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RECENT DEVELOPMENTS IN SEDIMENT TRANSPORT MECHANICS

BY JOHN F. KENNEDY*

(Presented as one of The John R. Freeman Lectures on Fundamental Hydraulic Processes in Water Resources Engineering, Fall, 1963.)

1. INTRODUCTION

1.1 *Scope*

The field of sediment transport includes flow in alluvial channels and channel stability, coastal processes and sediment transport by waves, river morphology, sediment transport in closed conduits, and aeolian transport. It is not possible in the allotted space and time to summarize all of these topics in any meaningful way. Therefore, the scope of this treatment will be limited to topics related to flow in alluvial channels, including properties of sediments, initiation of sediment motion, channel roughness, and methods of determining rates of sediment transport. It is this area of sediment transport that is generally of greatest interest to hydraulic engineers.

1.2 *Features of Flow in Alluvial Channels*

In the last hundred years, the state of knowledge of flow in rigid-boundary conduits and open channels has improved until it is now possible to predict for these channels the depth and velocity of flow, velocity distribution, slope of the energy gradient, and other flow properties with accuracy that is usually adequate for engineering purposes. Such is not the case for flow in channels whose boundaries are composed of materials transported in significant quantities by the

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flow. The difficulty is that it is the flow itself that determines the channel geometry, both the gross dimensions of the channel (cross-section shape, slope, and alignment) and the roughness texture of the channel surface, by deforming the bed into ripples, dunes, sand bars, etc. In addition, the moving fluid is a mixture of a liquid and a solid, and does not behave in exactly the same way or exhibit the same properties as the same fluid without suspended sediment. Thus, the situation is significantly more complicated than the relatively simple case of flow of a homogeneous fluid in a rigid-boundary channel.

2. PROPERTIES OF SEDIMENTS

Just as it is necessary to know the properties of a fluid before its motion can be formulated, so also it is necessary to know the appropriate properties of transported sediment before sediment transport phenomena can be examined.

It is generally agreed that the grain size of the sediment particles and the fall velocity of individual particles in the transporting medium are the most important properties affecting sediment transportability. Actually, these two properties are not wholly independent; for a given particle size, shape, and weight and specified fluid properties, the fall velocity can be determined.

Measures of grain size that are in common use are:

a. Sieve diameter: The size of the opening of a square mesh that will just allow the particle to pass.
b. Sedimentation diameter: Diameter of a sphere with the same density and settling velocity in the same fluid at the same temperature.
c. Fall diameter: The sedimentation diameter for water at 24°C.
d. Nominal diameter: Diameter of a sphere of equal volume.

Natural sediments contain a range of particle sizes; hence it is necessary to speak of the frequency distribution of grain sizes. The grain diameters are usually measured by sieving the particles through a nest of calibrated, graduated sieves. The frequency distribution is obtained by plotting the logarithm of the grain size against the percent (by weight) of the sample smaller than any grain size, as shown in Fig. 1. The median diameter, D_g, is the diameter such that 50% of the sample has a smaller diameter, and the geometric standard deviation is defined as $\frac{1}{2}\left(\frac{D_{50}}{D_{15.9}} + \frac{D_{84.1}}{D_{50}}\right)$, where $D_{15.9}$ is the di-

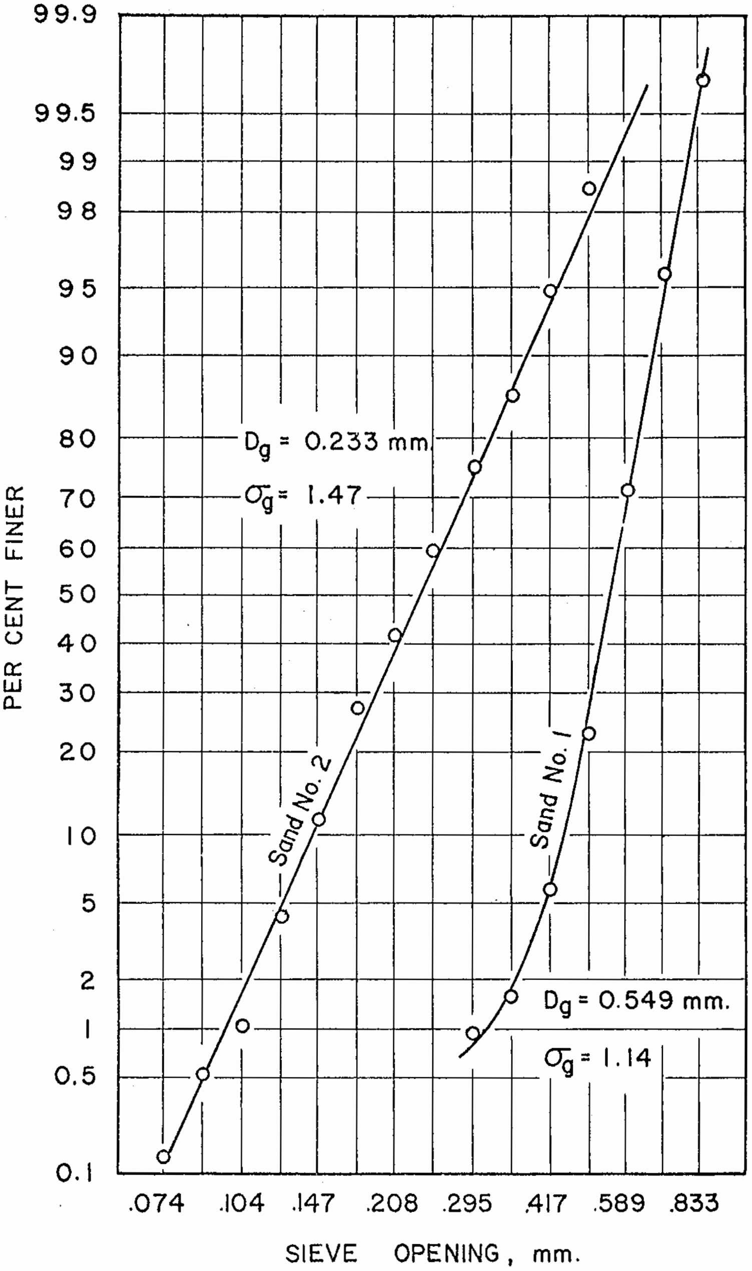


Fig. 1 Examples of cumulative distributions of sieve diameters plotted on log-probability paper.

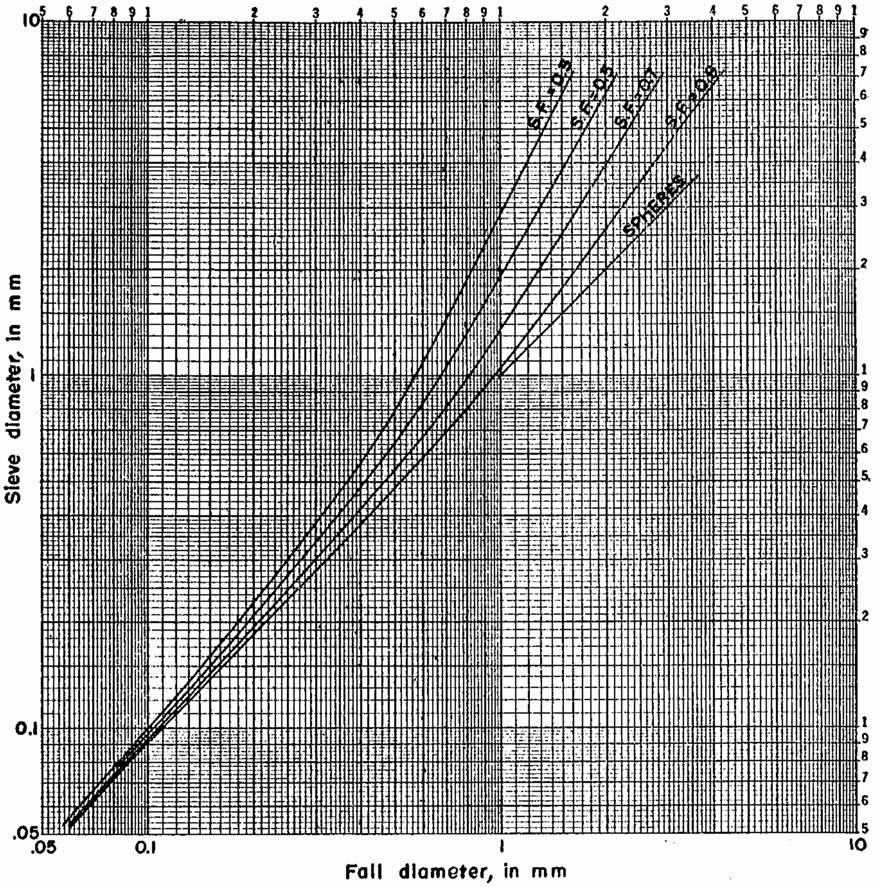


Fig. 2 Relation of sieve diameter to fall diameter for naturally worn quartz particles. (Ref. 1)

Notes: 1. Fall diameter is sedimentation diameter for fall in water at 24°C only.

2. Shape factor S. F. is defined in terms of the triaxial dimensions as:

$$S.F. = c/\sqrt{ab}$$

where

a = longest axis
b = intermediate axis
c = shortest of the three mutually perpendicular axes of the particle.

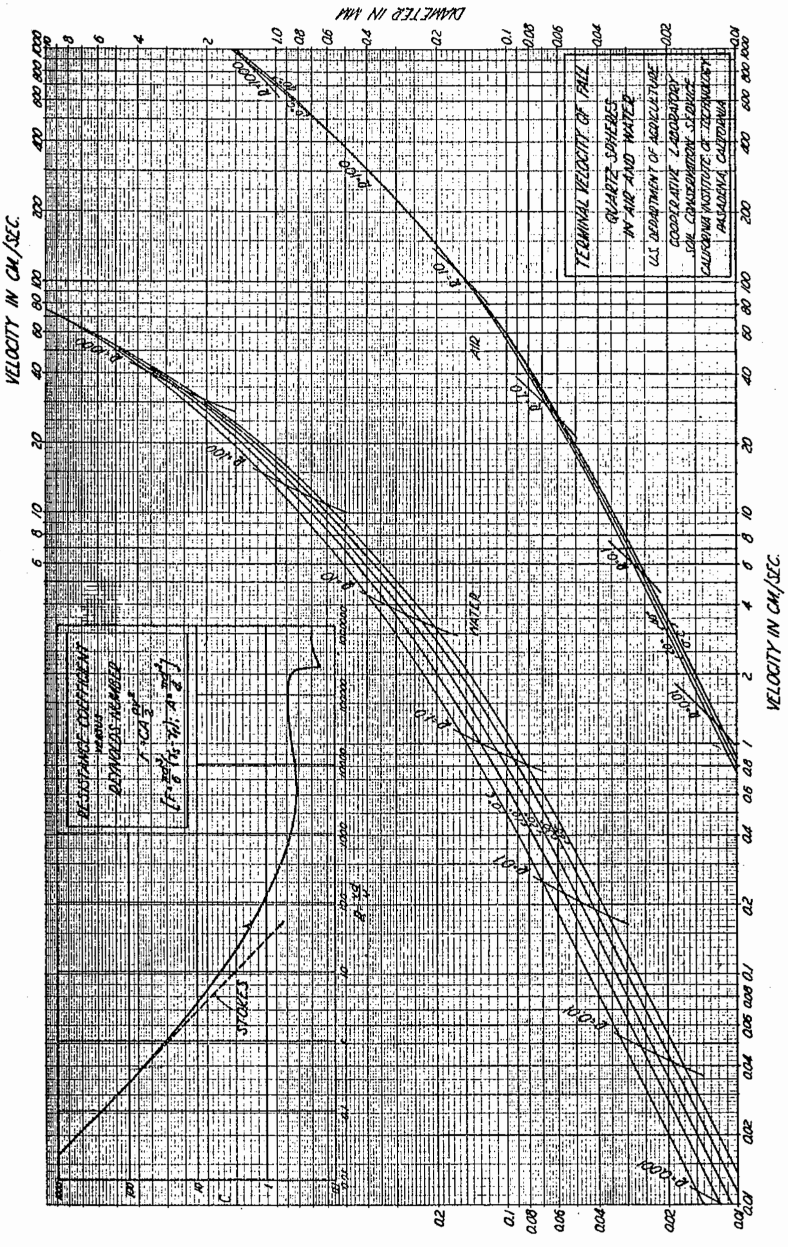


Fig. 3 Terminal velocity of fall for quartz spheres in water and air.

ameter such that 15.9% of the material is finer, and D_{50} and $D_{84.1}$ have similar meanings. It is noteworthy that the size distributions of natural sediments generally plot as straight lines on log-probability paper.

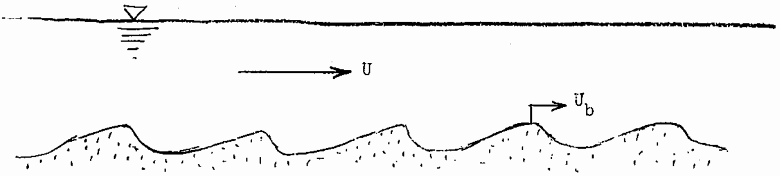
The fall diameters (sedimentation diameter in water at 24°C) can be obtained from the sieve diameter from Fig. 2 (1), which is based on a large body of experimental fall velocity and sieve diameter data. In this figure, the shape factor is $c/\sqrt{ab}$, where a, b and c are respectively the longest, intermediate, and shortest of three mutually perpendicular axes of the particle. For most natural sand grains, the shape factor is about 0.7. With the fall diameters or sedimentation diameters known, the fall velocities can be obtained from Fig. 3.

