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PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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PLASTIC STATE OF STRESS AROUND A DEEP WELL

By H. M. Westergaard, Member*

The analysis that follows is a result of conversations with Dr. Karl Terzaghi, who raised this question: What distributions of stress are possible in the soil around an unlined drill hole for a deep well? What distributions of stress make it possible for the hole not to collapse but remain stable for some time either with no lining or with a thin "stove-pipe" lining of small structural strength?

Notation. Let:

z = vertical coördinate, measured downward from the horizontal surface.

r = horizontal radial distance from the axis of z, which is the axis of the cylindrical drill hole.

a = radius of the hole.

b = value of r at the boundary between the regions of plasticity and elasticity; a function of z.

w = weight of the soil or rock with its content of water, per unit of volume, reduced below the water table by the weight of water per unit of volume; though w may be a function of z, the purposes of this analysis are served sufficiently well by assuming w to be a constant.

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$\sigma_r, \sigma_\theta, \sigma_z$ = horizontal radial pressure per unit of area, horizontal circumferential pressure per unit of area, and vertical pressure per unit of area, respectively, minus the hydrostatic pressure of the water in the hole.

τ_{rz} = shearing stress in the directions of r and z; positive when it acts upward on the parts having smaller values of r.

p = value of σ_r at the cylindrical surface of the hole, at $r = a$.

q = constant, appearing in equations (10) and (11) for the plastic state, measurable as a pressure in pounds per square inch.

k = ratio, appearing in equations (10) and (11) for the plastic state.

F, f, Z = stress functions; F and f are functions of both r and z; Z is a function of z only.

Equations of Equilibrium. Whether the material is in an elastic or plastic state, the stresses must obey the following two equations of equilibrium, which are derived by considering the forces on a small wedge-shaped block of the dimensions dr, $rd\theta$, dz*:

$$\frac{\partial(r\sigma_r)}{\partial r} - \sigma_\theta + \frac{\partial(r\tau_{r\theta})}{\partial z} = 0 \tag{1}$$

$$\frac{\partial(r\sigma_z)}{\partial z} + \frac{\partial(r\tau_{r\theta})}{\partial r} = wr \tag{2}$$

Stress Functions. It is possible to express the four stresses in equations (1) and (2) in terms of two stress functions F and f. No matter what functions F and f are chosen, if they permit the differentiations called for, the following stresses will be found to satisfy the equations of equilibrium, (1) and (2):

$$\sigma_r = wz + \frac{1}{r}\frac{\partial^2 F}{\partial z^2} + \frac{f}{r} \tag{3}$$

$$\sigma_\theta = wz + \frac{\partial f}{\partial r} \tag{4}$$

*Compare, for example, S. Timoshenko, "Theory of Elasticity," (McGraw-Hill Book Company, Inc.) 1934, p. 309.

$$\sigma_z = wz + \frac{1}{r}\frac{\partial^2 F}{\partial r^2} \tag{5}$$

$$\tau_{rz} = -\frac{1}{r}\frac{\partial^2 F}{\partial r \partial z} \tag{6}$$

Solution under Conditions of Elasticity. If the soil or rock could be counted on to obey Hooke's law for homogeneous, isotropic material, the following stress functions would be found applicable:

$$F = 0, f = -wza^2/r \tag{7}$$

The corresponding stresses,

$$\sigma_r = wz(1 - a^2/r^2), \sigma_\theta = wz(1 + a^2/r^2), \sigma_z = wz, \tau_{rz} = 0 \tag{8}$$

are of the type represented in Lamé's formulas for thick-walled cylinders; and they satisfy not only the equations of equilibrium, (1) and (2), but also the equations of compatibility of deformations.* The most severe combination of stresses defined at each depth by equations (8) occurs at $r = a$ and is

$$\sigma_r = 0, \sigma_\theta = 2wz, \sigma_z = wz, \tau_{rz} = 0 \tag{9}$$

Plasticity. The combination of pressures in equations (9) could not be maintained at a great depth. Before such a state could be reached, the material would either break or flow toward the hole. The flowing indicates a plastic state which relieves the stresses. It may be assumed then that within a radius $r = b$, dependent on z, the material is in a plastic state, but that outside this radius it remains in an elastic state so far as changes of stresses are concerned.

Let σ_1, σ_2, σ_3 denote the three principal pressures in the state of stress at a point, in the order of rising magnitude. It will be assumed for the purpose of this analysis that the law of plasticity can be stated in terms of these stresses by the formula†,

$$\sigma_3 - (k + 1)\sigma_1 = q \tag{10}$$

with the values of k and q constant. The value of k may be as much as 3. Equation (10) is equivalent to the statement that the limiting curve for Mohr's circles is a straight line. It will be assumed further that an elastic state obeying Hooke's law, with unchanged values of

*S. Timoshenko, loc. cit., p. 56, p. 312.

†Compare, "A study of the Failure of Concrete under Combined Compressive Stresses," by Frank E. Richart, Anton Brandtzaeg, and Rex L. Brown, University of Illinois Engineering Experiment Station, Bulletin No. 185, 1928, 102 pp., especially pp. 78, 79, and 91.

the modulus of elasticity and Poisson's ratio, is possible when the left side of equation (10) is less than q. Finally it will be assumed that there are no sudden jumps in the stresses at the boundary $r = b$ between the regions of plasticity and elasticity.

Solution. The following assertions require verification, which is obtained after conclusions have been drawn from them: First, F will be such a function that at any depth that is not relatively small the term $\partial^2 F/\partial z^2$ in equation (3) will be relatively so insignificant that it may be ignored. Secondly, F may be determined so that τ_{rz} in equation (6) will vanish at $r = a$ and be relatively insignificant everywhere. Thirdly, F may be determined so that in addition at each place σ_z will lie between σ_r and σ_θ.

When these assertions are accepted, it becomes possible to state that at each point σ_r, σ_z, σ_θ are the three principal pressures in the order of rising magnitude. Then equation (10) takes the form

$$\sigma_\theta - (k + 1)\sigma_r = q \tag{11}$$

With the term $\partial^2 F/\partial z^2$ omitted in equation (3), the pressures σ_r and σ_θ will depend on f only. The following form of f will be examined for the region of plasticity:

$$f = (p + q/k)r^{k+1}/a^k - (wz + q/k)r \tag{12}$$

Substitution in equations (3) and (4) gives

$$\sigma_r = (p + q/k)(r/a)^k - q/k \tag{13}$$
$$\sigma_\theta = (k + 1)(p + q/k)(r/a)^k - q/k \tag{14}$$

Equation (13) gives the desired value $\sigma_r = p$ at $r = a$; and the two equations (13) and (14) satisfy the requirement of plasticity in equation (11). The pressure p may have to be assumed to be zero; but under some circumstances a small positive pressure p may perhaps be credited either to whatever strength the lining has, or to a "mud-head" added to the water pressure.

For the region of elasticity f may be taken as

$$f = -Za^2/r \tag{15}$$

with Z to be determined as a function of z. Substitution from equation (15) in equations (3) and (4) gives

$$\sigma_r = wz - Za^2/r^2, \sigma_\theta = wz + Za^2/r^2 \tag{16}$$

If dZ/dz varies only slowly with z, and if σ_z remains close to wz in this region, these pressures will conform sufficiently to the type represented in equation (8), and may therefore be accepted as stresses in the region of elasticity.

At the boundary $r = b$ between the regions of plasticity and elasticity the pressures should have the same values whether computed from equations (13) and (14) or from equations (16). By specifying coinciding values of $\sigma_\theta + \sigma_r$ one finds

$$\left(\frac{b}{a}\right)^k = \frac{2(kwz + q)}{(k+2)(kp + q)} \tag{17}$$

and similarly, by specifying coinciding values of $\sigma_\theta - \sigma_r$ one finds

$$Z = \tfrac{1}{2}(kp + q)\left(\frac{b}{a}\right)^{k+2} = \frac{kwz + q}{k+2}\left(\frac{b}{a}\right)^2 \tag{18}$$

At the boundary $r = b$ one finds now, either by equations (13), (14), and (17), or by equations (16) and (18):

$$\sigma_r = (2wz - q)/(k+2) \tag{19}$$

$$\sigma_\theta = [2(k+1)wz + q]/(k+2) \tag{20}$$

The circumferential pressure σ_θ in equation (20) is the greatest pressure at the depth z.

