pounds per square foot for each additional foot of cover up to 9 feet. At 9 feet of cover the live load becomes 600 pounds per square foot and remains at this value for all covers exceeding 9 feet.

b. The live load shall be taken as a local concentration of 200 kips on four wheels, 12 feet between axles and 6 foot gauge. Each of these wheel loads shall be considered distributed over an area of 2 feet by 2 feet on the pavement and then through the soil and roof at a slope of 1 horizontal to 2 vertical.

Where the cover over the subway roof was less than five feet, H20-S16 highway loadings were investigated. In special places where conditions warranted, some of the stated requirements were not strictly adhered to.

I might mention here that New York State has recently adopted a modified H20-S16 highway loading. They required a train of all 20 ton trucks instead of the 20 ton truck followed by a train of 15 ton trucks. This is not serious in short span structures but it can almost double the live load in long span structures.

Train loads and trolley loads, similar to the usual axle loading but selected to meet actual local operating conditions, were used wherever such loadings were present.

Loads of adjacent or superimposed buildings were also provided for. In places where it might be possible that a new building could be constructed over the roof of the subway, particularly on vacant ground, a hypothetical building of probable maximum height was designed and provision made for such loading.

The lateral earth pressure was determined from present theories of soil mechanics, always providing for the maximum possible conditions.

We were asked to give consideration to the possible use of these structures as bomb shelters. Any underground structure affords a certain amount of protection but the difficulty is knowing what we shall protect against. During World War II high explosive bombs

penetrated some 24 feet thick heavily reinforced concrete submarine pen roofs. The Nagaski bomb, releasing energy equivalent to 20,000 tons of T N T was more powerful than the Hiroshima bomb. These were detonated in the air. Does Russia have earth penetrating atomic bombs or will we have the nightmare of hydrogen bombs, in addition to all of our other worries?

While it is possible to design a structure that will not collapse even at ground zero under a 20,000 ton type air burst atomic bomb, the cost, except in deep structures such as tunnels, would be prohibitive. We did, however, analyze parts of the structure for resisting air burst 20,000 ton bombs at varying distances from zero. The load at close distances, in addition to being of an extremely high magnitude is transitory rather than static and involves principles of vibration.

While a structure which is extremely massive and stiff will resist destruction, flexibility is a desirable feature. In analyzing a structure to withstand a single burst, we work in the plastic rather than the elastic range of the material.

To get back to what I hope are more realistic considerations than the atomic bomb, I will discuss a few problems, first of the Tremont Street project and then of the Washington Street section. Despite our efforts to produce a balanced completed facility, it was not possible to alter the existing old structure to meet all of the present day criteria. It was decided however to design the new work according to modern standards rather than to those of some 50 years ago. This does not imply that the new section is wasteful of materials nor that the old structure is unsafe, but while they are, admittedly, not equal the difference is not great.

Wherever possible, the old members were used and new members will be connected to them. This will require the removal of concrete and the local strengthening of old members at connections and other locations where the new framing conditions produce greater than allowable safe stresses.

The connections between the existing members in the Tremont Street section presents a very difficult problem and while we have surveyed existing conditions, many members are encased in concrete and therefore not accessible. Because of this, provision must be made for field fitting in many instances.

With references to the Washington Street project, there are

several interesting problems. A new section of elevated structure will be constructed from a point near the Forest Hills Station to a new earth embankment adjacent to the right of way of the N. Y. N. H. & H. R. R.

The open cut method of construction is contemplated for both the Tremont Street and Washington Street sections and the excavation will be decked over. Steel solder beams will probably be used to resist the pressure of the adjacent earth. It is fortunate that most of the open cut work in the Tremont Street section occurs in the Common.

While the Washington Street section is of reinforced concrete, a combination of structural steel and reinforced concrete is contemplated for the stations.

Time does not permit a discussion of the problems connected with the traffic interchanges at Columbus Avenue and at Dudley Street nor a description of a very excellent alternate route at Dudley Street proposed by Mr. Myott. Several alternate schemes were studied for traffic interchange all of which required an analysis of bus, trackless trolley, rapid transit, vehicle and passenger traffic.

We have a very difficult problem where the subway extends under the combined rights of ways of the N. Y. N. H. & H. R. R. and the B. & A. at Washington Street near the north end of this section. Alternate designs have been prepared for a shield driven tunnel and an open cut subway at this location. Our estimates indicate that the open cut method is the more economical but possibly more difficult to construct. This work must be done with minimum interference with railroad traffic and the railroad engineers have studied our various proposed methods and have been very cooperative. It is anticipated that alternate bids will be asked for both tunnel and open cut methods.