3. The Fluid-Bed Interface

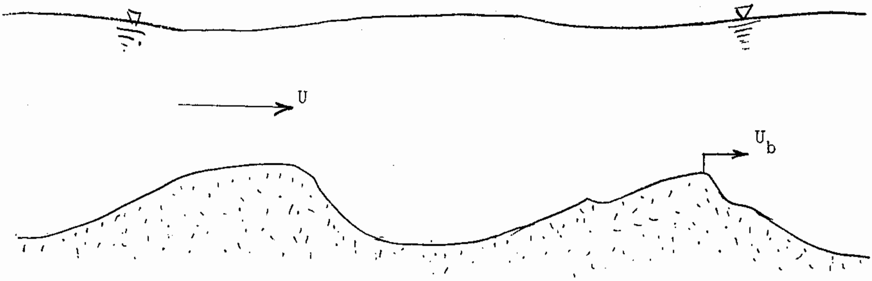
The behavior of the interface between the fluid and the sediment bed is of interest because this is the region from which the transported sediment originates, and the configuration of the interface plays a large role in determining the roughness of the channel. In this section discussions of the different types of bed roughness and the conditions for the initiation of sedimentation are presented.

3.1 *Forms of Bed Roughness in Alluvial Channels*

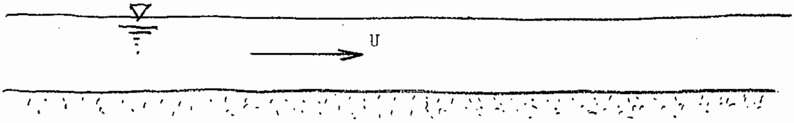
The interface between a moving fluid and an erodible bed is inherently unstable over a wide range of flow conditions, and an initially flat bed will spontaneously develop surface irregularities. For a constant depth of flow, the sequence of bed features with increasing velocity is ripples, dunes, flat bed, and antidunes. These are illustrated schematically in Fig. 4. The primary distinction between ripples and dunes is size, dunes being longer and higher than ripples by a factor of five or more. These two types of features can exist simultaneously, ripples being superimposed on the upstream slopes of the dunes. The flat bed configuration generally occurs at Froude number between about 0.5 and 0.8; however, for some sand sizes and flow depths the flat bed does not occur and instead, features that are not of any one well defined type occur between dunes and antidunes. Antidunes generally occur at Froude numbers greater than about 0.8. At Froude numbers greater than about 1.0, the stationary surface waves that form above and in phase with antidunes become so high that they are



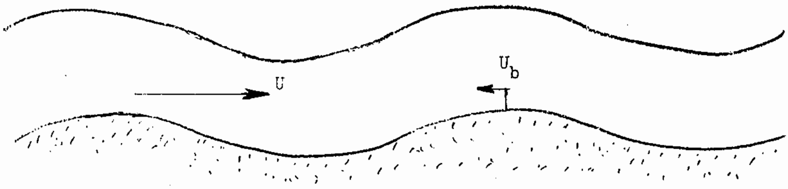
a) Ripples. Bed features move downstream, and water surface is nearly flat.



b) Dunes. The dunes move downstream, and the water surface is slightly wavy.



c) Flat bed.



d) Antidunes. Antidunes move upstream, and the water surface is wavy with an amplitude greater than the antidune amplitude.

Figure 4. Major Forms of Bed Roughness in Alluvial Channels.

unstable and break in the upstream direction, resembling the breaking of ocean waves as they approach a shore. The accompanying agitation levels the bed. Antidunes then reform and grow until the accompanying waves break, the whole process being repeated with a period of one to several minutes.

The mechanism responsible for the formation of these different types of bed features is not well understood. Various theories have been put forth to explain their occurrence, but none of the theories is without some major flaw, and the problem cannot be considered as wholly solved. Furthermore, there does not exist at the present time any reliable method for predicting what type of bed configuration will accompany a given flow, or what the length and height of the bed features will be. The lack of an adequate understanding and formulation of the mechanics of dunes and antidunes is without question one of the greatest impediments to further progress in fluvial hydraulics.

3.2 *Initiation of Sediment Motion*

In the design of stable channels, it is frequently necessary to know the upper limit of velocity or bed shear stress below which the sediment particles on the bed will not move. This information is also required as a parameter in several sediment transport formulas.

For the case of a flat bed, Shields (2) deduced from physical and dimensional arguments based on the forces involved that incipient motion is primarily dependent on the shear stress acting on the bed, and that the dimensionless quantities involved are

$$\frac{\tau_c}{\rho_f g (S_s - 1) D}$$

and

$$\frac{\sqrt{\tau_c/\rho_f}\, D}{\nu} = \frac{U_* D}{\nu}$$

where

τ_c = bed shear stress for incipient motion
ρ_f = fluid density
g = gravitational constant
S_s = specific gravity of sediment
D = grain diameter
ν = kinematic viscosity of fluid
$U_* = \sqrt{\tau_c/\rho_f}$ = shear velocity at incipient motion.

From experimental data he obtained the relation shown in Fig. 5. For sufficiently large values of U_*D/ν (greater than about 10),

$$\frac{\tau_c}{\rho_f g(S_s - 1)D} = \text{constant} = 0.06.$$

For quartz ($S_s = 2.65$) particles in water ($\rho_f g = 62.4$ lb/ft^3),

$$\tau_c = 0.020\,D$$

if τ_c is in lbs. per sq. ft. and D is in mm.

Shields curve (Fig. 5) is applicable only to a flat bed. Actually a flat bed at low velocities is seldom encountered in earth canals because if the flow has even been great enough to move the bed particles, ripples or dunes will have been generated and will remain after the velocity has decreased. The problem of ascertaining the critical tractive force or velocity for incipient motion on a dune-covered or rippled bed is far more complicated than for a flat bed because the problem involves the complex geometry and flow conditions at the bed. It has

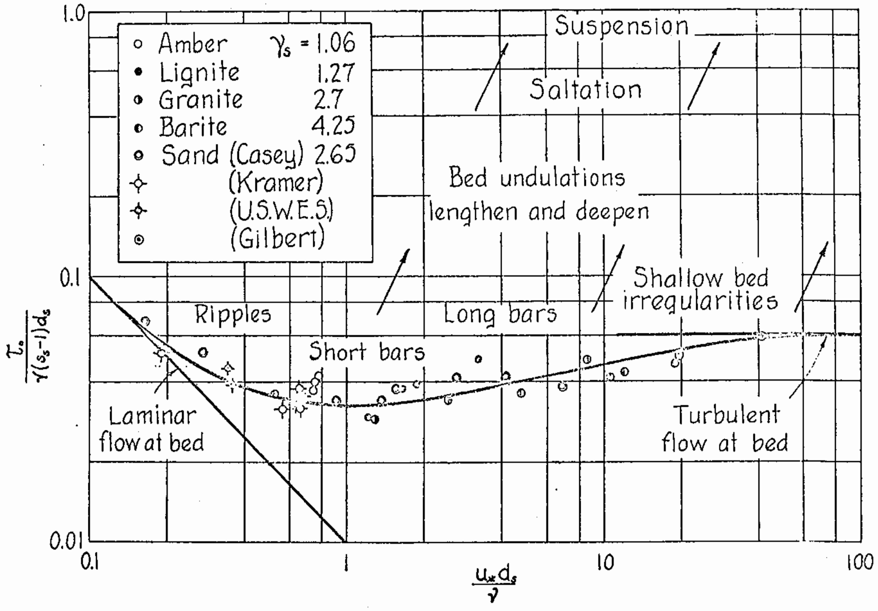


Fig. 5 Shield's curve giving the conditions for incipient sediment motion on a flat bed. (Ref. 2)

been found experimentally (3) that on a rippled or dune-covered bed, incipient motion occurs at a lower velocity and slightly higher bed shear stress, τ_c, than on a flat bed.

4. Roughness of Alluvial Channels

The friction factor for flow in rigid boundary conduits can be predicted quite accurately because the roughness of the boundary is constant for a given material and can be determined once and for all time. This happy situation does not exist for the case of erodible-bed channels in which the bed configuration, and hence the roughness, varies radically, as discussed in Section 3.1 and illustrated in Fig. 4. In fluvial hydraulics the problem that arises in predicting channel roughness is that the channel roughness depends strongly on the bed configuration which in turn depends on the depth and velocity of flow (and also the properties of the bed material and fluid). However, the depth and velocity of flow are a function of the channel roughness. Thus there is no fixed starting point from which to proceed to calculate the rate of energy dissipation for the stream. An additional difficulty is that the relation between the flow conditions and the bed configuration, and the relation between the bed geometry and channel roughness have not yet been adequately formulated.

An example of the variability of alluvial channel roughness is shown in Fig. 6 (4), which shows the variations of the bed friction factor,[1] f_b, slope, s, and bed shear velocity, U^*_b, with depth and velocity. These data were obtained in laboratory experiments with a constant discharge of 0.50 cfs per foot. It is seen that for these experiments the bed friction factor is the most variable quantity involved, varying by a factor of almost six, while the slope varies by a factor of less than five, and the depth and velocity by a factor of about 2.5. Furthermore, it is seen that for a given slope (e.g., $s = 0.0021$) and discharge, it is possible to have two different depth-velocity combinations; a small depth, high velocity and flat bed, or a large depth, low velocity and dune bed. Many natural alluvial channels also have this non-uniqueness of depth and velocity as a function of slope and discharge (5,6).

The question that thus arises is: How does a stream with a given discharge and slope, in the range of values where the non-uniqueness occurs, know which depth and velocity to assume? It is observed in

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[1] The subscript "b" indicates that the quantity has been corrected for the effects of the smooth, vertical walls of the laboratory flume.

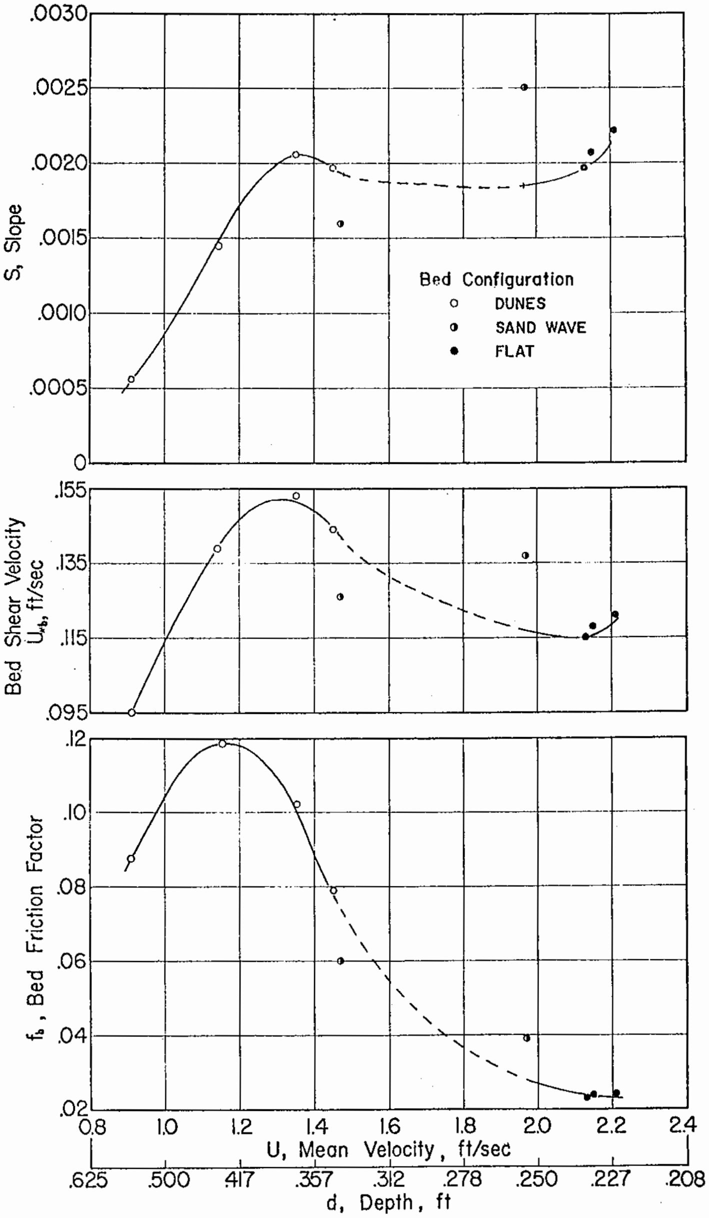

Fig. 6 Variation of slope, bed shear velocity, and bed friction factor with mean velocity and depth for constant discharge (0.50 cfs/ft) experiments in a laboratory flume. (Ref. 4)

both laboratory experiments and in natural streams that the deciding factor is the sediment transport rate; the flow with the smaller depth and higher velocity has a greater capacity to transport sediment. Therefore, when a stream has a relatively large sediment load imposed on it, the bed will be flat, and the velocity will be relatively high.

The above discussion indicates that the roughness of alluvial channels and thus the friction factor for flow in such channels is intimately connected to the form of the bed configuration. As was discussed in Section 3, the problem of predicting the bed form is by no means solved, and therefore, neither is the problem of predicting the roughness of alluvial channels. Several methods have been proposed for predicting the friction factor for flow in alluvial channels. A good summary of available methods has been given by Simons and Richardson (7). Most of these utilize the regime concept of flow in alluvial channels, which attempts to predict the properties of flow in alluvial channels from data on many other channels that have performed satisfactorily.