It remains to verify the assertions that were made concerning the stress function F. This may be done by drawing diagrams for σ_r and σ_θ and thereafter estimate diagrams for σ_z showing σ_z as an intermediate pressure. The values of σ_z determine $\partial F/\partial r$ through equation (5) except for an integration constant, which is defined by the condition $\tau_{rz} = 0$ at $r = a$. Results of such a procedure were found to support the assertions referred to.

Conclusion. The plastic action makes it possible for the great circumferential pressures that are necessary for stability to occur not at the cylindrical surface of the hole but at some distance behind the surface, where they may be combined with sufficiently great radial pressures. The formulas that have been derived serve to explain the circumstances under which the drill hole for a deep well may remain stable.

DYNAMIC PILE DRIVING FORMULAS

By A. E. Cummings*

For well over 100 years engineers have attempted to derive dynamic pile driving formulas which would serve to establish the static bearing capacity of piles. Some of these formulas are relatively simple and others are fairly complicated. Practically all of them are based on a simple energy equation which may be stated in mathematical form as follows:

$$Wh = Rs \tag{1}$$

The left-hand side of this equation represents the energy available in a hammer of weight W which has fallen a distance h. It is customary to define R as the "resistance to penetration" and s as the "distance the pile penetrates under one hammer blow". The product Rs is the work done in driving the pile a distance, s, against a resistance, R. These definitions of R and s contain certain implied assumptions as to the nature of these quantities. The definition of s does not state whether it is the permanent penetration or the maximum penetration. It is easy to measure the permanent penetration and this is done on all pile driving jobs. The maximum penetration, which includes the temporary elastic compression of the pile and the soil, cannot be measured without special apparatus. It is only in very rare cases that this quantity is measured on a pile driving job. The definition of R implies one of two assumptions. Either, it means that R is constant throughout the full depth of penetration; or, it means that R is the average value of a variable resistance to penetration.

This phase of the pile driving problem can be more readily understood from a graphic representation of the relationship between resistance and penetration under a single hammer blow. In Fig. 1, resistance is plotted on the horizontal axis and penetration is plotted on the vertical axis. Fig. 1 (a) is a graphic representation of Equation (1). The penetration is assumed to be a definite quantity ($0s$) and the resistance is assumed to be uniform for the full depth of penetration. This graph is a work diagram and the shaded area repre-

*Raymond Concrete Pile Company, Chicago, Illinois.

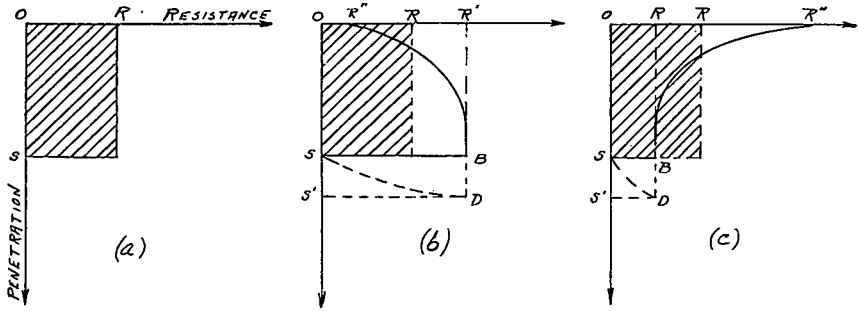

FIG. 1.

sents the work done in moving the pile a distance s against a resistance R. This work, Rs, would be equal to the work, Wh, that is available in the hammer at the bottom of the stroke, provided there were no energy losses.

In actual pile driving, the resistance-penetration diagram would not look like Fig. 1 (a), because there is always some temporary elastic compression of the pile and the surrounding soil. As far as resistance to penetration is concerned, there is very little information available about this. However, the probability in favor of a variable resistance is much greater than that in favor of a constant resistance. There could be only one possible constant resistance but the possible number of variable resistances is practically infinite. Figs. 1 (b) and (c) are resistance-penetration diagrams showing the temporary elastic compression together with two possibilities of variable resistance. The shaded area of Fig. 1 (a) has been superimposed on the other two figures to show how actual pile driving differs from the assumptions on which Equation (1) and Fig. 1 (a) are based.

Referring to the problem of resistance, Fig. 1 (b) assumes that the initial resistance, R'', is very small and that the resistance increases with the depth of penetration until it reaches a fairly constant value of R'. Fig. 1 (c) assumes a high initial resistance, R'', which decreases with the depth of penetration until it reaches a fairly constant value of R'. In either case, the resistance R' at the end of penetration is not the same as R and the two quantities are related by the equation

$$R' = CR \tag{2}$$

in which C is a coefficient that would be more or less than 1.0 depend-

ing on whether the actual diagram were like that of Fig. 1 (b) or (c). It is suggested that Fig. 1 (b) might apply to a pile driven into sand in which the resistance would increase as the moving pile compacted the sand. It is also suggested that Fig. 1 (c) might apply to a pile driven into clay. The high initial resistance would be due to the well known phenomenon of "set-up" that occurs in clays during a temporary interruption of driving. In any case, the best measure of the probable future bearing power of the pile will be the comparatively uniform resistance, R'.

Referring to the question of penetration, Figs. 1 (b) and (c) show the permanent penetration, s, and the maximum penetration, s'. The distance ss' on the penetration axis represents the temporary elastic compression of the pile that occurs during impact. This temporary elastic compression produces an energy loss and the amount of this energy loss is represented by the triangular areas sBD on the diagrams.

From Figs. 1 (b) and (c) it can be seen how Equation (1) must be modified in order that it may represent the dynamics of pile driving. To take the variable resistance into account, a coefficient, C, must be applied to R. To take the energy losses into account, a term for these must be included. The revised equation would then be

$$Wh = CRs + Q \tag{3}$$

where Q represents all energy losses that occur during impact.

A rational approach to the dynamics of pile driving could be made by means of work diagrams such as these. In order to put numerical values on the diagrams it would be necessary to make accurate field measurements on actual pile driving jobs and this is by no means easy. When special apparatus is used to measure the temporary elastic compression in the field, it should be kept in mind that neither the frame of the pile driver nor the ground immediately in front of the pile driver can be considered as fixed reference points. The quantity to be measured is quite small and it cannot be measured accurately if the recording device is influenced by the vibration of the pile driver frame or by the movement of the ground that takes place when the hammer strikes. A high speed motion picture camera equipped with a telescopic lens and set up at a considerable distance from the pile driver should be able to make an accurate space-time record of the behavior of the pile during driving.

As far as the resistance to penetration is concerned, it is prac-

tically impossible to measure the resistance itself. However, if the space-time graph is accurately recorded, it can be used to determine the manner in which the resistance varies with the penetration. A velocity-time curve can be deduced from the space-time curve by graphical methods. In a similar manner, an acceleration-time curve can be deduced from the velocity-time curve. With the acceleration known, it is possible to calculate the resisting force at any point along the path of penetration. Experiments[1] with small copper cylinders under the impact of a falling weight have indicated a variable resistance of the type shown in Fig. 1 (b). This experimental work was done in a laboratory with accurate photographic instruments. Field experiments[2] have been made with wood piles and drop hammers to determine the space-time graphs in actual driving. However, the accuracy of these test results is open to question because the measuring apparatus was supported on the ground immediately in front of the pile driver and braced against the pile driver frame.

In spite of the fact that field measurements and work diagrams represent the most rational approach to the dynamics of pile driving, relatively few engineers have used this method. A. M. Wellington derived the Engineering News Formula on the basis of his experience and a work diagram similar to Fig. 1 (c). Practically all of the other dynamic pile driving formulas that are to be found in engineering literature have been derived by means of mathematics and theoretical mechanics. Equation (1) is used as a starting point and the derivation of the pile driving formula is based on assumptions about the energy losses that occur during impact. The great number of pile driving formulas that can be found in engineering literature is an indication of the wide variety of assumptions that have been made concerning these energy losses.