Our final plans of the Tremont Street section and preliminary plans of the Washington Street project are nearing completion and I hope that world conditions will permit the construction of these important improvements in the not too distant future.

PREVENTIVE MAINTENANCE

By John G. Ward*

Preventive Maintenance is insurance against costly breakdowns. All equipment needs maintenance and good maintenance involves proper kowledge, tools, and parts for its proper performance. Preventive maintenance means that equipment must have reasonable attention at regular intervals. (Care of a family automobile is a familiar example of preventive maintenance.)

Instruction Manual

The first step toward an understanding of the construction, operation and adjustment of equipment is a study of the instruction manual furnished with the equipment. After new equipment has been installed, the operator should obtain the instruction manual from the contractor, or if the installation is made before the operator is employed, he should locate the manual as quickly as possible. In many cases, instruction manuals and installation drawings, are handed over, by the contractor, to persons not concerned with actual maintenance and operation. If the plant is municipally owned, the operator should check to see if the instruction manuals are at City Hall; in industrial plants he should check with supervisory personnel.

When manuals can not be found, the operator should write to the manufacturer, giving complete nameplate information and requesting a manual, which will be provided at little or no charge.

Once the manual has been obtained, the operator should study it carefully, underlining points considered most important. Marginal notes are also helpful. Until operations are fully understood, the instruction manual should be followed exactly, when adjustments are being made. If there are any questions, the operator should write the manufacturer, who will be only too glad to answer questions and assist in any possible manner.

If an operator observes the installation of equipment, he should assure himself that the installation has been made in accordance with the manufacturer's instructions.

*Service Manager, Builders-Providence, Inc., Providence, R. I.

Manuals should be indexed and filed so that anyone can identify and readily find the manual desired.

Tools

Special tools, which may be required for maintenance operations, generally are furnished with the equipment. They should be obtained from the contractor, upon completion of the installation of the equipment. If these tools have been mislaid, new ones should be obtained from the manufacturer. They are not costly, but are invaluable, as most of them are especially suited for the adjustments to be made.

Tools should be located in a specific place, preferably in a wall cabinet, handy to the equipment. Each tool should be labelled as to the equipment for which it is to be used. In large plants, it may be preferable to keep all tools in a centrally located cabinet, under the responsibility of a specific person.

Spare Parts

Spare parts are always important; they are doubly so in times of shortage of critical materials. A certain number of parts should always be on hand to insure prompt repairs and continuous operation of equipment. The number and kind of parts to be stocked depends on the equipment and can best be determined from a careful study of the instruction manual.

Manufacturers always make the fastest possible shipment of orders for replacement parts, but prompt shipment of parts made of critical materials may not always be possible. This situation may cause a shutdown; another reason for having the necessary parts on hand, when needed.

Spare parts recommended by the manufacturer will usually be listed in the instruction manual, or the list may be obtained from the manufacturer. This list represents the minimum number of spare parts which should be stocked.

Where more than one unit of a kind exists in a plant, it may not be necessary to carry more than one set of spare parts; or the number of any particular parts to be carried as spares may be designated according to the number of equipment units. Thus, an operator can be reasonably sure of good protection for a small investment.

Spare parts should be labelled carefully and kept with the special maintenance tools. If lubricants, recommended by the manufacturer, are not obtainable locally, write the manufacturer for a supply, or for the name of a substitute which can be obtained locally.

Preventive Maintenance Schedule

Having the knowledge, tools, and necessary parts, the next step is to establish a preventive maintenance schedule, and if recommended procedures are not in the instruction manual, write the manufacturer.

A sound maintenance schedule will include arrangements for a regular visit from a factory service man, once or twice a year. The service man will disassemble, clean, lubricate, and readjust the equipment. He will also: Anticipate trouble and replace any parts which show signs of wear, but have not failed; replace obsolete parts with ones of newer design; assist in setting up maintenance schedules and in the instruction of plant personnel in the proper methods of maintenance, repair and adjustment.

Maintenance Records

For best maintenance results, a card file (or book) should be set up, with a card for each piece of equipment. The file should be so arranged as to bring the required maintenance operation to the attention of the operator on the date established for that operation.

Each record card should show the following:

1. Type of equipment
2. Manufacturer's serial number
3. Date installed
4. Location of equipment
5. Calendar scheduling of maintenance operations
6. Notes on trouble shooting (transferred from the instruction manual)
7. Lubricants recommended by the manufacturer

Each record card should have blank spaces for:

1. Date maintenance performed
2. Initials of maintenance man
3. Unusual maintenance operations for specific trouble
4. Parts, emergency calls, and costs of each

This latter information simplifies the job for the next person, anticipates further parts requirements, and indicates the desirability of replacing obsolete or worn out equipment.