The method utilizing analysis to some extent that has gained the widest acceptance is that of Einstein and Barbarossa (8). These investigators assumed that the force exerted by the fluid on the bed can be divided into two additive parts: one due to the grain roughness (i.e., the "skin friction" exerted on the bed), and one resulting from the form drag on the ripples and dunes. They further divided the hydraulic radius into two parts

$$R = R' + R''$$

where R is the hydraulic radius of the section, R′ is the hydraulic radius of the equivalent area A′ for which the flow energy is dissipated by skin friction, and R″ is the hydraulic radius for the equivalent area A″ for which the flow energy is dissipated by form drag on the bed irregularities. The effect of grain roughness can be computed from either the Manning-Strickler equation,

$$\frac{U}{\sqrt{g R' s}} = 7.66 \left(\frac{R'}{D_{65}}\right)^{1/6}$$

or the logarithmic equation

$$\frac{U}{\sqrt{g R' s}} = 5.75 \log_{10} \left(12.2 \frac{R'}{D_{65}}\right)$$

where U is the mean velocity, D_{65} is the sand size such that 65% of the bed material is finer, and s is the channel slope. The effect of bed irregularities is obtained from a graphical relation, obtained from data on natural streams, between the dimensionless quantities

$$\frac{U}{\sqrt{g R'' s}}$$

and

$$\psi' = 1.68 \frac{D_{35}}{R'S}.$$

Einstein and Barbarossa's empirical relation between these quantities is shown in Fig. 7. Using the Einstein-Barbarossa procedure to obtain the depth of flow for a given discharge in a channel of known slope,

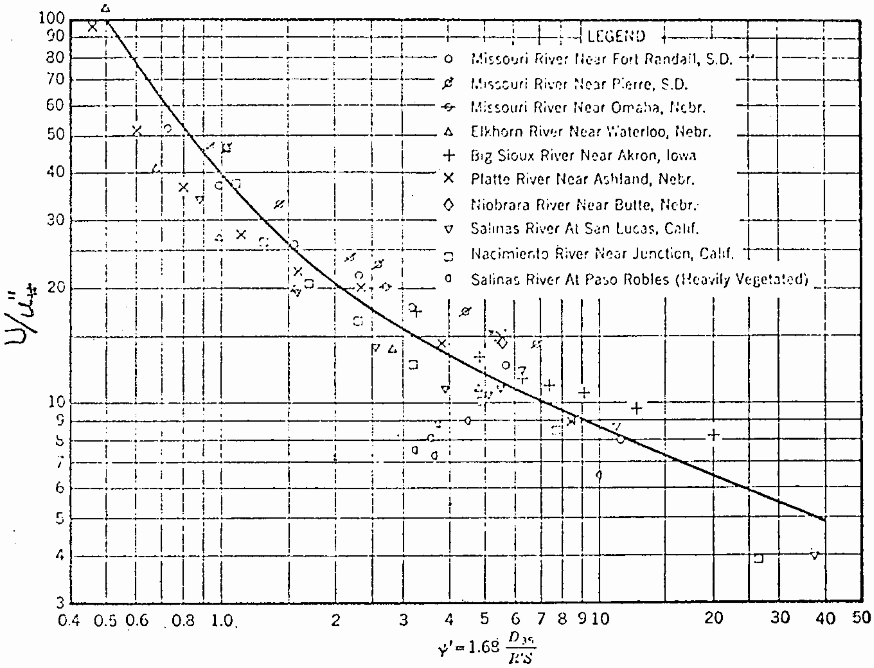

Fig. 7 Bar resistance curve for the Einstein-Barbarossa method of predicting alluvial-channel roughness. (Ref. 8)

geometry and bed material involves straightforward, but rather tedious, trial and error procedure that is described by Einstein and Barbarossa (8). Thus, the Einstein-Barbarossa procedure can be used to develop a stage-discharge relation for an alluvial stream. The parameters involved have been replotted by Vanoni and Brooks (9) in a way that eliminates the need for trial and error solution.

The agreement reported between measured stage-discharge relations and those predicted by the Einstein-Barbarossa method ranges from poor (10) to very good (11). With the possible exception of the regime methods, this is probably the best available method for predicting stage-discharge relations for alluvial streams. Its obvious deficiency is that it does not take account of the effects, discussed in Section 3, of the sediment discharge rate on the bed configuration and channel roughness.

5. SEDIMENT DISCHARGE

If the problem of predicting the roughness of an alluvial channel appears to be a dense forest, predicting the sediment transport rate of an alluvial stream can only be described as a jungle. With the single exception of the suspended local equation, presented below, the present state of knowledge of the mechanisms involved in the entrainment, transportation, and deposition of sediment particles is quite unsatisfactory.

5.1 *Definitions*

The total sediment discharge of a stream is the average quantity of sediment passing a section of the stream per unit time. It is convenient to distinguish two different types of load, as follows:

> Suspended load: That part of the total load that is transported at some distance above the bed, supported by the vertical components of the fluid turbulence, and carried along at approximately the forward velocity of fluid.
>
> Bed load: The fraction of the total load that moves along or very close to the bed by rolling, sliding, and saltating.

Note that there is continuous interchange between the bed load and suspended load, and in the region near the bed it may not be possible to say whether the transported material is bed load or suspended load.

Another classification is on the basis of the relative particle size, as follows:

Bed material load: The part of the total load that is composed of particles found in appreciable quantities in the bed.

Wash load: Finer particles, which are not found in appreciable quantities in the bed, and which have a nearly uniform concentration over the depth of the channel.

Finally, there is a distinction based on the limitations of the measuring method employed.[2] Due to the physical size of samples used for field measurements, it is impossible to measure the bed load, or the suspended load near the bed, without seriously disturbing the bed and hence the transport rate. The load that is sampled by the measuring device used is called the measured load. The difference between the total load and the measured load is called the unmeasured load.

5.2 *Suspended Load*

The suspended load per unit width, q_s, of a stream is

$$q_s = \int_b^d c(y)\, u(y)\, dy \tag{1}$$

where

b = small distance above the bed, taken as the dividing line between the bed load and suspended load

d = stream depth

c(y) = sediment concentration at a distance y above the bed

u(y) = horizontal velocity at a distance y above the bed.

The distribution of sediment concentration is given by the theoretically derived equation

$$\frac{c}{c_a} = \left(\frac{a-y}{y}\,\frac{a}{d-a}\right)^z \tag{2}$$

where

a = distance above the bed at which the sediment concentration is c_a

$z = \dfrac{w}{kU_*}$

w = particle fall velocity

k = Kármán's universal constant

$U_* = \sqrt{gRS}$.

---

[2] A good summary of the methods and instruments used for field measurements of sediment has been prepared by the Federal Inter-Agency Committee on Water Resources (12).

Kármán's constant, k, varies from about 0.2 for heavily sediment-laden flow to 0.4 for clear fluid. Equation (2) is known as the suspended load equation. It was first developed by A. T. Ippen, at the suggestion of Theodor von Kármán, and was first published by Rouse (13). It is based on the equation for continuity of sediment motion, and utilizes the hypothesis that the rate of turbulent mixing of the sediment is proportional to the rate of turbulent momentum exchange within the fluid. Experiments by Vanoni (14) and others have demonstrated that equation (2) predicts the sediment distribution quite accurately. It has also been verified in field measurements by Nordin and Dempster (11), and many others.

The quantity u(y) appearing in equation (1) can be obtained from the Kármán-Prandtl velocity distribution

$$u = u_{max} + \frac{U_*}{k} \ln y/d \qquad (3)$$

where u_{max} is the maximum velocity in the section (at $y = d$). Equations (2) and (3) may now be substituted into equation (1) for q_s. The resulting integral is not readily evaluated, but tables (15) and a nomograph (16) have been prepared for its evaluation.

5.3 *Bed Load*

Because no entirely satisfactory formula or method for prediction of bed load discharge has been obtained, there are many formulas and methods in the literature, and new ones are still appearing. In general, there are two different types of equations:

a) Those of the form[3]

$$q_b = f[(\tau_o - \tau_c)^n]$$

where

q_b = bed load discharge per unit width
τ_o = shear stress acting on bed
τ_c = critical shear stress for incipient motion
n = number that is dependent of flow parameters and sediment properties.

Duboy's formula and the Meyer-Peter formula are of this type.

b) Equations based on detailed analyses of the fluid motion (es-

---

[3] f [ ] is to be read "function of."

pecially the fluid turbulence) and the sediment motion induced by the fluid motion. Examples of these types of analyses are the Einstein bed load function, and the Lane and Kalinske entrainment function.

It is not possible in the allotted space to present, even in summary form, the available bed load formulas. For a presentation and discussion of the different formulas, the reader is referred to the excellent book by Leliavsky (17), the chapter of Rouse's *Engineering Hydraulics* written by Brown (18), and the lecture notes prepared by Vanoni, Brooks, and Kennedy (10).

5.4 *Total Sediment Load*

The total load is the sum of the bed local and suspended load. Actually, many of the formulas that go by the name "bed load formula" are total load formulas for a limited range of conditions. The difficulty encountered in evaluating the suspended load discharge from equation (1) arises in obtaining c_a in equation (2). Einstein (15) has resolved this by evaluating c_a a distance of two grain diameters above the bed. He presents a rather involved method for obtaining c_a at this level. The Einstein method (15), coupled with the Einstein-Barbarossa procedure for predicting channel roughness, constitutes the only available analytic method for predicting the stage-discharge and sediment transport relations for an alluvial stream. The original Einstein procedure has been simplified by Colby and Hubbell (22).

5.5. *Summary and Recommendations*

It is not possible to recommend strongly any of the available transport formulas or methods since they all give errors of up to 100% (10). Only the distribution of suspended sediment can be reliably predicted, from equation (2), but this formula requires knowledge of the concentration c_a at some reference level.

The following recommendations for computing sediment discharge rates were put forth by Vanoni, Brooks, and Kennedy (10):

"1. Values of sediment discharge calculated by formulas are to be considered as estimates only since the errors involved may be 100 percent or more.

2. When possible, it is desirable to use more than one formula to calculate sediment discharge. By comparing the results from several formulas one gets a rough idea of their reliability.

3. Calculated sediment discharge rating curves which have slopes on logarithmic graphs which are in the neighborhood of unity are too flat. These curves should not be used, especially in cases where one is interested in the difference between or the ratio of sediment discharges for different flows in the same channel. In such cases the use of sediment discharge rating curves with slopes of between 2 and 3 on logarithmic graphs should give more reliable results."

6. Regime Methods

The preceding sections have discussed analytic and semi-analytic methods for determining the various quantities needed to analyze or design alluvial channels. With the possible exception of the suspended load equation, the various methods were observed to be, in most cases, only marginally adequate, and have provided the practicing engineer with few useful tools. Another approach to the problem is to put analysis aside and study the interrelation of the various quantities in existing channels that have been found to perform satisfactorily. The relations between the variables are presented either in graphs, or in equations (usually not dimensionally homogeneous) derived from the graphical relations.

Three relations are usually presented:

a. A flow formula which gives the required slope.
b. A relation between channel depth, discharge, and bed material size.
c. A relation between channel width and the flow parameters and bed material properties.

The regime method had its origin in India among the British engineers who were responsible for building and operating the extensive canal systems that distributed irrigation water. Originally, the method was quite unreliable, but as more data became incorporated into the relations and the formulas were refined, the reliability improved until now the regime method is widely used with considerable success. It is a valuable approach for the engineer who must design alluvial channels.

Probably the best (because it is based on the largest body of accurate data) set of regime relations for canals has been developed by Simons and Albertson (20). An extensive treatment of the regime of natural streams has been prepared by Leopold and Maddock (21). Blench's book (22) summarizes the philosophy and history of the regime method, and also presents a set of regime relations.

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FLOW THROUGH POROUS MEDIA

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1. INTRODUCTION

The flow of a fluid through the complicated pore system geometry that is imposed by the boundaries of many solid particles broadly categorizes the hydraulic process called flow through porous media. This pore system geometry is often considered constant and not subject to change due to consolidation of the soil matrix or swelling of the individual particles themselves. This restriction will be imposed in the subsequent development, thus permitting us to concentrate on the hydraulic processes occurring within this constant pore system geometry. In most cases the civil engineer is concerned with the flow of liquids, hence only incompressible fluids will be considered.

Advances in our knowledge of flow through porous media are being reported in various technical journals (e.g. ASCE Technical Journals, Journal of Fluid Mechanics, Journal of Geophysical Research, Journal of Applied Physics, Journal of Hydraulic Research, Industrial and Engineering Chemistry, Chemical Engineering Science, Economic Geology, A.I.Ch.E. Journal, Chemical Engineering Progress, Proceedings of the Royal Society of London, etc.). Many professions (e.g. civil engineers, hydrologists, petroleum engineers, chemical engineers, soil scientists, nuclear engineers, etc.) are currently involved in research in the general area of flow through porous media. The specific areas of interest range from effects on a molecular scale (1) to considerations of entire ground water basins (2). Our theoretical advances result from an increased understanding of the physical process involved coupled with greater use of mathematics. High speed computational procedures have facilitated the obtaining of otherwise obscure solutions. The increased understanding of the physics of flow through porous media also results, at least in part, from more sophisticated research equipment. The work and advances in all disciplines has bearing on the broad topic of water resources.

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The objective of these notes will be to review briefly the latest thinking concerning the fundamentals of flow through porous media with the emphasis on new areas of interest.

2. Basic Equations

2.1 *Continuity Equation*

The continuity equation or equation for the conservation of the mass of the bulk liquid takes the form

$$\frac{\partial \rho}{\partial t} + u \frac{\partial \rho}{\partial x} + v \frac{\partial \rho}{\partial y} + w \frac{\partial \rho}{\partial z} = 0 \qquad (1)$$

where ρ = bulk density of liquid $[M/L^3]$ and u, v, w = seepage velocity components $[L/T]$. The seepage velocity is the average velocity of the liquid in the pores of a small but finite volume (macroscopic volume) which includes both pore spaces and solid particles.