It is not the purpose of this paper to discuss the question of safety factors. Accordingly, all formulas will be written in terms of the so-called ultimate load. Furthermore, it will be assumed that the reader is familiar with the fact that the dimensions of all quantities must be chosen in such a way that the formulas will be dimensionally homogeneous.

Some formulas are based on the assumption that the elastic com-

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[1]B. W. Dunn—"A Photographic Impact Testing Machine for Measuring the Varying Intensity of an Impulsive Force"—Journal of the Franklin Institute, Vol. 144, 1897, page 321.

[2]E. P. Goodrich—"The Supporting Power of Piles", Transactions, American Society of Civil Engineers, Vol. 48, 1902, page 180.

pression of the pile is the only energy loss that needs to be considered. The formula for the strain energy of a compressed strut is taken from static theory and is inserted in Equation (1) as follows:

$$Wh = Rs + \frac{R^2L}{2AE} \tag{4}$$

In the last term of Equation (4), L is the length of the pile, A is the cross-sectional area of the pile and E is the Young's Modulus of the pile material. Stated in words, Equation (4) says that some of the hammer energy is used up by the temporary compression of the pile and that the remainder of the energy is available to drive the pile a distance, s, against the resistance, R. When Equation (4) is solved for R, it yields the following pile driving formula that is usually attributed to Weisbach (about 1850).

$$R = -\frac{sAE}{L} + \sqrt{\frac{2WhAE}{L} + \left(\frac{sAE}{L}\right)^2} \tag{5}$$

The use of the last term in Equation (4) is open to serious criticism on at least two counts. First, the expression is taken from static theory and it is well known that the elastic compression under impact is something entirely different from the elastic compression due to a static force. Second, the expression is derived on the assumption that all of the resistance, R, is applied at the point of the pile. When part of the resistance is applied along the sides of the pile, the expression becomes invalid. In order to take into account the fact that part of the resistance might be developed along the sides of the pile, Rankine proposed the use of half of the actual pile length as the effective length in Equation (4). However, it is not to be expected that the temporary elastic compression can be calculated with any reasonable degree of accuracy by means of an expression taken from static theory without modification for use in a dynamic problem.

Other writers have assumed that pile driving is an impact problem that should be handled on the basis of the impact theory established by Newton. As is well known, the Newtonian theory of impact involves a coefficient of restitution which is used to determine the loss of energy that occurs during the impact of two more or less elastic bodies. For the type of collision known as direct central impact, the Newtonian theory gives the following equation for the energy loss:

$$Q = \tfrac{1}{2}(1 - n^2)\frac{Mm}{M+m}(V - v)^2 \tag{6}$$

In this equation, Q is the energy loss; n is the coefficient of restitution; M and m are the masses of the colliding bodies and V and v are their velocities. If M and V refer to the hammer, then $M = W/g$ and $V^2 = 2gh$, where g is the acceleration of gravity. If m and v refer to the pile, then $v = o$ and $m = P/g$, where P is the weight of the pile. The substitution of these values in Equation (6) gives

$$Q = Wh\frac{P(1 - n^2)}{W + P} \tag{7}$$

If the energy loss represented by Equation (7) is added to the right-hand side of Equation (1), the result is

$$Wh = Rs + Wh\frac{P(1 - n^2)}{W + P} \tag{8}$$

and this may be reduced to

$$Rs = Wh\frac{W + n^2P}{W + P} \tag{9}$$

Equation (9) has been used as the basis of several well known pile driving formulas. About 1820, Eytelwein published his formula

$$R = \frac{Wh}{s\left(1 + \frac{P}{W}\right)} \tag{10}$$

which is obtained from Equation (9) by assuming perfectly inelastic impact ($n = o$). About 1850, Sanders published his formula

$$R = \frac{Wh}{s} \tag{11}$$

which is obtained from Equation (9) by assuming perfectly elastic impact ($n = 1.0$).

Equations (10) and (11) are both based on the fundamental assumption that pile driving is a problem in Newtonian impact. In connection with this assumption it is of considerable interest to note that Newton[3] himself excluded from his impact theory the case of ". . . bodies . . . which suffer some such extension as occurs under

[3]I. Newton—"Philosophiae Naturalis Principia Mathematica." Third Edition (1726), Scholium to Corrollary VI, p. 24.

the strokes of a hammer." Newton deduced his impact theory as a part of the proof of his third law of motion. However, it is evident from his own words that he did not intend the theory to be applied to an impact problem of the type represented by pile driving. Newton's impact experiments were made with spheres suspended as pendulums. When the spheres were at rest they were just tangent to one another. One sphere was then pulled away and allowed to swing in an arc and strike the stationary sphere. The movements of both spheres after impact were carefully measured. These spheres were free bodies except for the restraints produced by the strings on which they were suspended. These restraints affected only the path in space which the spheres had to follow. There were no restraints that could contribute to the elastic distortions of the spheres themselves. In his explanation of his third law of motion, Newton[4] mentions two colliding bodies and gives his rules for their behavior provided that ". . . the bodies are not hindered by any other impediments." In the pile driving problem, the movement of the pile is hindered by external reactions from the surrounding earth and the conditions are not the same as those under which Newton made his experiments with spheres.

The effect of external restraints has been investigated[5] in connection with the problem of the transverse impact of beams. Heavy spheres were allowed to strike transversely against a simply supported beam and the subsequent movements of the beam and the sphere were measured. The elementary impact theory of Newton could not account for the results. The reason for this is easy to understand. Whereas the stationary sphere in Newton's experiment could move after impact as a free body, the beam in Hodgkinson's experiment could not. The supports restrained the ends of the beam so that the velocity of the ends was zero. In order to account for his experiments, Hodgkinson had to modify Newton's equations. He found that he had to use one-half of the mass of the beam instead of the actual mass. Subsequently, this problem was investigated mathematically by Homersham Cox.[6] By considering the strain energy of the bent beam, Cox was able to analyze the problem and he found a mass reduction factor of 17/35 as compared with Hodgkinson's ½.

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[4]I. Newton—"Philosophiae Naturalis Principia Mathematica." Third Edition (1726), Law III, p. 14.

[5]Eaton Hodgkinson—"Impact on Beams." Report of the Fifth Meeting of the British Association in 1835. London, 1836, p. 93.

[6]H. Cox—"On Impacts on Elastic Beams." Cambridge Philosophical Transactions (1849), Vol. IX, Part I, p. 73.

Insofar as pile driving is concerned, these experiments show that a dynamic pile driving formula cannot be based on simple Newtonian impact as is done in the derivation of Equation (10). The restraining effect of the earth surrounding the pile is sufficient to put the pile driving problem beyond the scope of the elementary Newtonian theory. Furthermore, Equation (10) is a dangerous formula to use for very light piles driven with heavy hammers. The P/W ratio in the denominator is the only energy loss that is included in the equation. As P becomes very small in relation to W, this factor tends to vanish. Equation (10) would therefore approach Equation (11) in which there is no allowance for energy loss of any kind. As a matter of fact, when very light piles are driven with heavy hammers, there is a very considerable energy loss due to temporary elastic compression of the pile.

Some writers have published pile driving formulas that are based on the following expression:

$$Wh = Rs + \left[Wh \frac{P(1-n^2)}{W+P} \right] + \left\{ \frac{R^2L}{2AE} + \frac{R^2L'}{2A'E'} + K \right\} \quad (12)$$

In this equation, L' is the length of the driving cap, A' is the cross-sectional area of the driving cap, E' is the Young's Modulus of the driving cap material and K is the temporary elastic compression of the soil surrounding the pile. The other letters have the meanings previously stated. This equation is often referred to as the "complete" pile driving formula, and it appears to have originated with Redtenbacher.[7]

Although Equation (12) has been used for many years as the basis of certain pile driving formulas, it does not appear to have been noticed that the expression cannot be a true equation. This is readily understood from a consideration of the meaning of the several terms that appear in the expression. The term on the left (Wh) is the total energy available in the hammer at the bottom of the stroke. The first term on the right (Rs) is the energy used in moving the pile a distance s against a resistance R. The bracketed terms are energy losses.