Preventive Maintenance Operations

Safeguard the equipment against possible damage by operators, passers-by, or injury from any cause, whatsoever.

Operate standby equipment (or equipment infrequently used) at regular intervals.

Inspect the condition of equipment, and observe its efficiency of operation and accuracy of performance, at regular intervals.

Clean the equipment regularly, using the correct method, equipment and cleaner, as indicated in the instruction manual.

Lubricate equipment with the proper kind and correct quantity or lubricant at the proper intervals.

Replace all parts showing wear which will interfere with accurate performance of the equipment.

Paint exposed surfaces for preservation from corrosion, erosion, etc.

Overhaul at prescribed intervals, by disassembly, cleaning, lubrication, replacement of worn parts, painting, reassembly, and testing.

Subscribe to a manufacturers service plan.

Special Precautions

While the above recommendations are general and will apply to all types of equipment, specific equipment requires specific variations of the details of application of these rules. Equipment should be handled with the respect due it, without abuse and without neglect. Treat it properly and it will give long life and satisfactory performance.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

OCTOBER 17, 1951.—A Joint Meeting of the Boston Society of Civil Engineers and of the Northeastern Section of the American Society of Civil Engineers was held this date at Northeastern University, Boston, Mass. Members of Student Chapters and Civil Engineering students of the New England Colleges were especially urged to attend.

A catered dinner was held in University Commons Hall, Northeastern University from 6:00 to 7:00 P.M. Student delegations were present from Northeastern University, Massachusetts Institute of Technology, Tufts College, Rhode Island State College, University of Massachusetts, Brown University and University of New Hampshire.

President Wilbur called the meeting to order at 7:15 P.M., and extended a cordial welcome to the students and expressed appreciation of the coöperation of the officers of the student organizations and of the faculty members in making this event so successful.

Secretary Robert W. Muir announced that the following had been elected to membership:

JUNE 20, 1051—*Grade of Member*—Frank Holzer.

OCTOBER 15, 1951—*Grade of Member*—Richard W. Albrecht, William J. Faulkner, John X. Foley, Clarence H. Pratt, Leo Marc G. Wolman.

Grade of Junior—Frank T. Smith, Jr.

President Wilbur called upon Secretary Robert W. Moir to read the names of applicants for membership.

President Wilbur introduced Howard J. Williams, President of Northeastern Section, A.S.C.E., and asked him to conduct any matters of business required by that Society.

President Wilbur then presented a quartet of Northeastern Students (Bob McCauley, Dick Mudgett, Walter Arvidson, Dick Emerzian) who sang five numbers, including two encores, to the delight of all present.

President Wilbur then introduced the speaker of the evening, Dr. D. B. Steinman, Consulting Engineer, New York, who gave a most interesting illustrated talk on "Romance of Bridges".

After a short discussion the meeting was adjourned. A tremendous and spontaneous standing ovation was accorded to Dr. Steinman.

Three hundred thirty-seven members and guests attended the dinner and four hundred five attended the meeting.

The meeting adjourned at 9:10 P.M.

ROBERT W. MOIR, *Secretary*

NOVEMBER 19, 1951.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts and Sciences, 28 Newbury Street, Boston, Mass., and was called to order by President John B. Wilbur. at 7:05 P.M.

President Wilbur announced that the reading of the minutes of the previous meeting (October 17, 1951) would be waived unless there was objection.

It was *VOTED* "to dispense with the reading of the minutes of the October 17, 1951 meeting".

President Wilbur announced the death of the following members:

Samuel P. Coffin, who was elected a member February 16, 1910, and who died November 2, 1951.

William T. Morrissey, who was elected a member March 17, 1937, and who died November 6, 1951.

Secretary Robert W. Moir announced that the following had been elected to membership:

Grade of Member—George M. Castaldo, Paul O'Connell, Eudell G. Whitten.

Grade of Junior— Stanley B. Goldberg, Salvatore A. Martorana, Robert B. Nunley, Frederick Roach, Jr., Charles E. Serpis.

President Wilbur called upon Secretary Robert W. Moir, to read the names of applicants for membership.

President Wilbur announced that the December meeting of the Society would be a Joint Meeting with the Structural Section, to be held on December 12, 1951 (Dr. Karl Terzaghi to be the speaker).