2.2 *Conservation of Dispersing Mass Equation*

If a miscible contaminant is introduced into the flow of a liquid through porous media we can also write an equation for the conservation of the mass of the contaminant. (The term contaminant is used here in its broadest sense and could represent such things as sodium chloride, radioactive wastes, detergents, microorganisms, etc. In some cases heat could even be considered a contaminant.) For conservative contaminants (i.e. those which do not decay, are not absorbed, or do not engage in a chemical reaction) the conservation of dispersing mass takes the form (3)

$$\frac{\partial s}{\partial t} + u \frac{\partial s}{\partial x} + v \frac{\partial s}{\partial y} + w \frac{\partial s}{\partial z} = - \frac{\partial}{\partial x} (\overline{\overset{\circ}{u} \overset{\circ}{s}}) - \frac{\partial}{\partial y} (\overline{\overset{\circ}{v} \overset{\circ}{s}}) - \frac{\partial}{\partial z} (\overline{\overset{\circ}{w} \overset{\circ}{s}}) \qquad (2)$$

where s = average concentration of the mass of the contaminant in the macroscopic volume, $\overset{\circ}{u}$, $\overset{\circ}{v}$, $\overset{\circ}{w}$ are the variations of the actual liquid velocity at a point from the seepage velocities, u, v, w, and $\overset{\circ}{s}$ is the variation of the actual concentration of the contaminant at a point from the average concentration, s, in the neighborhood of the point (or in the macroscopic volume). On a microscopic level the actual velocity components, u_a, v_a, w_a are given by

$$\begin{aligned} u_a &= u + \overset{\circ}{u} \\ v_a &= v + \overset{\circ}{v} \\ w_a &= w + \overset{\circ}{w} \end{aligned} \tag{3}$$

and the actual concentration of the contaminant, s_a, is given by

$$s_a = s + \overset{\circ}{s} \tag{4}$$

The left-hand terms of equation (2) represent the local time rate of change of the average concentration of the contaminant in the macroscopic volume and the mass flux of the contaminant in and out of the macroscopic volume due to convection by the seepage velocities. The right-hand terms represent additional mass flux of the contaminant in and out of the macroscopic volume due to a process which will be called dispersion. The additional mass flux terms result from the macroscopic viewpoint taken in the derivation of equation (2). These dispersion terms account for the additional mass flux of the contaminant due to the complicated velocity pattern in the macroscopic volume and also, within a pore space, to the process of transverse molecular diffusion associated with the parabolic velocity distribution typical of laminar flow. Equation (2) is also valid for turbulent flow through porous media.

If we consider the case of unidirectional flow parallel to the x-axis the convective terms containing v and w vanish. But it is clear that $\overset{\circ}{u}$, $\overset{\circ}{v}$ and $\overset{\circ}{w}$ still exist, hence the additional mass flux terms must be retained. Assuming that the additional mass flux terms can be related to the gradient of the average concentration by a Fickian-type diffusion equation, we have

$$\begin{aligned} \overline{\overset{\circ}{u}\overset{\circ}{s}} &= -D_1 \frac{\partial s}{\partial x} \\ \overline{\overset{\circ}{v}\overset{\circ}{s}} &= -D_2 \frac{\partial s}{\partial y} \\ \overline{\overset{\circ}{w}\overset{\circ}{s}} &= -D_3 \frac{\partial s}{\partial z} \end{aligned} \tag{5}$$

where D_1 is called the longitudinal dispersion coefficient since the flow is in the x-direction and D_2 and D_3 are called lateral or transverse dispersion coefficients. The dispersion coefficients are kinematic quantities having the units of L^2/T. Substituting (5) into (2) gives

$$\frac{\partial s}{\partial t} + u\frac{\partial s}{\partial x} = \frac{\partial}{\partial x}\left(D_1\frac{\partial s}{\partial x}\right) + \frac{\partial}{\partial y}\left(D_2\frac{\partial s}{\partial y}\right) + \frac{\partial}{\partial z}\left(D_3\frac{\partial s}{\partial z}\right) \qquad (6)$$

Equation (6) is sometimes called the convective-dispersion equation. A discussion of the dispersion coefficient will be found in section 5.

2.3 *Equation of Motion*

For laminar flow through porous media the Navier-Stokes equations are sufficient within the pore system. But in order to obtain a useful equation of motion for the complicated flow pattern we must resort to a consideration of the macroscopic volume similar to the treatment of dispersion. Assuming that the inertia terms in the Navier-Stokes equations are vanishingly small for the extremely slow motions usually associated with flow through porous media and integrating the remaining normalized terms through the pore system of the macroscopic volume leads to the following equations for the superficial velocity components:

$$\begin{aligned} U &= -C_x\frac{d^2g}{\nu}\frac{\partial h}{\partial x} \\ V &= -C_y\frac{d^2g}{\nu}\frac{\partial h}{\partial y} \qquad (7) \\ W &= -C_z\frac{d^2g}{\nu}\frac{\partial h}{\partial z} \end{aligned}$$

The coefficients C_x, C_y, C_z are dependent only upon the geometry of the pore system. For an isotropic medium $C_x = C_y = C_z = C$. The superficial velocity components (U, V, W) represent a fictitious average liquid velocity that would exist in the macroscopic volume if no particles were present. The seepage velocities and superficial velocities are related by the expressions

$$\begin{aligned} U &= \theta u \\ V &= \theta v \\ W &= \theta w \end{aligned} \tag{8}$$

where θ is the porosity. $d[L]$ is the normalizing length and is equal to the average grain size (e.g. the 50 per cent grain size determined from a standard gravimetric sieve analysis). g is the gravitational acceleration $[L/T^2]$ and $\nu[L^2/T]$ is the kinematic viscosity of the bulk liquid. $h[L]$ is the piezometric head and is given by

$$h = \frac{p}{\gamma} + y \tag{9}$$

where $p =$ liquid pressure $[F/L^2]$ and γ $[F/L^3]$ is the specific weight of the bulk liquid. Gravity has been assumed here to act in the negative y-direction.

Equations (7) can be written as

$$\begin{aligned} U &= -\frac{k_x g}{\nu}\frac{\partial h}{\partial x} \\ V &= -\frac{k_y g}{\nu}\frac{\partial h}{\partial y} \\ W &= -\frac{k_z g}{\nu}\frac{\partial h}{\partial z} \end{aligned} \tag{10}$$

where $k_i (k_i = C_i d^2, i = x, y, z)$ is called the intrinsic permeability and is dependent upon only the geometry of the pore system. It has the units of length squared. The Darcy permeability, K, is given by $K_i = \dfrac{k_i g}{\nu}$ and has the units of velocity. The Darcy permeability is seen to be dependent upon the pore system geometry and the liquid that is flowing.

Equations (7) and (10) are valid for laminar flow through porous media when the inertia terms in the Navier-Stokes equations are negligible. This regime of flow is referred to as the linear laminar regime because the velocities are linearly related to the piezometric gradient. The parameter which generally describes the relative magnitude of the inertial forces to viscous forces is the Reynolds number. For porous media flow this can be written using the seepage velocity and the average particle size as the characteristic length, $\mathbb{R} = \frac{ud}{\nu}$. As the Reynolds number approaches unity the inertia forces approach the same order of magnitude as the viscous forces and hence can no longer be neglected in the equation of motion. Although this type of flow may still be laminar, equations (7) and (10) are no longer valid. This regime of laminar flow which is characterized by a non-linear relationship between the seepage velocity and the piezometric gradient is calleld the non-linear laminar regime. At some higher Reynolds number ($\mathbb{R} \approx 60$) (4) inception of turbulence occurs and as the velocity increases the flow will become fully turbulent.

For an isotropic porous medium $k_x = k_y = k_z = k$ and from equations (10) we see that the superficial velocity components are derivable from a potential function, $\phi = -\frac{kg}{\nu} h$, $\left(\text{i.e. } U = \frac{\partial \phi}{\partial x}, V = \frac{\partial \phi}{\partial y} \right.$ $\left. W = \frac{\partial \phi}{\partial z} \right)$ The substitution of ϕ into the condition for an incompressible fluid $\left(\frac{\partial U}{\partial x} + \frac{\partial V}{\partial y} + \frac{\partial W}{\partial z} = 0 \right)$ yields Laplace's equation

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (11)$$

Two-dimensional problems and problems with radial symmetry have been the focus of most attention. For these two-dimensional problems we may also define a stream function, ψ, such that

$$U = \frac{\partial \psi}{\partial y}$$
$$V = -\frac{\partial \psi}{\partial x} \qquad (12)$$

The existence of a velocity potential implies irrotationality. If the stream function is substituted into the condition for irrotational flow for two-dimensional flow $\left(\frac{\partial V}{\partial x} = \frac{\partial U}{\partial y} \right)$ we see that the stream function also satisfies Laplace's equation. The properties of these two mutual orthogonal families of curves (ψ = constant and ϕ = constant) are utilized in the construction of flow nets.

3. Boundary Conditions

If we have a bounded region within which Laplace's equation is valid everywhere and if the conditions on the boundaries of the region are specified, then a unique solution exists for ϕ and ψ. Commonly encountered boundary conditions will be reviewed briefly (5).

3.1 *Impermeable Boundary*

Along an impermeable boundary such as line ED in Fig. 1, $\frac{\partial \phi}{\partial N} = 0$ and ψ = constant.

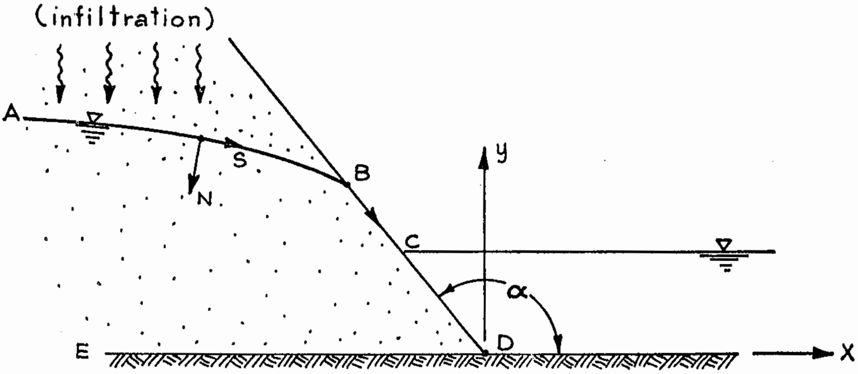

FIG. I DIAGRAM ILLUSTRATING BOUNDARY CONDITIONS

3.2 *Seepage Surface*

A surface of seepage (line BC in Fig. 1) is a boundary of the medium itself and also the flow region along which the flow is exposed to atmospheric conditions. If the atmospheric pressure is taken as zero, then along this surface $\phi + Ky = 0$. If α is the angle which the surface of seepage makes with the x-axis, then taking the derivative of $\phi + Ky = 0$ with respect to S, the direction of flow, gives

$$\frac{\partial \phi}{\partial x} \cos \alpha + \frac{\partial \phi}{\partial y} \sin \alpha = -K \sin \alpha$$

where $\cos \alpha$ and $\sin \alpha$ have been substituted for $\frac{dx}{dS}$ and $\frac{dy}{dS}$ respectively. For vertical surfaces of seepage, $\alpha = \pi/2$, and $\frac{\partial \phi}{\partial y} = -K$, (i.e. the velocity is vertical and is equal to $-K$).

3.3 *Phreatic Surface*

The phreatic surface, water table, or free surface (line AB in Fig. 1) is a boundary of the flow region which is still contained in the porous medium but considered exposed to atmospheric conditions. The basic difference between the surface of seepage and the phreatic line is that generally the surface of seepage is known and its geometry can be stated, but the position of the phreatic line and its shape are not known a priori. The condition $\phi + Ky = 0$ is also valid along the phreatic line. If there is no infiltration or evaporation, the phreatic line may be considered a streamline (i.e., $\psi = \text{constant}$). If water is added or lost at a constant rate along the phreatic line, then $\psi = ex + \text{constant}$ along the phreatic line where e is the volume of water added or lost per unit length along the phreatic line per unit time.

3.4 *Reservoir Boundaries*

When boundaries of the flow region are reservoirs of water (line CD in Fig. 1), the condition $\phi = \text{constant}$ pertains.

3.5 *Dispersion Problems*

Boundary conditions for the contaminant must also be established for dispersion problems. Generally we can state that no mass flux of the contaminant occurs across impermeable boundaries or the phreatic surface $\left(\text{i.e., } \frac{\partial s}{\partial N} = 0\right)$. The distribution of the concentration of the contaminant is sometimes known and constant along other boundaries. If it is not constant, its time variation must be known. One important exception to the condition of no mass flux across the phreatic line would be the addition of a contaminant to the flow system through infiltration due to rainfall (e.g., radioactive fallout).

4. METHODS OF SOLUTION

Many groundwater flow problems do not require consideration of the dispersion of some contaminant. In some problems where two liquids of different density are involved, approximate solutions may be obtained on the assumption that the liquids are immiscible (i.e. no mixing at the interface between the two liquids). The method of solution to these problems where dispersion is absent requires only the determination of ϕ and ψ. An excellent treatment of many such problems is found in the recently translated text by Polubarinova-Kochina (6). For the problems where the distribution of some contaminant is desired, s(x,y) must also be determined. The methods of solution can be generally classified as (a) analytical, (b) numerical, (c) graphical, (d) models and analogs and (e) approximations.