The term in square brackets is the Newtonian energy loss derived from Equation (6). In Newton's experiments, both spheres were elastically distorted during the collision and a small amount

[7]F. Redtenbacher—"Prinzipien Der Mechanik und des Maschinenbaues"—1859.

of heat was generated in the spheres. However, Newton did not attempt to analyze the problem by computing elastic distortions or any other *particular* kind of energy losses. He based his theory of impact on what is now called the coefficient of restitution and, by definition, the coefficient of restitution includes *all* of the energy losses that occur in a given case of Newtonian impact. The term in curled brackets in Equation (12) refers to particular energy losses in the form of elastic distortions but these are already included in the Newtonian term. Accordingly, the term in square brackets and the term in curled brackets are mutually exclusive and one or the other of them should be eliminated. When both terms are included, some of the energy losses are being subtracted twice.

Pile driving formulas derived from Equation (12) are usually fairly complicated. However, they appear to be authoritative because they seem to have taken everything into account. Actually, such formulas take too much into account because some of the same energy losses are included in both of the bracketed terms. Furthermore, Equation (12) is based on the same questionable assumptions as are Equations (4) and (8). The term in curled brackets assumes that it is possible to calculate the temporary elastic compressions from the expressions contained within the brackets. The term in square brackets assumes that inertia losses can be calculated by the elementary Newtonian impact theory. As a matter of fact, the temporary elastic compressions cannot possibly be computed with any reasonable degree of accuracy by means of those expressions in the curled brackets. Neither can the term in square brackets give a proper answer as to the effect of inertia.

It is true, of course, that some of the hammer energy is dissipated in producing the temporary elastic compressions. It is also true that the inertia of the pile is a factor in the problem although it does not play as important a part in pile driving as it does in the Newtonian impact theory. Furthermore, it is the writer's considered opinion that engineers have paid too much attention to inertia and too little attention to other phases of the problem that are at least as important as inertia if not more so. Pile driving is far more closely related to the St. Venant-Boussinesq[8] theory of the longitudinal impact of rods than it is to the Newtonian theory of the impact of spheres. This theory of the longitudinal impact of rods is largely

[8]J. Boussinesq—"Application des Potentiels", p 508.

concerned with the transmission of stresses within the pile and the British Building Research Board[9] has recently demonstrated the fact that the theory can give reasonably accurate values for the stresses that actually occur in a full sized pile during driving. The experiments were made with precast concrete piles in which small quartz crystals were cast at several points along the length of the pile. By means of a piezo-electric strain recorder connected to these quartz crystals, a photographic record was made of the strains in the concrete during driving. The stresses in the concrete were then deduced by combining the mathematical theory with the strain gauge records. The determination of these stresses is of considerable practical importance especially in cases where there is danger of damage to the pile due to hard driving. Of equal importance from the standpoint of practical pile driving, is the fact that the theory demonstrates a relationship between stress transmission and inertia.

The analysis of St. Venant and Boussinesq is concerned with two distinct cases of longitudinal impact; (1) the rod fixed at one end and struck longitudinally at the other and (2) the rod free at one end and struck longitudinally at the other. Since the pile is never free and is always restrained by the surrounding soil, no further mention will be made of case (2). The case of the rod fixed at one end and struck at the other is illustrated in Fig. 2. The origin

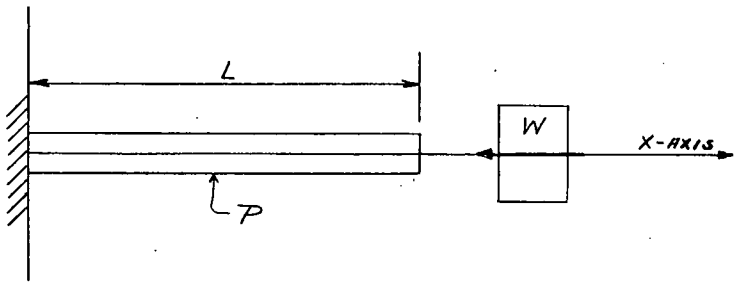

FIG. 2.

of co-ordinates is at the fixed end and the longitudinal axis of the rod is the x-axis. When the weight, W, strikes the free end, a wave of compression starts along the rod toward the left with a velocity

$$a = \sqrt{\frac{Eg}{w}} \tag{13}$$

[9]Glanville, Grime, Fox and Davies—"An Investigation of the Stresses in Reinforced Concrete Piles During Driving"—Building Research Technical Paper No. 20, London, 1938.

where a is the velocity of the stress in the rod, g is the acceleration of gravity and E and w are, respectively, the Young's Modulus and the unit weight of the material of which the rod is made. The equation of motion of an elementary length, dx, of the rod is the standard wave equation

$$\frac{\partial u}{\partial t^2} = a^2 \frac{\partial u}{\partial x^2} \tag{14}$$

in which t is the time, u is the longitudinal displacement of the element dx from its position of equilibrium and a is the velocity of stress in the rod. The general solution of Equation (14) is

$$u = f(at - x) + F(at + x) \tag{15}$$

where f and F are arbitrary functions. The problem consists in determining the functions f and F for various intervals of time after the beginning of impact.

When the first wave of compression traveling down the rod reaches the fixed end, it is reflected there and returns to the free end. The time required for the wave to complete one cycle from the free end and back to the free end is $2L/a$. The values of f and F are determined for the first interval $2L/a$ by means of the terminal conditions of the rod and the initial conditions that occur at the beginning of impact. The solution then proceeds as a step by step process and the values of f and F are determined for successive time intervals that differ from one another by $2L/a$. When the functions f and F have been determined, it is possible to calculate the displacement and the stress at any cross section of the rod at any time after the beginning of impact.

The maximum compressive stress in the rod occurs at the fixed end. Its magnitude depends on the elastic properties of the rod, the ratio of the weight of the striking body to that of the rod and the velocity of the striking body at the instant of impact. When the ratio of the weight of the striking body to that of the rod is less than 5, the theory[10] gives the following equation for the maximum stress at the fixed end:

$$p_{max} = \frac{2EV}{a}\left(1 + e^{-\frac{2P}{W}}\right) \tag{16}$$

In Equation (16), E is the Young's Modulus of the rod, a is the

[10] J Boussinesq—"Application des Potentiels", p. 545.

velocity of stress in the rod, V is the velocity of the striking body at the instant of impact, P is the weight of the rod, W is the weight of the striking body and e is the base of the Naperian logarithm. Hereafter, in this discussion, the striking body will be called the hammer and the rod will be called the pile.

As developed by St. Venant and Boussinesq, this theory is based on certain assumptions and some of these are not fulfilled in practical pile driving. These assumptions are:

(1) That the sides of the pile are free and that there is no side friction which would affect the stress waves running up and down the pile.
(2) That stress waves in the hammer may be neglected.
(3) That there are no flexural vibrations of the pile.
(4) That the pile behaves as a linearly elastic rod.
(5) That the hammer strikes directly on the head of the pile and that the surfaces of contact are two ideal smooth parallel planes.
(6) That the lower end of the pile is fixed.

In addition to these assumptions, the theory does not include the effect of dissipation of energy due to propagation losses in the pile.

Referring to assumption (1), skin friction would tend to reduce the amplitudes of the stress waves traveling in the pile and this would reduce the stresses themselves. Propagation losses in the pile would also reduce the stresses. The effect of neglecting both the skin friction and the propagation losses is therefore on the safe side and the theoretical stresses will be higher than the actual stresses. As to assumption (2), the hammer is usually a heavy block of iron or steel and, for all practical purposes, it can be considered as a rigid body.

As far as assumption (3) is concerned, it has been demonstrated[11] that flexural buckling of a foundation pile under static loads is a remote possibility even in very soft soils. The same thing would apply to dynamic loads as long as the pile and the hammer were in good alignment and the force of the hammer blow was concentric with the longitudinal axis of the pile. In practical pile driving, the pile and the hammer are not always in perfect alignment so that the hammer blows are often slightly eccentric. In extreme cases, piles have actually been broken because of misalignment and the eccen-

[11]A. E. Cummings—"The Stability of Foundation Piles Against Buckling Under Axial Load", Proceedings of the Highway Research Board, Vol. 18, Part II, p. 112.

tricity of the hammer blow. However, for the amount of misalignment and eccentricity that would be tolerated on an actual pile driving job, it can be shown that the bending stresses are relatively small and that the strain energy of bending is only a small percentage of the energy supplied by the hammer. Accordingly, no serious error is involved by the neglect of flexural vibrations in the application of the theory of longitudinal impact to pile driving.