President Wilbur then introduced the speakers of the evening:

E. B. Myott, Supt. of Engineering and Maintenance of M.T.A. Subject—"Engineering in Connection With Transportation Facilities of M.T.A."

Ole Singstad, Partner of Singstad and Baillie, Consulting Engineers of New York City. Subject—"Design and Construction Methods of the Proposed Tunnel between Park Street and Scollay Square for the M.T.A."

Emil H. Praeger, Partner of Prager-Maguire, Consulting Engineers of Boston, Providence and New York City. Subject—"Basis of Structural Design for Proposed Subways for the M.T.A."

A short discussion period followed after which members gathered in the Lounge where a collation was served.

One hundred fifty-six members and guests attended the meeting. The meeting adjourned at 9:25 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

JUNE 2, 1951.—Some forty-five members of the Boston Society of Civil Engineers met at the recently constructed sewage disposal plant at Nut Island in Quincy, Massachusetts. A brief discussion was arranged through the courtesy of Mr. Frederick W. Gow, Chief Engineer of the Construction Division and Mr. T. A. Gerrigan, Chief Sewerage Engineer, respectively, of the Metropolitan District Commission. Mr. Karl R. Kennison who was present kindly consented to explain the plant design. Groups were guided through the new plant by guides supplied by Mr. Joseph Hanlon, Superintendent of the Sewage Treatment Plant. Mr. Anthony O'Malley of the Metropolitan District Commission, Construction Division, together with Mr. Joseph Hanlon, Superintendent of the Sewage Treatment Plant, gave every possible coöperation to make this inspection a complete success.

Luncheon was held at the Fox and Hounds Restaurant in Quincy where twenty-five members were present. The ideal weather helped to make this interesting trip very pleasant and enjoyable.

ABRAHAM C. BOLDE, *Clerk*

OCTOBER 9, 1951.—A meeting of the Sanitary Section was held at 7:45 P.M. at the Society Rooms after an informal dinner at Patten's Restaurant at which fifteen members and guests attended.

The meeting was conducted by Chairman William E. Stanley after a report was read by the Clerk of the Section relative to the inspection trip which took place on June 2, 1951. Mr. John Hitchcock then read a paper illustrated by many slides describing sewage disposal plants visited by him in England and Holland. After an informal discussion, the meeting was adjourned at 9 P.M.

A. C. BOLDE, *Clerk*

STRUCTURAL SECTION

OCTOBER 10, 1951.—A joint meeting of the Structural and Transportation Sections was held in the Society Rooms with Chairman Ernest L. Spencer of the Transportation Section presiding.

The speaker was Mr. Eric Reeves, Senior Structural Engineer of Maguire-Fay Spofford whose subject was "Structural Design Problems on the John F. Fitzgerald Expressway."

Mr. Reeves first outlined the need for this important project and then described the overall plan for the Expressway. After discussing the economic studies of the type and spacing of the bents, he described the architectural motif and the subsurface conditions and their effect on the design. There are numerous important and unusual structural details involved in this structure, such as the connections of the deep girders to the bents, and these Mr. Reeves discussed in some detail. He concluded by describing the safety devices which have been provided in the design.

In the discussion period, Mr. Delano emphasized the design difficulties which were caused by the existing buildings and utilities. Dr. Wilbur also discussed this aspect of the problem in addition to commenting on the bridge which is to carry the Expressway across the Charles River.

The attendance was 76.

J. M. BIGGS, *Acting Clerk*

HYDRAULICS SECTION

NOVEMBER 7, 1951.—A meeting of the Hydraulics Section was held in the Society rooms.

The meeting was called to order by the Chairman of the Section, Mr. Gardner K. Wood. The minutes of the previous meeting were read and declared approved as read by the Chairman.

The announcement was made that the next meeting, February 20, 1952, would be a joint meeting with the main Society and that the speaker would be Major General Pick who would speak on the subject of the Missouri River Basin.

The Nominating Committee was elected consisting of the latest three past chairmen. These are John G. W. Thomas, James F. Brittain, Elliot F. Childs.

The Chairman introduced the first speaker, Henry M. Paynter, Professor of Hydraulic Engineering at M.I.T., who spoke on "Unsteady Flow Studies" underway at the Institute. Professor Paynter introduced the second speaker, Guiseppe Evangelisti, Professor of Hydraulic Structures and Mathematics at the University of Bologna. Professor Evangelisti spoke on recent unsteady flow studies in Italy with special emphasis on the research on the problem of surge-tank stability.

The meeting adjourned at 9:40 P.M.

L. W. RYDER, Clerk

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The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

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BOSTON SOCIETY OF CIVIL ENGINEERS

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