4.1 *Analytical Solutions*

Exact analytical solutions for ϕ and ψ usually require simple geometries to the flow boundaries. The most common techniques for exact solutions are the use of conformal mapping including the hodograph plane and separation of variables. Transforms, such as the Fourier transform, are useful for infinite or semi-infinite regions. Superposition of separate solutions is also valid since Laplace's equation is linear in ϕ and ψ. The treatment of the conformal mapping technique is beyond the scope of these notes and the reader is referred to (5), (6) and (7).

4.2 *Numerical Solutions*

Numerical solutions are feasible using finite difference representation for the differential equations and high speed computational procedures. The boundary conditions must be specified.

Laplace's equation for the stream function, ψ, becomes in finite difference representation

$$\psi_o = \frac{1}{4}(\psi_1 + \psi_2 + \psi_3 + \psi_4) \qquad (13)$$

where ψ_i (i = 1, 2, 3, 4) represent the value of ψ at the four lattice points surrounding the interior node point with a value of ψ_o. The precision of the numerical procedure depends on the fineness of the lattice grid. Special difference equations similar to equation (13) must be written for nodes having unequal arms to the surrounding lattice points.

Consider the flow under a sheetpile (see Fig. 2). The flow picture is symmetrical about the sheetpile. Line BC is a streamline and may be given the value of $\psi = 0$. Line ED is a streamline and may arbitrarily be given the value of $\psi = 1$ for ease in computation. Line AE is a continuation of ED and also has the value of $\psi = 1$. It happens that the location of AE does not significantly affect the flow picture near the sheetpile as long as it is reasonably far from the sheetpile. Lines AB and CD are lines of constant potential and hence the streamlines must be perpendicular to them at intersection. This requirement induces symmetry of the ψ values about these potential lines and permits writing the difference equation at the nodes along AB and CD in terms of interior lattice points only. For the problem shown in Fig. 2 there are 36 nodes and hence 36 difference equations. Electronic digital computers are capable of solving this set of 36 simultaneous algebraic equations in seconds. Once the values of ψ at the node points are known, the streamlines may be sketched in. The accuracy of the procedure can be increased by superimposing a finer grid over all or only a portion of the flow.

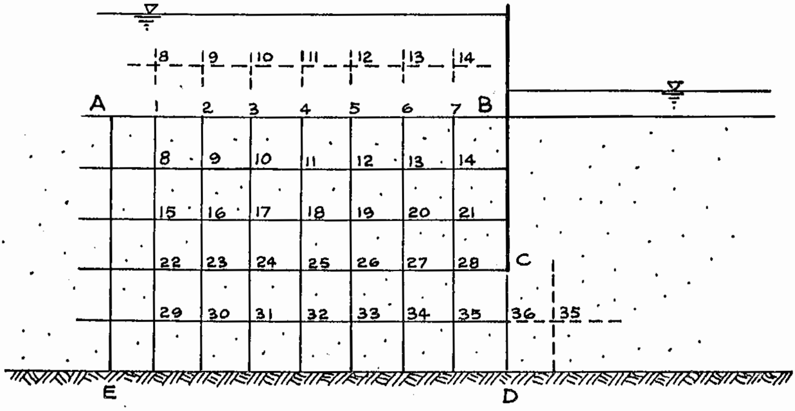

FIG. 2 FLOW UNDER A SHEETPILE WITH LATTICE GRID SUPERIMPOSED FOR NUMERICAL SOLUTION

4.3 *Flow Nets*

Sketching of flow nets is still a very useful tool. Theoretically there is only one correct net for a given problem (e.g. the one shown in Fig. 2). The experience of the sketcher determines how closely this exact net is approached.

4.4 *Models and Analogs*

Models and analogs are particularly useful in research. The sand model and Hele Shaw model are widely used (8), (9). The electric analogy and membrane analogy are other types of models (10).

4.5 *Approximations*

The most widely used approximation is the Dupuit-Forcheimer approximation. It is especially useful when there is a free surface (i.e., unconfined flow). The Dupuit-Forcheimer approximation permits reformulation of the problem so as to eliminate the free surface boundary which is not known a priori. The basic assumption used in the formulation of the Dupuit-Forcheimer equation is that the variation of the piezometric head with y is negligible. This assumption requires that vertical velocity components also be negligible which implies small gradients of piezometric head. The final equation, including infiltration due to rainfall ($e > 0$) or evaporation ($e < 0$) is

$$\frac{\partial^2(h^2)}{\partial x^2}+\frac{\partial^2(h^2)}{\partial y^2}+\frac{\partial^2(h^2)}{\partial z^2}+e=\frac{2\theta}{K}\frac{\partial h}{\partial t} \tag{14}$$

Provisions for leakage through the base of the aquifer can also be included. This non-linear partial differential equation can be linearized by introduction of a new variable that represents small deviations of the piezometric head from some average value, $\bar{h}$, typical to the problem under consideration. This linearized equation takes the form

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}+\frac{e}{2\bar{h}}=\frac{\theta}{K\bar{h}}\frac{\partial h}{\partial t} \tag{15}$$

There is no need to resort to equation (15) for steady state problems since equation (14) is linear in h^2 in those cases.

5. Dispersion Coefficients

For the simple case of unidirectional flow parallel to the x-axis through an isotropic homogeneous porous medium, we need only consider two dispersion coefficients (11), D_1 and D_2. Equation (6) becomes for this case

$$\frac{\partial s}{\partial t}+u\frac{\partial s}{\partial x}=\frac{\partial}{\partial x}\left(D_1\frac{\partial s}{\partial x}\right)+\frac{\partial}{\partial y}\left(D_2\frac{\partial s}{\partial y}\right)+\frac{\partial}{\partial z}\left(D_2\frac{\partial s}{\partial z}\right) \tag{16}$$

We see that even for this simple case we must retain D_1 and D_2 inside the derivative. We intuitively suspect that D_1 and D_2 are dependent upon the geometry of the pore system which is constant in this case since the medium was specified isotropic and homogeneous. The other important variable that may still vary is the seepage velocity, u. From experiments (12), it is found that the dispersion coefficients are dependent upon the absolute magnitude of the seepage velocity. This dependency can be written in equation form as (13), (14)

$$\frac{D_1}{\nu} = a_1 \mathbb{R}_d^{n_1}$$
$$\frac{D_2}{\nu} = a_2 \mathbb{R}_d^{n_2} \tag{17}$$

where a_1 and a_2 are coefficients that are primarily dependent upon particle shape and possibly size distribution. For nearly uniform media, experiments have shown that a_1 and a_2 are independent of particle size. $\mathbb{R}_d$ is called a grain size Reynolds number and is given by

$$\mathbb{R}_d = \frac{|u|d}{\nu} \tag{18}$$

The exponents n_1 and n_2 are primarily dependent upon particle size distribution. Experimental evidence confirms that n_1 and n_2 are independent of particle shape and size.

The ratio of the longitudinal and lateral dispersion coefficients should be of the form

$$\frac{D_1}{D_2} = a \mathbb{R}_d^{n} \tag{19}$$

where $a = a_1/a_2$ and $n = n_1 - n_2$. In accordance with the above discussion, a and n are functions only of the media characteristics. For nearly uniform porous media composed of spheres (13), (14), $a_1 = 0.66$, $a_2 = 0.037$, $n_1 = 1.2$ and $n_2 = 0.7$. For nearly uniform angular particles $a_1 = 0.90$ and $n_1 = 1.2$. Preliminary evidence indicates that n_1 may approach unity for highly non-uniform media.

The longitudinal dispersion coefficient has been correlated with the intrinsic permeability (13). This correlation is expressed as

$$\frac{D_1}{\nu} = b \mathbb{R}_k^{n_1} \tag{20}$$

where b is a dimensionless coefficient dependent upon only the particle shape and size distribution. The exponent n_1 is primarily a function of size distribution. For nearly uniform media composed of spheres, $b = 54$ and $n_1 = 1.2$. For nearly uniform media composed of angular particles $b = 88$ and $n_1 = 1.2$. $\mathbb{R}_k$ is called the permeability Reynolds number and is formed using the square root of the intrinsic permeability as the characteristic length. The correlation expressed in equation (20) provides a method of estimating the dispersion parameter from a knowledge of the more easily measured permeability.

6. Dispersion with Adsorption

Some phenomena involving the transport of a contaminant through porous media are subject to dispersion and adsorption. The adsorption process involves the transfer of the contaminant between the liquid and solid phases (i.e., the contaminant may be absorbed from the liquid by the solid particles, or the liquid may be acquiring the contaminant from the solid phase). To account for this adsorption process, an additional term must be included in the conservation of dispersing mass equation (6). This becomes (15)

$$\frac{\partial s}{\partial t} + \left(\frac{1-\theta}{\theta}\right)\frac{\partial F}{\partial t} + u\frac{\partial s}{\partial x} = \frac{\partial}{\partial x}\left(D_1\frac{\partial s}{\partial x}\right) + \frac{\partial}{\partial y}\left(D_2\frac{\partial s}{\partial y}\right) + \frac{\partial}{\partial z}\left(D_3\frac{\partial s}{\partial z}\right) \qquad (21)$$

where F is the concentration of the contaminant in the solid phase. The relationship between F and s depends on the exact mechanism of interphase transfer. One such relationship is

$$F = C_1 s + C_2 \qquad (22)$$

which gives

$$\frac{\partial F}{t\partial} = C_1\frac{\partial s}{\partial t} \qquad (23)$$

Equation (23) states that the rate of change of the concentration in the solid phase is proportional to the rate of change of the concentration in the liquid phase. This implies that the adsorption process takes place instantaneously when changes in the local concentration, s, of the contaminant occur. This is often referred to as the equilibrium case.

Assuming the above relationship between F and s, equation (21) becomes

$$\varepsilon \frac{\partial s}{\partial t} + u \frac{\partial s}{\partial x} = \frac{\partial}{\partial x}\left(D_1 \frac{\partial s}{\partial x} \right) + \frac{\partial}{\partial y}\left(D_2 \frac{\partial s}{\partial y} \right) + \frac{\partial}{\partial z}\left(D_3 \frac{\partial s}{\partial z} \right) \quad (24)$$

where

$$\varepsilon = 1 + \frac{C_1(1 - \theta)}{\theta}.$$

An expression such as equation (21) which includes the adsorption-desorption term would be applicable to problems such as reclamation of salt encrusted soils by fresh water flushing (15) and removal of some contaminant (e.g., detergents) by passage of the contaminated solution through a porous bed of surface active particles (16).

7. Expanded Media

The flow of a liquid around the grains of a porous medium exerts a seepage force on the particles. For sufficiently high vertical seepage velocities the individual particles will be supported by these upward forces. This condition is referred to as an expanded or fluidized bed. It has applications in many chemical processes, hydraulic gradation of sands, back-washing of filter beds, sediment entrainment, etc.

The analysis of the total resistance to flow by an expanded bed is complicated because of the interference of one particle on the flow pattern of another. This interference produces unsteady effects resulting in violent random movements of the particles within the constant limits of the expanded bed.

Consider the upward flow of water through a column containing a porous medium. At some critical seepage velocity the drag force on a typical particle will equal the submerged weight of the particle and a state of incipient expansion of the bed results. A further increase in the seepage velocity will cause the particles to expand, thus reducing the seepage velocity (due to the increase in porosity) and a quasi-steady state obtains wherein the limits of the expanded bed remain constant but the individual particles are subject to violent random fluctuations within the expanded bed limits. Statistically the average drag force on each particle must equal the submerged weight of the particle in this quasi-steady condition. Utilizing the drag equation and conservation of energy principle, an expression for the superficial velocity can be derived for the expanded bed,

$$V = \sqrt{\frac{4}{3(1-\theta)} \frac{gd}{C_D} \frac{\partial h}{\partial y}} \qquad (25)$$

where C_D is the drag coefficient appropriate for this condition.

Experimental results (17) on uniform spheres are plotted in Fig. 3 as C_D versus $\mathbb{R}$, where $\mathbb{R} = \frac{Vd}{\nu}$.

$\mathbb{R}_I$ is the Reynolds number which defines the state of incipient expansion. $\mathbb{R}_s$ is the extrapolation of the experimental curve for the expanded porous bed to intersection with the well-known curve for one sphere. Significantly, $\mathbb{R}_s$ agrees with the Reynolds number that would be given using the settling velocity of a single sphere. The Reynolds numbers

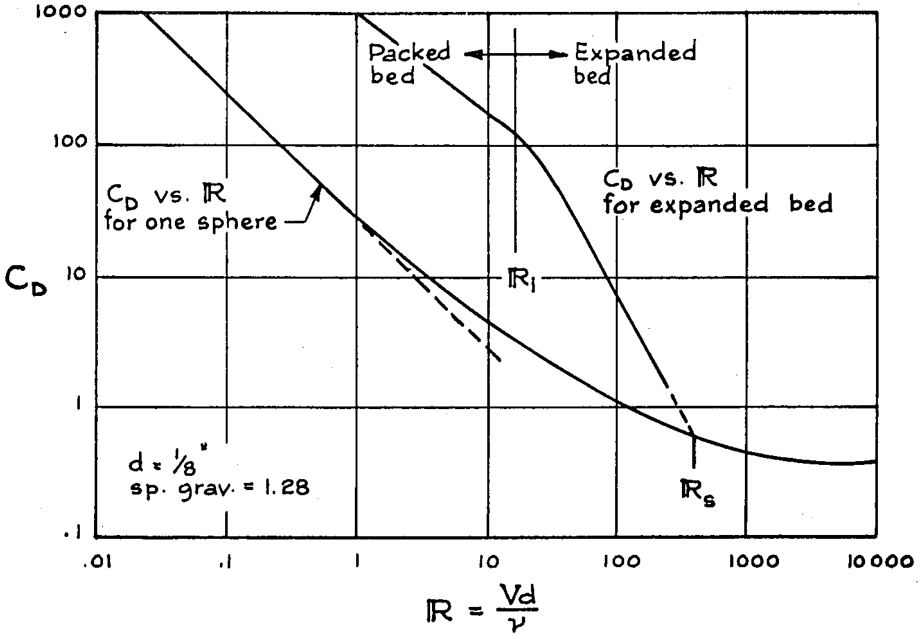

FIG.3 C_D VS. IR FOR A SINGLE SPHERE AND A PACKED AND EXPANDED BED OF SPHERES between $\mathbb{R}_i$ and $\mathbb{R}_s$ give the velocity at which the whole bed would settle were the flow to be abruptly stopped.