Assumption (4) is reasonably valid for most of the types of piles that are used commercially. Composite piles or any other kinds of piles composed of two or more separate sections do not satisfy this assumption. The transmission of stresses across the joints of such piles is a special problem that is not included in the analysis.

Referring to assumption (5), almost all practical pile driving is done with some sort of cushion or driving block between the hammer and the pile head. The effect of such an object is to reduce the stresses so that the actual stresses are less than those given by the theory. As to assumption (6), the point of the pile is hardly ever fixed in the sense required by the theory. The resistance at the pile point is more or less elastic depending on the nature of the soil into which the point of the pile is being driven. The elastic reaction at the pile point leads to smaller values of the stresses than are obtained from the theory which is based on zero displacement at the fixed end.

Because of these various assumptions, any stresses calculated from Equation (16) will be considerably higher than the actual stresses. However, in order to get a closer approximation to the actual stresses, the engineers of the British Building Research Board made certain changes in the assumptions. They used assumptions (1), (2), (3) and (4) without modification. In place of assumption (5), they introduced into the analysis a term to take into account the elastic behavior of the cushion block. They also modified assumption (6), by assuming an elastic reaction at the pile point and then solving the equations subject to the boundary condition that the pressure at the pile point was proportional to the downward movement of the point. On the basis of these modified assumptions, they developed a complete solution of the differential Equation (14). This complete solution includes long and complicated mathematical expressions so that its use for a practical problem would involve laborious numerical calculations. In order to avoid this wherever possible, the

engineers of the British Building Research Board developed approximate solutions for certain specific conditions.

For the purpose of this discussion, it is sufficient to consider one of these approximate solutions in which the maximum stress in the pile is given by

$$p_{max} = \frac{EV}{a}\sqrt{\frac{W}{P\left(1+\frac{E}{TL}\right)}} \tag{17}$$

In this equation, T is a stiffness constant that refers to the cushion block and the other letters have the meanings previously stated. Because of the approximations involved in the derivation of Equation (17), its use is limited to heavy hammers, soft cushion blocks and short piles that are driven against a practically rigid bottom. The approximate accuracy of Equation (17) can be determined from the expression

$$\frac{P}{3W\left(1+\frac{E}{TL}\right)} \tag{18}$$

which represents the ratio of terms neglected to terms retained.

In order to demonstrate the significance of this theory of longitudinal impact, the pile driving problem shown in Fig. 3 will be

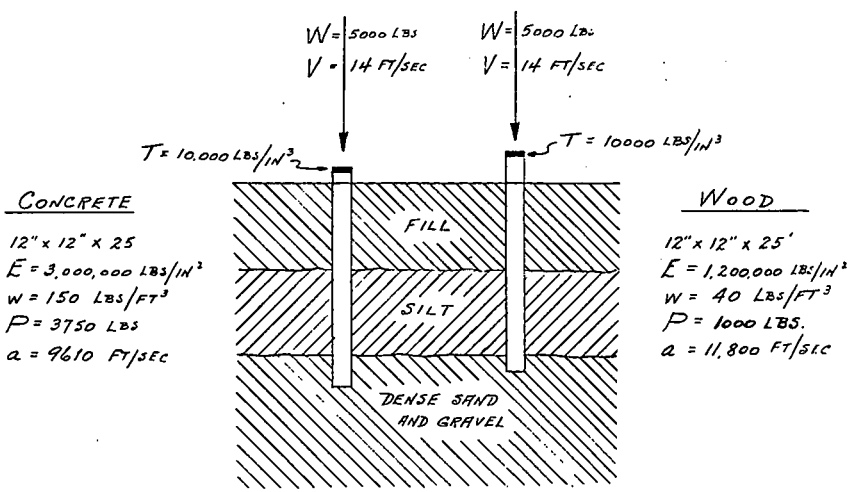

FIG. 3.

discussed numerically. A timber pile and a precast concrete pile are to be driven side by side as shown. Both piles are driven with a single acting steam hammer having a 5,000 pound ram falling 3 feet. The dimensions and properties of the piles are shown on the sketch and the velocities, a, are calculated from Equation (13). In the St. Venant-Boussinesq analysis, no cushion block is included and T does not appear in Equation (16). In the modified theory developed by the engineers of the British Building Research Board, a cushion block is included so that a numerical value for T is required for use in Equation (17). According to experiments made by these engineers, a value of 10,000 pounds per cubic inch for T would represent as soft a cushion block as would ordinarily be used in practical pile driving. The inaccuracy involved in using Equation (17) for the problem shown in Fig. 3 is determined by substituting the necessary numerical values in Equation (18). For the precast concrete pile the inaccuracy is 12.5% and for the timber 4.75%.

Calculation with Equation (16) shows that, in the case of the timber pile, the maximum unit stress at the pile point is about 4,750 pounds per square inch and the total force is about 685,000 pounds. For the precast concrete pile, Equation (16) gives a maximum unit stress of about 10,700 pounds per square inch and a total force of about 1,540,000 pounds. These theoretical unit stresses and forces are much higher than the probable actual values because of the assumptions on which Equation (16) is based. In particular, no allowance has been made for the cushion block and it has been assumed that the pile point is fixed on a rigid support. Also, no account has been taken of stress reduction due to propagation losses and friction on the sides of the pile.

If similar calculations are made with Equation (17), the results are a maximum unit stress of 2,700 pounds per square inch with a total force of 389,000 pounds for the timber pile and a maximum unit stress of 3,570 pounds per square inch with a total force of 514,000 pounds in the concrete pile. The effect of including the cushion block is immediately apparent. However, even these stresses are higher than the probable actual values because of the assumption about the rigid point resistance and because of the neglect of side friction and propagation losses.

Nevertheless, these calculations demonstrate several important facts about the subsequent carrying capacities of these two piles. Even

though the calculations do not give the actual stresses that would occur in the piles, the numerical results are satisfactory for comparative purposes. The comparisons will be made on the basis of the stresses obtained with Equation (17) because these include the effect of the cushion block and are therefore closer to the actual stresses. When the hammer strikes the concrete pile, a stress of 3,570 pounds per square inch will be exerted on the dense sand and gravel under the pile point. When the hammer strikes the wood pile the stress will be 2,700 pounds per square inch. The penetrations of these two piles under a single hammer blow will be proportional to the stresses and the concrete pile will penetrate farther per blow than the wood pile. Let it be assumed that the two piles are required to be driven to the same final resistance (equal penetrations per blow). When this has been accomplished, the concrete pile will have penetrated somewhat deeper into the dense sand and gravel than the wood pile. In other words, for the same given downward movement on both piles, the concrete pile will have to penetrate the dense sand and gravel until its point encounters a total resistance of 514,000 pounds, whereas the wood pile will only penetrate until its point encounters a resistance of 389,000 pounds. The subsequent carrying capacities of these piles will be proportional to these total forces. That is, for equal downward movements under load, the ratio of the carrying capacities of these two piles would be as 514,000 is to 389,000. The concrete pile would carry 32% more load than the wood pile at equal settlements.

The fact that the theory of longitudinal impact leads to this conclusion is of considerable practical importance. The concrete pile in Fig. 3 weighs 3.75 times as much as the wood pile. The weights of the piles have been included in the analysis as can be seen from Equations (16) and (17). However, in spite of the greater inertia of the concrete pile, the calculations show that it will carry more load than the wood pile if both are driven to the same penetration per blow. On the other hand, if the indicated carrying capacities of these two piles, driven to the same final resistance, were to be calculated with a pile driving formula like Equation (10), the answer would be that the light wood pile would have a much higher indicated carrying capacity than the heavier concrete pile. Such a result is inevitable with a formula that considers only the inertia of the pile and none of its other properties. Nevertheless, there can be no question about

the validity of applying the theory of longitudinal impact to pile driving and the fact that this theory leads to conclusions that are diametrically opposed to Equation (10) simply shows that Equation (10) is based on a misconception of the real nature of the pile driving problem.

From this comparison it seems reasonably evident that the stress transmission characteristics of a pile are often far more important than its inertia. In many cases, the principal purpose of a pile foundation is to get the piles down through deep beds of soft soil until their points are embedded in a dense underlying stratum. If two piles of the same shape and size but of different weights and rigidities are driven for this purpose into the same soil with the same hammer to the same final resistance, the heavier more rigid pile will have a better chance of having satisfactory penetration in the hard stratum than the lighter more flexible pile. This is due to the fact that the heavy pile can transmit the driving stresses down to its point better than the light pile can. There is field evidence available to show that this actually happens and that the heavier more rigid pile will support a greater static load. It is conceivable that when pile driving is done according to a formula such as Equation (10), the required resistance for the design load might be developed by a very light pile even before the pile point reached the hard stratum. This would be due to the inability of the light pile to transmit large driving forces to its point. Under such conditions, Equation (10) could easily lead to an unsatisfactory foundation.

As far as the general application of the theory of longitudinal impact to pile driving is concerned, the investigations carried out by the British Building Research Board during the past few years represent the first effort that has been made to apply this theory to actual pile driving. The tests made in England were all made with precast concrete piles. Before the British engineers could calculate stresses from the theoretical equations, they had to determine by experiment the properties of their cushion blocks and they had to measure the permanent penetrations and the temporary elastic compressions of the piles as they were driven. A great deal of further experimentation would have to be done before the theory could be applied to all kinds of piles driven under various conditions. This experimental work would have to include the determination of the behavior under impact of the various types of cushion blocks used in actual pile driving.

It would also be necessary to measure, in the field, the temporary elastic compression as well as the permanent penetration.

In 1927, the Corps of Civil Engineers of the United States Navy made some experiments of this kind and a few of their results are shown in Fig. 4. In the upper diagram, the pile was a precast

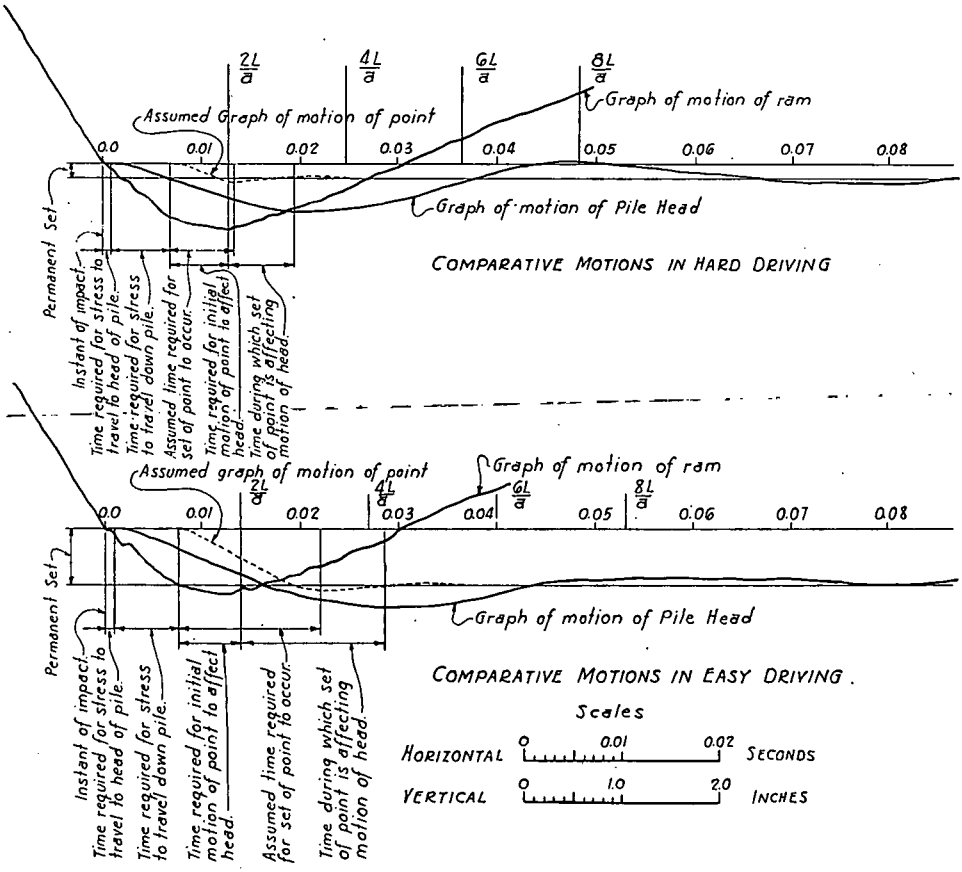

FIG. 4.

concrete pile 20 inches square and 53 feet long. In the lower diagram, the pile was of the same cross section and 65 feet long. In both diagrams the hammer was a No. 0 Vulcan single acting steam hammer with a ram weighing 7,500 pounds and operating on a 36 inch stroke at about 55 blows per minute. In both cases, the blow was cushioned

by about 19 inches of Douglas fir flat grain. The records were scribed on a rotating cylinder by means of pencils fastened to the pile head and to the hammer ram. The driving was being done with a floating pile driver and the measuring apparatus was mounted on a group of wood piles driven near the test piles to support the loading platform.

Both diagrams show the hammer ram approaching the pile head at a velocity of about 14 feet per second. Immediately after impact, the hammer ram continues to move downward with considerable velocity while the pile head begins to move much more slowly. The ram maintains a positive pressure on the pile head from the instant of impact until the time when the two curves cross. This time interval is the duration of impact and the vertical ordinates between the curves in this interval represent the temporary elastic compression of the cushion. When impact ceases, the ram proceeds on its up-stroke and the pile head follows a path that resembles a damped sine wave. In the case of hard driving, the pile head rebounds to an elevation slightly above its original elevation and then, after a few vibrations of small amplitude, comes to rest at an elevation lower than its original elevation by the amount of the permanent penetration.

These graphs show how the restraining action of the surrounding soil affects the motion of the pile. They also show several things that are of considerable interest from the point of view of the theory of longitudinal impact. The Navy engineers have shown on the diagrams the time required for the first longitudinal stress wave to reach the pile point and return to the head. The length of this time interval is $2L/a$ and the writer has marked off several additional intervals in multiples of $2L/a$. The upper diagram corresponds approximately to the case of a rod struck at one end and fixed at the other. For this case, the theory of longitudinal impact says that impact will cease in the interval between $2L/a$ and $4L/a$ whenever the ratio of the weight of the striking body to that of the rod is less than 1.73. In this experiment the weight ratio was about 0.3 and the hammer left the pile head between $2L/a$ and $4L/a$ as predicted by the theory. The lower diagram corresponds approximately to the case of a rod struck at one end and free at the other. For this case, the theory says that separation will occur immediately after the time interval $2L/a$ regardless of the relative weights of the bodies. The diagram for easy driving shows that separation did occur immediately after $2L/a$. Both diagrams show the effect of the longitudinal stress

wave on the motion of the hammer ram. Just before the time $2L/a$ the ram was moving along a fairly smooth curve. At $2L/a$ the stress wave returned to the pile head and threw the hammer ram into a series of minute secondary vibrations which show up clearly on the graphs.

Although these experiments were not made for the purpose of applying the theory of longitudinal impact to pile driving, they demonstrate the fact that the piles behaved as long elastic rods. With sufficient numerical data of this kind available, engineers could investigate the dynamics of pile driving far more carefully than has been possible heretofore. The statement is frequently made that the static carrying capacity of a pile is sometimes much greater and sometimes much less than the carrying capacity indicated by a dynamic formula such as Equation (4), (10) and (12). While this statement is true as far as it goes, it is the writer's opinion that at least a part of the difficulty is due to the fact that these formulas are based on assumptions which are only remotely related to the phenomena of actual pile driving. The fundamental problem in the development of a satisfactory dynamic pile driving formula is to determine the probable relationship between the dynamic resistance and the subsequent static carrying capacity of the pile. This problem cannot be solved by mathematics and theoretical mechanics alone. Accurate field measurements of the behavior of the pile during driving are absoultely essential.

In summing up the data set forth in this paper, the writer believes that the following conclusions may be drawn:

(1) Equation (1) is defective because it lacks the coefficient, C, and the energy loss, Q, which appear in Equation (3).