8. Present Studies

Present studies in the area of water resources include salt water intrusion and dispersion in coastal aquifers, mechanics of flow in the vicinity of recharge wells, spread of contaminants from recharge wells, clogging phenomenon, seepage from surface reservoirs, underground disposal of radio-active wastes, adsorption associated with leaching of salt encrusted soils, and removal of detergents by flow through activated carbon.

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CURRENT PRACTICE IN WATER TREATMENT

By Robert L. Meserve,* *Member*

(Presented at the New England Conference on Urban Planning for Environmental Health, Tufts University, September 8, 1965.)

For centuries, man has been confronted with the problem of obtaining a satisfactory water supply. His early experience included the search for adequate quantity; the discovered sources must then have undergone some primitive assessment as to quality, especially with regard to taste and general appearance. Although some ancient societies experimented with, and used, various methods of physical improvement, including coagulation, sedimentation, and filtration, the major concern until recently has been with supply and distribution rather than with quality control.

Time has compounded man's problems in water supply. Centralization of population, general population growth, and technological progress have multiplied the demands on available sources of water. Hygienic quality has become as important as quantity. The use of water bodies and courses for the disposal of a multitude of domestic and industrial wastes has made the modern day approach to water supply a far different consideration from that of even a few decades ago. The entire problem of furnishing a useful supply of safe, potable water represents one of the great challenges to 20th century man. Millions of dollars are spent annually in the improvement of inferior quality raw sources to a condition such that these supplies may be used at home and in industry without undue concern for harmful effects that might otherwise be encountered. Some of the current practices directed toward this improvement will be discussed here.

Disinfection

Current water treatment practice centers around one all-important goal: the provision of domestic water which is *safe* for the consumer. It follows, therefore, that the signal treatment process is that of disinfection. Although many large cities in Europe use ozone as a water disinfectant, chlorine is by far the most common germicide used in American water supply. Chlorine became available commercially in

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TABLE I
TREATMENT OF PUBLIC WATER SUPPLIES OF 100 LARGEST CITIES IN THE UNITED STATES, 1962
(From Ref. 2)

Treatment	No. of Cities	Population Served: No. in 1,000's	Population Served: Percentage of Total
A. *Surface Water*			
Chlorination	66	39,939	65.9
Sedimentation and coagulation	54	27,772	45.8
Filtration			
Slow sand	7	2,536	4.2
Pressure	2	356	0.6
Rapid sand	51	26,511	43.8
Iron removal	4	1,475	2.4
Softening			
Lime	10	4,649	7.7
Lime-soda ash	9	3,359	6.0
B. *Ground Water*			
No treatment	1	150	0.2
Chlorination	19	5,565	9.2
Iron and manganese removal	5	1,474	2.4
Sedimentation and coagulation	7	2,147	3.5
Rapid sand filtration	7	2,970	4.9
Softening			
Lime	3	1,055	1.7
Lime-soda ash	1	320	0.5
C. *Mixed Surface and Ground Water*			
No treatment	1	72	0.1
Chlorination	13	14,015	23.1
Iron and manganese removal	1	500	0.8
Filtration			
Rapid sand	8	1,921	3.2
Slow sand	1	33	0.1
Sedimentation and coagulation	7	8,770	14.5
Softening			
Lime*	2	340	0.6
Lime-soda ash	1	259	0.4
Cation exchange	2	698	1.2

* At least one city supplements lime softening with soda ash during critical periods.

TABLE II

WATER TREATMENT OPERATIONS IN THE NEW ENGLAND STATES, 1963—FACILITIES SERVING 100 OR MORE USERS

(Compiled by the author from data in Ref. 3)

	No. of Facilities by State						N.E.
TYPE OF TREATMENT	Conn.	Maine	Mass.	N.H.	R.I.	Vt.	Total
A. *Aeration*							
Contact beds or trays, coke or other	0	0	3	2	1	0	6
Patented aerator	1	0	2	0	0	0	3
Spray aerator	1	1	9	0	3	0	14
Overflow trays, cascade or other splash	1	0	1	1	1	0	4
Other types	0	1	0	0	0	0	1
Not specified	4	0	4	0	0	0	8
B. *Chemical Coagulation or Softening*							
Alum	17	10	14	5	9	3	58
Iron Salts	0	0	2	0	1	0	3
Lime	10	6	6	2	9	0	33
Soda ash	10	6	4	1	3	2	26
Activated Silica	0	0	0	0	0	0	0
Other coagulant	0	1	4	0	0	0	5
C. *Disinfection*							
Chlorine gas	53	50	72	18	9	27	229
Hypochlorite	48	76	40	54	19	34	271
Chlorine dioxide	0	2	0	0	0	0	2
Ozone	0	0	0	0	0	0	0
Other means	0	0	2	0	0	0	2
Dechlorination	0	0	2	1	0	1	4
D. *Filtration*							
Anthrafilt	0	0	2	0	2	0	4
Diatomaceous earth	0	2	0	0	0	0	2
Sand	10	20	24	6	10	6	76
Zeolite	1	1	1	2	2	0	7
Pressure	2	3	1	1	5	2	14
Gravity (slow)	2	6	16	3	0	0	27
Gravity (rapid)	17	13	13	6	8	4	63
Roughing	0	1	0	0	0	0	1
Unspecified	4	0	0	0	0	0	4
E. *Chemical Stabilization or Corrosion Control*							
Phosphate compounds	30	9	46	4	8	0	97
Chlorine gas	3	6	0	0	0	0	9
Hypochlorite	2	1	0	0	0	0	3
Sodium silicate	1	0	0	1	0	0	2
Alkali for pH adjustment	18	12	40	7	10	0	87
Unspecified	0	0	3	0	0	0	3

TABLE II (*Continued*)

	No. of Facilities by State						N.E.
TYPE OF TREATMENT	Conn.	Maine	Mass.	N.H.	R.I.	Vt.	Total
F. *Mixing Process, Unspecified Purpose*							
Air agitation	1	0	0	0	0	0	1
Baffles	0	3	2	0	0	0	5
Hydraulic (standing wave flume)	0	0	0	0	2	0	2
Injection or pump suction	0	7	0	0	0	0	7
Slow mechanical	1	2	7	0	3	0	13
Rapid mechanical	0	2	5	1	1	2	11
Other, or unspecified	0	3	3	0	1	0	7
G. *Ammoniation*							
Ammonium compound	0	2	0	0	0	0	2
Ammonia gas	0	7	0	0	0	0	7
Unspecified	1	0	0	0	0	0	1
H. *Sedimentation*							
Basins, baffled main portion	0	0	6	0	2	2	10
Covered basins, other than housed	2	4	7	1	2	0	16
Open basins, may be housed	0	4	1	0	8	0	13
Mechanical sludge removal	0	0	1	0	0	0	1
Unspecified	18	1	7	0	0	2	28
I. *Taste and Odor Control*							
Activated carbon	3	6	6	2	6	0	23
Chlorine dioxide	0	1	3	0	1	0	5
Sulfur dioxide	0	1	0	0	1	0	2
Ozone	0	0	0	0	0	0	0
Other	0	1	0	0	0	0	1
J. *Fluoride Adjustment*							
Hydrofluosilicic acid	5	3	1	0	0	0	9
Sodium silicofluoride	3	2	5	0	7	1	18
Sodium fluoride	4	11	15	3	3	3	39
Other fluorides	0	0	0	0	0	0	0
Fluoride reduction	0	0	0	0	0	0	0
K. *Summary*							
No treatment	48	44	191	44	19	126	472
Total number supplies, 100 or more users, no duplication	148	194	376	115	46	184	1,063*
Percent receiving some type of treatment	67.6	77.3	49.2	61.7	58.7	31.6	55.6

* Actually 1,427 communities of 100 or more are supplied by 1,063 separate facilities.

1910. That year produced an important legal decision also, when the court, having ordered a New Jersey water company to furnish water "pure and safe," rendered a favorable opinion on the question of chlorination of the Jersey City supply: "I do therefore find and report that [chlorination] is capable of rendering the water delivered to Jersey City pure and wholesome . . . and is effective in removing from the water those dangerous germs which were deemed . . . to possibly exist therein at certain times" (1). The application of chlorine, in one form or other, has been common practice in this country ever since. Usually it is added in sufficient quantity to result in a small residual after 10 or 15 minutes contact. There is no question that the process has all but eradicated many water-borne diseases, but whether chlorination as it is presently performed is effective against viral organisms is an unknown requiring continuing research. As indicated in Table I, of the water supplies for the 100 largest cities in the United States in 1962 (2), 98, serving a combined population of almost 60,000,000, were chlorinated. In New England, as indicated in Table II, of the 1,063 different supplies serving communities of 100 or more in 1963, 502, or 47%, applied chlorine for the purpose of disinfection (3).

Filtration

Modern filtration of water dates from the 19th century in England, and has evolved from the early slow sand filters (SSF) to the rapid sand filter (RSF) developed largely by the Lawrence (Mass.) Experiment Station. The differences in the two processes are evident primarily in the flow-through rates, and therefore, in the relative areas required.

Most of the filtering[1] in the SSF is accomplished at the surface of the sand bed after enough particulate matter has collected to form a *schmutzdecke;* typical rates are in the vicinity of 4 mgad (million gallons per acre per day). In the RSF the entire bed of 24-30 inches of filter media effectively participates in particulate removal, at a rate approximating 120 mgad. (There is some tendency toward higher rates, and experimentation has indicated that there may be little reduction in effluent quality with rates as high as 230 mgad provided pretreatment is adequate (4).) Filter media is usually sand, but may include crushed coal, or, in the case of pressure filters, magnetite or diatomaceous earth.

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[1] "Filtering" is a misleading term, implying a purely mechanical straining action. Evidently other processes occur within the filter as well, including biological adsorption, flocculation and sedimentation.

Again referring to the 1962 data on supplies for the 100 largest U. S. cities (Table I), filtration was accomplished in 76, and 87% of these were RSF installations. In New England there were 63 RSF, 27 SSF and 14 pressure filters operating in public water supply in 1963 (3).

Coagulation

The clarification of colloidal suspensions by coagulation and sedimentation is not a modern technique. The Chinese used alum for this purpose centuries ago in the treatment of river waters. Londoners did the same thing in Old England for the improvement of turbid pump water. Whereas the electrochemistry of the process was little understood in the earlier applications, the present-day approach to coagulation has at hand a fairly well established body of technology. Commercial alum ($Al_2(SO_4)_3 \cdot 14H_2O$), according to Ockerhausen (5), is the most commonly used coagulant, and is an excellent index of the "vigor of the water industry"—approximately 300,000 tons are used annually by municipal and industrial water plants. Iron salts are also employed in substantial quantity. Recent experimentation with coagulant aids may develop new applications of great importance. Cationic organic polyelectrolytes, although not generally approved as yet for public water supplies, may eventually find economical usage for specific removal problems (6).

The coagulation process, usually associated with surface water supplies, involves 1) a short flash-mix period for immediate distribution of the added chemical, 2) formation of a gelatinous floc from the ionized chemical (coagulation), 3) a longer mixing period offering continued opportunity for contact of the floc with colloidal particles in the raw water (flocculation), and 4) a final period of quiescent settling or sedimentation, which allows gravity removal of agglomerated matter. The settled water is much reduced in turbidity, perhaps substantially improved with regard to color, and has undergone some reduction in bacteria and other organisms. The process is followed by filtration.

The "jar test" continues as the most useful mechanism for establishing and maintaining optimum chemical dosage, both in the laboratory for experimentation and design, and in the full-scale plant. Adjustment of pH is often required for efficient operation. In 1962, of the 100 largest cities in the U. S., 66 used only surface water supplies and an additional 14 used a combination of ground and surface waters. Of these 80 facilities, 61 were treated by coagulation and sedimentation.

Of 20 ground water supplies, 7 used coagulation. In New England in 1963, 58 plants used alum coagulation and 3 used iron salts, as reported by the USPHS. (See Tables I and II.)

Taste and Odor Control

Tastes and odors may be imparted to water by decaying vegetation, by algae and protozoa, and by a host of organic and inorganic compounds, some in extremely minute quantities. Since its first practical application at New Milford, N. J., in 1930 (7), the use of activated carbon[2] for adsorption of taste and odor-causing compounds has become standard. Generally the practice has been to add the carbon in a powdered form, or in a slurry, and, after sufficient contact time, to remove the carbon and adsorbed matter in a sedimentation process. Some plants use beds of activated carbon in granular form. As carbon in this form can be regenerated by several processes, it is possible that this type of application may find increasing future usage (4). A continuous dosage in the powdered form of a few parts per million is in many cases sufficient to remove odors and tastes associated with both organic and inorganic matter.

Other chemical additives for control of tastes and odors include chlorine dioxide, sulfur dioxide and ozone. Breakpoint chlorination results in the destruction of chloramines. Aeration may be attempted for removal of odors associated with gases. In New England in 1963, 23 plants utilized activated carbon and 8 used other compounds for taste and odor control. (See Table II.)

Corrosion Control

Of the several chemical stabilization processes used in water treatment, some of those used for corrosion control will be discussed here. Corrosion of metals occurs in the presence of an oxidizing agent and moisture containing an electrolyte. It is important, therefore, in order to control corrosion, to produce and maintain a barrier between water and the metal surfaces with which it comes in contact. All metal pipelines, tanks, meters, valves, hydrants, and other appurtenances are subject to corrosion, and thus must be protected against it. Properly applied, paint serves as an adequate protective film. In much of New England, with its coastal location, the salt-laden atmosphere offers a

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[2] "Activated" carbon is manufactured by closely controlled charring of wood at a temperature below 500°C, followed by slow controlled burning at a temperature near 800°C. Finely divided activated carbon is estimated to provide 3,000,000 square feet of adsorptive surface per cubic foot (8).

real challenge to water works personnel. Metal water storage facilities in this area require a continuing maintenance program—it is about average for such facilities to require repainting every 3 to 5 years. Inland, paint coatings may stand up for 7 years or more, but vigilance for tell-tale signs of corrosion is necessary nevertheless.

Most cast iron pipe is now cement-lined before installation, but many appurtenances need additional protection—usually furnished by chemical treatment. Addition of phosphates in the form of sodium hexametaphosphate has become rather common practice. In 1963 there were 97 such facilities in New England, representing almost 10 percent of all the supplies serving communities of 100 or more. Complex phosphates act as sequestering agents in forming soluble complexes with iron that may be in the water initially, and with iron dissolving at anodic areas in a pipeline. The resulting complex serves to prevent precipitation of iron as tuberculation and "red water" (9). Polyphosphates serve further as corrosion inhibitors apparently by depositing thin films on metal surfaces by adsorption. Usual dosages of metaphosphates are on the order of 2 to 6 mg/1 to maintain a smaller residual (8). (The housewife has made phosphate treatment an ordinary occurrence through the use of "Calgon" as a conditioning agent.)

Adjustment of pH is another important method of stabilization. Water in the acid range serves to promote many corrosion processes. Treatment which promotes the formation and maintenance of a calcium carbonate film in the system is effective in controlling corrosion, but it may not result so much from the film itself as from the fact that a high pH is maintained at the metal surface which interferes with the corrosion process (10). Upward adjustment of pH through the addition of lime or other alkalies, such as sodium hydroxide, is a common method of corrosion control. In New England, 87 facilities included such a process as regular treatment in 1963. Other methods of corrosion control include aeration for reduction of carbon dioxide, deoxygenation for boiler feed water, and addition of chromates to cooling waters.

Iron and Manganese Removal

The 1962 Drinking Water Standards (11) changed the previous recommended limits on iron and manganese in drinking water from a combined concentration of 0.3 mg/1 to 0.3 mg/1 for iron and .05 mg/1 for manganese. Neither has public health significance in commonly encountered concentrations but the colors and stains, and possibly tastes, associated with these elements make their occurrence objectionable. In addition, manganese may interfere with quality control (11). Iron and manganese are frequently problematical in New England ground waters. Treatment for removal of both iron and manganese is similar and often consists of precipitation by oxidizing the compound, as of iron, from the soluble ferrous state to the insoluble ferric state. Aeration, performed by using diffusion nozzles, surface aerators, cascades or sprays, may also be accomplished by contact or catalysis processes. Beds of coke or other crushed material, besides providing opportunity for aeration, may act as satisfactory contact agents, and already-precipitated iron and/or manganese serve as a catalyst for further removal in the beds. Oxidation by other agents, such as potassium permanganate and chlorine, may also be effective. Both iron and manganese compounds may be reduced in water softening precipitation processes and some may be removed in ordinary coagulation. The treatment for removal, however, has seldom been accomplished readily by any process. In 1963 there were 6 contact bed or tray facilities in New England water supply and 14 spray aerators. Other applications are indicated in Table II.

Softening

Calcium and magnesium ions are the usual causative agents of hardness. These ions react with fatty acid radicals in soap to form unwanted precipitates resulting in added expense to the consumer.

Treatment involves a precipitation process, usually from an addition of lime, or lime and soda ash. If bicarbonate is available in sufficient quantity in the raw water, the addition of hydrated lime results in the formation and precipitation of calcium carbonate, and thus the removal of hardness-causing calcium. Otherwise, normal carbonate ions are added directly through the use of soda ash (sodium carbonate). Magnesium is removed as a hydroxide and its removal may be aided by providing an excess of lime. Sedimentation and filtration are necessary finishing operations.

Fortunately, New England communities are not often saddled with extreme hardness problems, although the fact that there were 33 facilities applying lime[3] and 26 applying soda ash in 1963 indicates that some problem exists.

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[3] Some of these may have been for purposes other than softening.

Fluoridation

Since the first installation in 1945, fluoridation of water supplies has been a controversial issue in the United States. Its most rapid period of growth was from 1950 to 1953; this was followed by a period of growing and well-organized opposition (from 1953 to 1962 the communities served by nearly 100 systems in the United States voted to discontinue fluoridation) (12). By the end of 1961, however, according to Maier, almost 2200 communities (42,000,000 persons) were receiving artificially fluoridated water.

Although "practically every physical, mental, or moral illness to which human flesh is heir has been attributed by the opponents of fluoridation at one time or another to drinking fluoridated water"[4] fluoridation has been approved by an impressive list of medical, dental and health authorities throughout the world as a safe and effective means of reducing the occurrence of dental caries.

The optimum dosage ranges from 0.7 to 1.2 mg/1 depending on annual average maximum daily temperature (11). Application does not require a large investment in equipment. Some communities have voted fluoridation in again after having previously voted to discontinue it. On a national basis the most commonly used chemical compound as a source of fluoride, because of its low cost, is sodium silicofluoride, a crystalline powder, for which hydrofluosilicic acid is the raw material. It is usually introduced to the supply through a dry-feeder and solution sequence. Silicofluoride is handled and applied with difficulty. Sodium fluoride, with its unique solubility characteristics, lends itself to almost foolproof application but the chemical is comparatively expensive. Hydrofluosilicic acid, a corrosive liquid, is often applied by pumping directly from the drums used for shipment. In New England, in 1963, there were 66 systems using fluoridation, 39 of which used sodium fluoride as the chemical source and 18 used sodium silicofluoride.

Conclusion

Only the more universal treatment practices have been considered here. Nothing has been mentioned of watershed management, which in itself is really a method of preventive treatment; of reservoir control of tastes and odors through application of chemical algicides such as

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[4] F. J. Maier in reference (12), p. 215.

copper sulfate; of removal of specific ions by ion exchange, and many others.

Gradually, the engineer and the plant operator have developed a scientific and useful approach to old treatment methods and old water quality problems; however, the problem is not static. New waste products entering our streams and the underground reservoir, mushrooming population and demand for water, the necessity for reuse of waste effluents, and recreational use of reservoirs, will soon make the present-day approach to water treatment old-fashioned, outmoded and obsolete. The day of the ordinary water treatment problem is coming to an end—soon every potential water supply will demand an extraordinary approach. The sanitary engineer is thus confronted with a continuing challenge which cannot go unmet. This is the most vital aspect of urban planning for environmental health.

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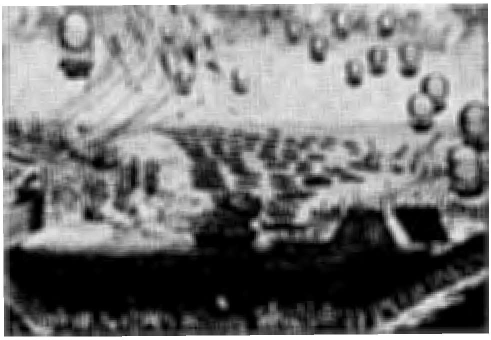
A VIEW FROM DOVER CASTLE: THE ENGLISH CHANNEL TUNNEL

By Frank P. Davidson*

(Presented at a meeting of The Boston Society of Civil Engineers, May 17, 1965.)



In addressing the oldest engineering society in the United States, it is some comfort to reflect that the project which gave rise to your invitation has, itself, a history reaching back more than two hundred years. In 1751, a prize was awarded by the Amiens Academy to Monsieur Desmarets for an outstanding design of a channel tunnel.[1] A half-century later, Mathieu Favier, a young French engineer, had his sketches brought to the attention of Napoleon Bonaparte, who showed them approvingly to Charles James Fox.[2] And in 1878, actual excavation of galleries was commenced by channel tunnel companies formed in the United Kingdom and in France.

After the British government cancelled authorization of the project in 1883, following the vigorous objections of Lord Wolsley, a prolonged debate ensued as to the desirability of a channel tunnel. The matter was not finally settled until the historic exchange of messages between Queen Elizabeth II and General DeGaulle on February 8, 1964.

As many of you are aware, the Channel Tunnel Study Group, under the supervision of the Joint Commission of Surveillance, has been carrying out the final engineering and geophysical survey for the British and French Governments. This work has been under way for many months. As a non-engineer, it would be invidious on my

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* President, Technical Studies, Inc., American participant in the Channel Tunnel Study Group, and member of the governing board of the Study Group.

[1] French Technical Bulletin 7-8, 1964, p. 1.

[2] T. Whiteside, "Tunnel under the Channel," Simon & Schuster, 1961, p. 15.

part to attempt to describe the technical alternatives and problems of this vast and historic undertaking: should any of you have occasion to visit Dover Castle during the summer months, you will readily observe for yourselves the scope and quality of the effort directed from our field headquarters, with its array of ships and special devices.

The results of the geophysical survey which began last summer are still being studied and interpreted. However, the continuity of the lower chalk stratum across the channel has been confirmed, and the limits of this geological layer have been defined with sufficient accuracy to determine the alignment of a bored tunnel. Of course, a further purpose of the supplementary program of investigations has been to obtain sufficient information in order to choose the type of tunnel to be built (bored or immersed). Perhaps both types will be found technically feasible: proposed routes for an immersed tube are under investigation.

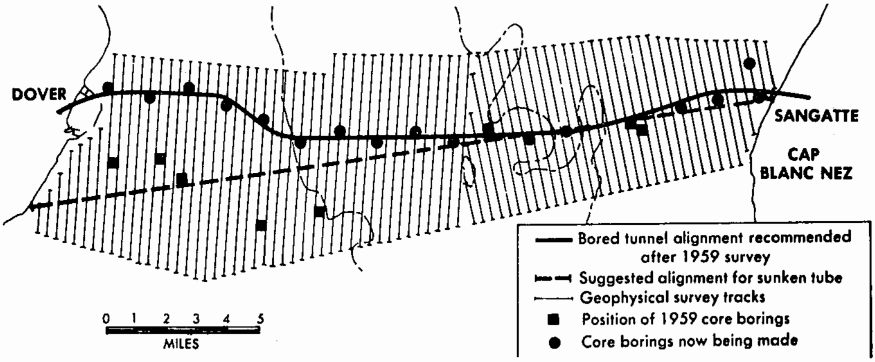

FIG. 1.—POSSIBLE TUNNEL ROUTE AND LOCATION OF EXPLORATORY WORK.

The program of core borings commenced last autumn with several converted tank landing craft, using ex-navy boom defense vessels as mooring vessels for the drilling ships. The accelerated arrangements put in hand for the coming months include a new type of ship belonging to the French Institut de Pétrole, the "Terrebel," which uses a drill controlled by a flexible tube from the surface. In addition, drilling platforms will be employed for the shallower bore-holes and agreements have been concluded for the use, starting in June 1965, of the "GEM 111" and the "NEPTUNE I" drilling platforms. The

"GEM 111" platform belongs to the "Compagnie Générale d'Equipements pour les Travaux Maritimes" (G.E.M.), affiliated with the Hersent and De Long groups. The "NEPTUNE I" platform, which belongs to the Neptune Company, has been put at the disposition of the Channel Tunnel Study Group during a short period on its way to oil-drilling operations for the North Sea French group, through Total Oil Marine Limited.

Consulting Engineers for the Channel Tunnel site investigations are: Sir William Halcrow & Partners; Livesey & Henderson; Rendel, Palmer & Tritton; Société d'Etudes Techniques et Economiques (SETEC) and Société Générale d'Exploitations Industrielles (SOGEI).

The core borings are being accomplished by an Anglo-French joint venture consisting of George Wimpey & Co., Limited, and Forasol. Position fixing equipment has been supplied by Decca Navigator Co., Ltd. Geophysical work is being carried out by scientists from Edgerton, Germeshausen and Grier, Inc., of Boston. Earlier studies have benefited from American as well as European technology. Parsons, Brinkerhoff, Quade & Douglas of New York reported on the feasibility of a bridge or of a combination tube-bridge. Marine Geophysical Services and Alpine Geophysical Associates undertook an early Sparker survey, following a useful sonar survey made by Telephonics Corporation (of Long Island). A basic report on a bored tunnel was prepared by the Bechtel Corporation, Brown and Root, Inc., and Morrison-Knudsen Company, Inc., and a consortium including Raymond International, Inc., Kaiser Engineers and Constructors, Inc., De Long Corporation, Healy Tibbits Construction Co., Macco Corporation, Peter Kiewit Sons' Company, and Tavares Construction Co., Inc. submitted a useful report on an immersed tube. Prior to the decision by the Joint Commission and the Governments in favor of a tunnel, as opposed to a bridge, Steinman, Boynton, Gronquist & London prepared an independent calculation of the cost of constructing a bridge.