(2) The customary methods of evaluating the energy losses are erroneous. The three usual energy loss deductions are:
   (a) Only the temporary elastic compression—Equation (4).
   (b) Only the Newtonian loss—Equations (10) and (11).
   (c) Both losses combined—Equation (12).

All of these are defective because:
   (a) The temporary elastic compression is taken from static theory.
   (b) The Newtonian theory of impact does not apply to plie driving.
   (c) Some losses are deducted twice when (a) and (b) are combined.

(3) The theory of longitudinal impact indicates that the stress transmission characteristics of the pile are often more important than its inertia. This theory is particularly applicable to cases where piles are driven through soft soils into hard soils for the purpose of developing most of the supporting power near the point. In such cases the subsequent carrying capacity of the pile will depend on its ability to transmit driving stresses to the pile point.

(4) The theory of longitudinal impact could be applied to piles not driven to point bearing provided that the necessary field measurements were made. The analysis would probably have to be based on energy methods.

(5) Equation (3) is a satisfactory expression for a dynamic pile driving formula based on energy methods provided that the terms on the right-hand side are evaluated by means of field measurements and not by calculations based on erroneous applications of theoretical mechanics.

The reader should keep in mind the fact that all of this discussion refers only to the dynamics of pile driving and a dynamic pile driving formula is primarily a yard-stick to help the engineer to get reasonably safe and uniform results over the entire job. However, it is the writer's opinion that all engineers engaged in pile driving operations should understand the fundamental aspects of the problem and particularly the underlying assumptions on which dynamic pile driving formulas are based. In spite of all of the effort that has been expended on the problem in the past, pile driving is not yet an exact science. At the present time, the installation of a satisfactory pile foundation is largely a matter of experience and good judgment combined with a careful soil investigation.

In conclusion, the writer wishes to express his sincere thanks to Professor Karl Terzaghi and to Professor Arthur Casagrande for their encouragement and for their helpful criticisms and suggestions about the preparation of this paper.

NOTATION

The symbols used in this paper have the following meaning:

W	denotes	weight of hammer or hammer ram.
h	"	height of fall of hammer or hammer ram.
R	"	average resistance to penetration under one blow.
s	"	permanent penetration under one blow.

R''	"	initial resistance to penetration under one blow.
R'	"	final resistance to penetration under one blow.
C	"	a numerical coefficient.
Q	"	energy loss.
L	"	length of pile.
A	"	cross sectional area of pile.
E	"	Young's Modulus of pile material.
n	"	Newton's coefficient of restitution.
g	"	acceleration of gravity.
M	"	mass of hammer.
m	"	mass of pile.
V	"	velocity of hammer at bottom of stroke.
v	"	velocity of pile just before impact.
L'	"	length of driving cap.
A'	"	cross sectional area of driving cap.
E'	"	Young's Modulus of driving cap material.
K	"	energy loss due to elastic compression of soil.
a	"	velocity of stress in pile.
w	"	unit weight of pile material.
t	"	time.
u	"	longitudinal displacement of element of pile.
f	"	arbitrary function in general solution of wave equation.
F	"	arbitrary function in general solution of wave equation.
e	"	base of Naperian logarithms.
p	"	unit stress in pile cross section.
T	"	stiffness constant for cushion block.

CONCRETE DESIGN FOR LARGE PRECAST CONCRETE PIPE

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers held on November 8, 1939.)

By Preston M. Putnam*

The Metropolitan District Water Supply Commission is now constructing an eighteen mile pressure aqueduct from the terminal chamber of the Wachusett Aqueduct at Southborough to the Charles River at Weston. This aqueduct consists of 1.8 miles of 12′-6″ precast concrete pipe, 3.0 miles of rock tunnel, 12.8 miles of 11′-6″ precast concrete pipe and 0.5 of a mile of 7′-0″ precast concrete pipe for a by-pass to the Weston Reservoir.

All the precast concrete pipe for this aqueduct is manufactured by the Lock Joint Pipe Company of Ampere, New Jersey, at their plants at Natick and Southborough.

The total amount of concrete involved in the manufacture of this pipe is over 100,000 cubic yards, all placed in walls varying from 8″ to 12″ thick, 12′ to 16′ high, which contain a relatively large amount of reinforcing. The design of this pipe required that the concrete be dense, durable, and of a high compressive strength.

Aggregate

To obtain this, the first factor to be considered was an examination of the aggregate that the contractor proposed to use. This aggregate was to come from a local deposit of gravel in Framingham. This pit had not been opened up previously and only a few shallow cuts had been made in it. Early explorations showed a rather large percentage of soft, disintegrated stone in these cuts, so deeper test holes were dug and additional samples of stone obtained. These samples were given the Deval and the Los Angeles Tests for wear and were found satisfactory. Both these tests agitate stone of one size with steel for a period of time. The percentage of stone that is broken down to a smaller size by this abrasive action is then obtained. Briquettes

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were also made mixing varying percentages of this soft rock, ground fine, and cement. There was not any substance in this soft rock that affected the strength of the cement. Later, while the plant was operating in this pit, no trace of this soft rock was found in the coarse aggregate. It is probable that as this softer stone went through the crusher, it was so broken down that it went into the sand or washed out.

The maximum size of the coarse aggregate, governed by the thin walls and heavy reinforcing, was limited to 1". When the plant had manufactured a sufficient amount of material to obtain representative samples, a series of tests were made of them.

The fine aggregate was screened by Standard Tyler sieves to see if it was well graded, and that all passed a ¼ inch mesh screen, and not more than 15 percent was retained on a No. 4 sieve. The Maximum and Minimum Grading Curves of the Massachusetts Department of Public Works were used to limit the grading curve of the aggregate. The fine aggregate was also tested by the standard color test to see if it was free of injurious amounts of vegetable and organic matter. Standard briquette tests were made comparing the sand with Ottawa sand. The results were very favorable.

The coarse aggregate, which consisted of screened gravel and crushed stone mixed, was tested to see if it was well graded, that its maximum size was 1 inch, and that none passed a ¼ inch mesh screen. It was also inspected for soft, thin and laminated pieces and that it was free of loam, clay and other improper substances.

Cement

The specifications for the cement used on this work are those adopted by the Board of Water Supply, City of New York, as developed by the late Thaddeus Merriman. It is believed that the cement which fulfills these specifications is one of uniform quality and insures a better grade of hard, strong and durable concrete.

This cement is tested and inspected by an independent laboratory to determine whether or not it meets the requirements of the specifications. The inspection is made at the mill during the burning and grinding of the clinker and the tests are both physical and chemical.

The physical tests determine the percentage of water, the setting time by the Gilmore needle, soundness, fineness, one day and seven

day tensile strengths of neat cement briquettes, seven day and twenty-eight day tensile strength of one to three mortar briquettes and the sodium sulphate characteristics.

The chemical tests determine the percentages of silica (SiO_2), ferric oxide (Fe_2O_3), alumina (Al_2O_3) lime (CaO), magnesia (MgO), and sulphuric anhydride (SO_3) and also determine the loss on ignition, insoluble residue, molecular ratio, lime silica ratio, alumina iron ratio, alkalinity, and free alkali. The chemical tests also include special tests to determine the sugar solubility of the cement.

A more complete discussion of this cement, the allowable limits of the above tests under these specifications, together with the reasons for them may be found in a paper presented by the late Thaddeus Merriman at a joint meeting of the Boston Society of Civil Engineers and the Designers Section on October 19, 1938 and published in the JOURNAL of the Boston Society of Civil Engineers in January 1939, Volume XXVI, Section 1.

PROPORTIONS

A series of sieve tests were also made on the proposed aggregates to determine the average fineness modulus for both the sand and the stone. From these results, in order to determine the greatest density, the maximum desirable fineness modulus of a mixture of the two was computed. This gave a ratio of the fine, to the total aggregate of about 41%. In addition, varying proportions of sand and stone were combined to determine which combination gave the maximum dry rodded weight. These results checked the previous computations that the mixed aggregate should consist of about 41% sand and 59% stone to give the maximum density. It is probable that other combinations of this aggregate, increasing the proportion of coarse aggregate, would be more economical but not so dense. In work of this nature dense concrete is of primary importance.