Boston has contributed in many ways to the success of the study program. The Sparker survey carried out by Edgerton, Germeshausen and Grier, Inc. has been an oustanding success. American Research and Development Corporation was an early investor in Technical Studies, Inc., and General Doriot himself serves on the company's Board of Directors. Monsieur Arnaud de Vitry, Chairman of Technical Studies, Inc. and a member of the supervisory committee of the Channel Tunnel Study Group, was graduated with high honors from M.I.T. and from the Harvard Business School.

The entire project has been an example of effective international and interprofessional coöperation. Since 1957, the program of studies has been planned and supervised by Monsieur René Malcor, Ingénieur en Chef des Ponts et Chaussées, and Horace J. B. Harding, president of the Institution of Civil Engineers in London. The international consortium reporting on traffic and revenue included De Leuw, Cather & Company, of Chicago, Société d'Etudes Techniques et Economiques (SETEC), of Paris, and the Economist Intelligence Unit, of London.

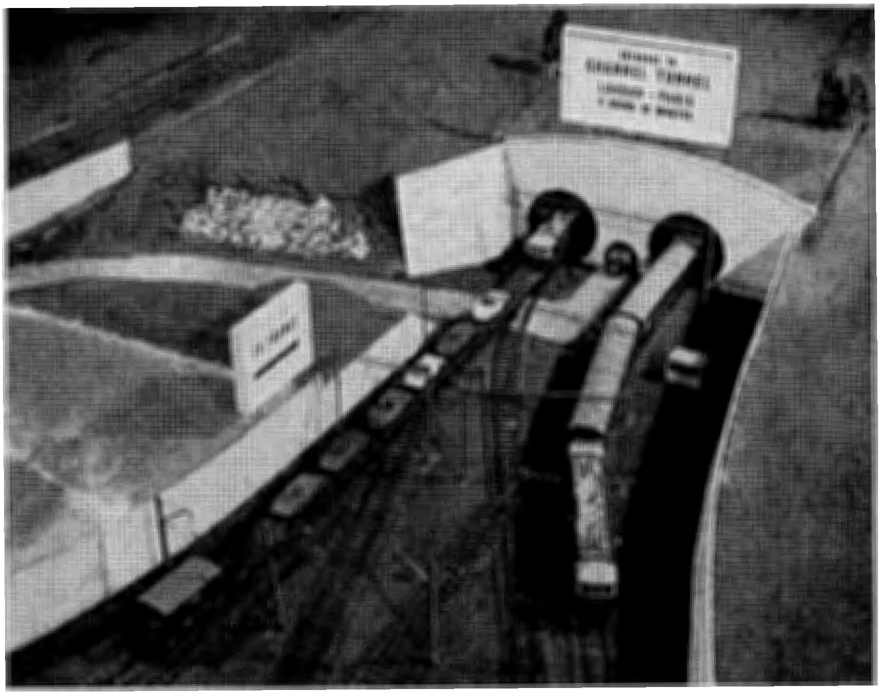

FIG. 2.—THE TUNNEL ENTRANCE. THERE WOULD BE TWO SINGLE TUNNELS WITH A SMALL MAINTENANCE TUNNEL SANDWICHED BETWEEN THEM. THE RAIL TUNNELS WOULD BE LARGE ENOUGH FOR CONTINENTAL TRAINS AND WOULD BE INTERCONNECTED AT SEVEN-MILE INTERVALS. ACTUAL RAIL LAYOUT AT APPROACHES WOULD DIFFER FROM THAT SHOWN IN THIS MODEL. COVERED SINGLE AND DOUBLE-DECK CARRIERS WOULD BE USED TO CARRY CARS AND THEIR PASSENGERS RATHER THAN OPEN VEHICLES AS SHOWN IN THE ILLUSTRATION. MODEL WAS BUILT BY THE BRITISH TRANSPORT COMMISSION'S PUBLICITY DEPARTMENT.

The international Channel Tunnel Study Group itself was formed in Paris during the month of July, 1957, to investigate, with all the resources of modern science, the feasibility of a cross channel link and to assist in its realization should a practical program emerge. The Study Group includes the two companies founded for this purpose in the nineteenth century (the Channel Tunnel Company, Ltd. and the Société Concessionaire du Chemin de Fer Sous-Marin entre la France et l'Angleterre) together with the International Road Federation (Paris office), the Suez Financial Company (formerly the Universal Suez Canal Company), and Technical Studies, Inc., of New York. The European participants benefited from the membership in their companies of governmental elements: the French National Railway System owns a fifty percent interest in the Société Concessionaire; the British Transport Commission is perhaps the principal shareholder of the Channel Tunnel Company and, as a result of Disraeli's famous action in 1875, Her Majesty's Government is the largest single shareholder in the Suez Financial Company.

Although Technical Studies, Inc. is privately held, its former directors include the present United States Undersecretary of State, and the present United States Secretary of the Navy. The Board of Directors today includes the Honorable Lewis W. Douglas, former U. S. Ambassador to the Court of St. James's; the Honorable Warren L. Pierson, former president of the Export-Import Bank; General Georges F. Doriot, former Deputy Director (research and development), War Department General Staff; Lewis A. Lapham, chairman, Executive Committee, Bankers Trust Company; N. Dean Jay, a director of Morgan Guaranty Trust Company of New York; Cyril C. Means, Jr., Esquire, former Arbitration Director of the New York Stock Exchange; Alfred E. Davidson, Esquire, former general counsel of the U. S. Foreign Economic Administration; and the speaker. As mentioned earlier, Monsieur Arnaud de Vitry is chairman of the company.

As the final survey nears completion, and the various arrangements for finance and management are perfected, the project is almost certain to encourage other large transportation links throughout the world. It may be that not the least service rendered by the channel tunnel will be the practical demonstration of a management system conducive to effective collaboration between national governments and the international capital markets.

CONCEPTS & MISCONCEPTIONS CONCERNING THE USE AND ABUSE OF CPM FOR CONSTRUCTION

By Herbert M. Priluck*

(Presented at a meeting of The Construction Section, B.S.C.E., March 24, 1965.)

I. Introduction

Those who expect to find some magic formula in Critical Path Method (CPM) to cure all the ills and pains of constructing a project will be surely disappointed.

CPM, at times, is being used as an advertising gimmick to indicate that whatever is in any way associated with it must, by association, be modern and possessing some mystical power. It recently was amusing to see in a widely read engineering magazine an advertisement proclaiming that X Brand patent forms kept Contractor A on the critical path, followed a few pages later by an advertisement stating that Y Brand power shovel keeps Contractor B off the critical path.

In order to gain an entree into the construction industry, some proponents of CPM have made grandiose claims about the wonders of CPM. Many usually circumspect contractors, architect/engineer firms and owners have been overawed by terms such as complete project control, parametric linear programming, electronic digital computer, management by exception, etc. Were these claims made about any other management tool, such as cost control, or time study, they would surely be received with ridicule rather than awe.

Physically, CPM is an inert series of lines, circles, words and numbers on paper. Any statement that CPM does, can do, or will do some miraculous task must be written off or at least taken with a grain of salt.

In truth, in the hands of a competent construction manager, CPM is a powerful tool which when skillfully applied can yield results not readily obtainable with less sophisticated projected management tools.

It is a new tool developed by the construction department of

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* President, Construction Planning Management Inc., 79 Stearns Road, Brookline, Massachusetts.

Dupont in the late 1950's to insure completion of their in-plant process change-over projects with a minimum of lost production time. The significant feature of CPM, which sets it apart as new, is the separation of planning from scheduling, and the treatment of each in a systematic manner. Thus, the necessity of juggling in one's mind the various parts of the work, their interrelation, the time it takes to do each job making up the project, and its effect on and place in the overall time-scale of the project, is eliminated.

II. Basic Concepts

It is necessary to outline the basic concepts of CPM as applied to construction.

Project planning scheduling and *control* is the function of CPM in construction.

CPM is a *project* management tool. It does not apply to a continuous flow process. A project must be defined as a group of interrelated activities making up a specific overall task which has a distinct start and a distinct finish. For example, CPM can be used to construct a bakery but is not useful in managing the production of cookies ad infinitum. It can be used to manage the formulation (or if need be the dissolution) of a business enterprise but not the on-going functions of the firm.

Planning, in the context of CPM, is the study of the activities comprising a project and their interrelationship, presented in a systematic manner. In the planning stage, a uniform set of symbols are used according to specific rules to represent graphically the plan of action for the project. This graphic representation is the CPM chart or arrow diagram. We are accustomed to such a graphic representation of *what* we will build in the form of architectual, structural and many other drawings. A CPM network is similarly a graphic representation of *how* we will build.

Scheduling, is the second phase of CPM application. A reasonable estimate must be made of the time required to perform each of the activities shown on the CPM chart developed in the planning stage. At this point the "method" aspect of CPM is most evident. With the CPM chart and the estimated time for each activity we can proceed in an entirely mechanical manner through addition and subtraction to determine the overall time required for the project, the earliest time each activity in the project can be started and finished, and the latest time each job within the project can be started and finished without extending the total time of the project. Activities having the minimum (usually zero) difference between the early and late times are critical. These scheduling calculations are valid only if the project is actually built as shown on the CPM chart if the individual jobs are done in the time estimated.

Control conjures up various visions of omnipotence. As used is discussing CPM application, control is a management science term which means no more than comparing what is actually being done with what was planned to be done. Semantically, perhaps, "monitor" would be a better word. As with any control system, CPM enables one to spot difficulties and, in some instances, helps one discover some alternative solutions. It is still, however, up to the management of the project to decide what must be done when unforeseen bottlenecks arise.

III. Aspects of Implementation

A few comments on some common misconceptions regarding CPM in construction are in order, as well as several, perhaps impertinent, remarks on several pertinent questions relating to the implementation of CPM. Let us first consider some current misconceptions.

"A person skilled in the techniques of CPM but having only a superficial knowledge of construction can plan and schedule construction." False! This will only produce a plan of action which will probably result in inaction. An analogy would be to say that because a structural detailer is facile with the graphical symbols used to represent a steel structure, he can do structural engineering.

"An electronic digital computer and a Ph.D. in mathematics is needed." False! To be sure, most, if not all, doctoral candidates equipped with the most advanced data processing equipment can add, subtract and manipulate the simple rules and symbols of CPM. They are "needed" in the same way a sixteen pound sledge hammer is "needed" to drive a 6d nail into a soft pine board. If you wish to present the information on a CPM construction schedule in a series of unrelated one line shots of information sorted out in numerous ways so as to produce reams of useless paper, a computer is indispensable.

"The simplicity of CPM makes it trivial." False! One could compare this to cost control where money spent divided by units of work done equals actual unit cost which is greater than or less than estimated unit cost. Anyone knowledgeable in construction cost control knows that this simple principle is complex in implementation. Similarly, the uncomplex principles of CPM require much thought in actual application to a construction project.

"By concentrating almost exclusively on critical activities, which are a small percentage of the total, one can successfully complete a construction project through 'management by exception.' " False! On a tightly scheduled construction project, especially in buildings, criticality is an accident of arithmetic. The great majority of activities will have less than two weeks leeway. To presume that portions of the work having but a few days leeway are safe, and will somehow take care of themselves, is a fatal mistake. Anyone experienced in construction knows how quickly a couple of weeks can go by in straightening out some of the problems that arise due to the inherent uncertainties of construction.

There are several points relating to the use or abuse of CPM on which I wish to express opinions developed from experience in planning and scheduling construction throughout the eastern U.S. for numerous contracting and engineering firms.

To what extent is CPM actually used? If you wish to count the work done by quantity surveyors between bids, the doodlings of the job engineer confined to the trailer on a rainy afternoon, the arrow diagram displayed on the wall of the main office but never looked at, the use of CPM is extensive. If however, you wish to count only those cases where CPM is used as the true plan for construction, prepared and implemented by a job management team skilled in obtaining the maximum results this system can yield, CPM has barely gained a foothold.

On the negative side, CPM can be a hindrance to reasonable project management. It can be used to observe as well as clarify. It can be used as a club as well as a tool. It can be backward looking, seeking to place or shift the blame for lack of progress rather than forward looking, seeking to bring the project to a successful conclusion. It can be a sales gimmick rather than an objective prequalification tool on negotiated work.

CPM should be specified only when *both* the specifier and *all* bidders are *both knowledgeable* and experienced with CPM, or an adequate provision is made to obtain assistance of someone who is. The result of doing otherwise will be the production of considerable paper which is useless to the owner, architect and engineers, and endured by the contractor only to insure processing of his monthly requisitions.

Requiring a CPM schedule with the bid appears, at first blush, to be a fine idea. The construction estimator does a commendable job in taking off and pricing the general construction work, collecting and analyzing subbids, and evaluating contingencies, overhead, profit, etc. in time to produce a firm bid on the specified date. To add the additional task of preparing a CPM schedule is quite impractical. Further, the great bulk of time, effort and expense would be expended on detail planning of jobs the contractor will never get to actually build.

When a consultant is retained to assist in applying CPM he should be regarded as a sort of part-time or day/labor construction manager who is thoroughly knowledgeable about CPM and who can effectively work with construction personnel in the field and office. In setting up a project he should train the project manager, superintendent and engineers involved so that they can use the tool he provides for them and not become a permanent expense to the client. In addition to providing in depth training for key men in a construction company by working with them on a few jobs, he should be able to conduct an interesting briefing session for those in the firm who need to have an overall understanding of the use of CPM without getting into the finer details of the system.

IV. Conclusion

Despite being somewhat oversold and underused or mis-used, CPM will eventually be accepted and knowledgeably applied by competent experienced construction managers based upon its real merit as a project management tool. Based on a solid foundation in planning, scheduling, and control, construction managers will readily be able to expand their techniques into expediting systems, multi-project operations, resource leveling and other advanced applications of CPM.

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