The next step was to determine what proportion of cement must be used to give the desired strength of concrete. This strength at the start of the work was set at 5,000 pounds per square inch or over, with a water-cement ratio of about five gallons per sack of cement. Numerous trial batches were made up before the start of the work, varying the proportions of the aggregate to a fixed proportion of cement and water to determine a workable mix. The results of the desired proportions of the aggregate as obtained from use of the fine-

ness modulus and dry rodded weights was used as a guide only. The first mix used was one that had a water-cement ratio of 4.9 with a 4″ slump and a cement factor of 8 sacks of cement per cubic yard of concrete. Tests for compressive strength on this concrete averaged about 6,000 pounds per square inch.

As the work progressed, the additional knowledge of the past mixes, added information concerning the aggregate, better control of the mix, the more efficient placing by the contractor's men and equipment, enabled us to reduce the mix to one that had a water-cement ratio of about 5.8 with a 3½ inch slump and a cement factor of 6.6 sacks of cement per cubic yard of concrete. This leaner mix did not materially affect the quality of the concrete. The average compressive strength was about 5,800 pounds per square inch, the density dropped very slightly, and the appearance and finish remained unchanged.

The manufacturers of concrete pipe have been in the practice of using a mix similar to our original one and the contractor felt some concern in reducing the amount of cement. However, most of the pipe made previously by them was of a smaller diameter, with thinner walls and so our engineers felt justified in making this reduction on pipe of this size.

Mixing

The concrete is mixed with two, two yard, mechanical mixers of a rotating drum type, set side by side. Cement is received in bulk cars of the bottom dump type. The cement is unloaded by a mechanical screw to a bucket conveyor and thence to either a cement hopper above the mixer or to a storage tank which has a capacity of a day's pour.

The aggregates are trucked to the work and dumped into a hopper under the roadway. From here they are conveyed by a bucket elevator to bins above the mixers. The aggregates and cement are proportioned by weight in an aggremeter which feeds either mixer.

The water is measured by an electric Spangler Metering Valve. This valve can be set for any desired amount of water and shuts off after that amount has passed. It also records the total amount of water that passes through it so that the total amount of water used in a day's pour can be checked.

The inspector on the job has absolute control of the amounts of

aggregate that are used in the mix. He is furnished with the dry weights of the aggregates and cement and the quantity of water to use for the desired mix. Several times a day, the Laboratory furnishes him with the moisture content in the aggregate. Prepared tables give him the correct wet weights of the aggregates and the amount of water to be added to adjust for the water in the sand and stone.

Of course, the sudden changes in the water content of the aggregates make it necessary for the inspector to change the various quantities of the mix before the Laboratory has time to give him the data. Once the desired mix has been determined in the Laboratory, visual inspection by an experienced man is still very important for good control.

Placing

The concrete is mixed at least two minutes and then is poured into two bottom dump buckets with a capacity of two yards each, the capacity of each mixer. These buckets are on flat cars which are hauled by a dinkey locomotive to the casting site. There the buckets are lifted by a locomotive crane and the concrete deposited into a rotating hopper. This hopper is centered on the axis of the circular forms. It has a chute extending to the top of the forms and is rotated by an electric motor. A gate at the outlet of the chute makes it possible to control the rate that the concrete is deposited. The concrete is compacted by means of three external electric vibrators placed on the outer forms.

At the start of the work, these rotating hoppers were not used and the concrete was deposited at a much faster rate than it is now. This did not give the vibrators sufficient time to expel the trapped air and the concrete showed quite a number of pock marks on the surface of the pipe. It was found that a slower rate of depositing the concrete materially helped to overcome this condition and the rotating hoppers were designed. Some experimenting was done with internal vibrators. It was necessary for these to penetrate 16 feet of comparatively stiff concrete. They were very unwieldy to handle on top of the forms and the men could not control them in the concrete sufficiently to produce the desired results and so they were abandoned. The concrete in the upper half of the pipe had less vibration than that in the lower half and showed more air pockets. This upper half is now spaded on the inside to give a smoother finish.

Casting Plant

The casting plant consists of 72 base ring forms set in a line. These are heavy steel castings placed in concrete and form the bell end of the pipe. They have grooves in which the reinforced steel cylinders set, as well as do the outer and inner forms. These forms are steel with one gate. The inner form contracts and the outer one expands when removing and each is lifted out as a unit. A standard guage railroad track runs parallel to the casting site on which locomotive cranes operate, placing forms, reinforcing and concrete.

After the concrete is placed, the top is covered and the concrete steamed at a temperature of 100 to 120 degrees. The forms are removed the day after pouring and the green pipe covered with canvas and steamed two additional days. On the third day, steel struts are placed inside the pipes to strengthen them during moving and the pipes are lifted off the base rings by a gantry crane which straddles them. This crane then turns the pipes on their side and they are rolled into the curing yard. There the pipes are cured an additional twelve days by keeping them moist inside and out. In the warmer weather this was done by a system of sprinklers installed on the top and inside the pipes. Now with colder weather, it is necessary to completely cover the pipes and bleed steam in the inside when the air temperature is below 45 degrees, creating a warm fog around the pipes. This required over 1000 pieces of canvas 18 feet square. At the end of this twelve day period, the concrete in the pipes is considered cured and the pipes are ready for delivery to the laying contractor.

Laboratory

At the beginning of the work, a concrete laboratory was established at the site. Here the moisture tests are made, aggregate is tested for grading and organic matter, and slump tests are made as well as concrete test cylinders. The laboratory is equipped with a compression machine and has a moist room for curing the test cylinders in warm moist air.

When the concreting started, a set of three concrete test cylinders were made for every 100 cubic yards of concrete poured, or five sets a day. Of these three cylinders, one was broken at seven days and two at twenty-eight days. Later the number of sets of cylinders were reduced to three a day and the seven day cylinders were omitted unless the mix was changed.

The concrete for the cylinders is taken from the chute of the rotating hopper as it is going into the forms. Care is exercised that this concrete is representative of that being placed and is not the tailings of a batch. The sample is taken to the laboratory and a slump test made of it. The test cylinders are then formed and allowed to set until the next day when they are placed in the moist room for curing. The standard A. S. T. M. methods are employed in making the slump tests and the cylinders. Steel molds are used in forming the test cylinders as a general thing, with cardboard molds used when extra cylinders are made. Both give good results but with the number of cylinders that must be made on a job of this size, the steel molds are more economical.

Two days before the cylinders are to be broken, they are removed from the moist room and capped with neat cement. After the caps have sufficiently set, they are covered with wet burlap until the cylinders are broken. The compression testing machine has a capacity of 200,000 pounds or about 7,000 pounds per square inch on a 6" diameter cylinder. It operates hydraulically and the pump is manually operated.

At the time a sample of concrete is taken, the added water at the mixer is noted and samples of the aggregate obtained. These are dried out to determine the moisture content and the water-cement ratio of the concrete is computed. The reports of the results of the tests on these concrete cylinders note the brand of cement used, the nominal mix, slump, water-cement ratio, location on the work, date cast and day broken, age, where cured, temperature cured, density, and compressive strength.

Several other interesting experiments were performed in the laboratory. One of these was to study the effect of water curing concrete. Twelve blocks were cast from the same sample of concrete. The blocks were divided into groups of three each and cured fourteen days. One group was cured outside without any water except scattered showers, the second group was cured by watering every half hour during the day only, the third group was cured in the moist room and the fourth inside the pipes. The last three groups, all of which were cured with water of varying amounts, gave about the same results, showing that daylight watering has nearly the same effect as continued water, except possibly on a hot, dry, summer's night.

There was, however, a marked contrast between the dry cured

group and the other three. The compressive strength of the dry group was nearly 50% less than the wet groups. The fractures of these blocks showed that it is possible to determine by visual inspection whether concrete is water cured or not. The fracture of the dry cured concrete showed practically no shear of the sand and stone, had poor bond, and loose uncombined particles of cement were present, giving the concrete a dusty, pasty color. The water cured concrete showed complete fracture of both sand and stone, good bond and there was no loose cement apparent, but rather the cement, sand and stone blended together as one homogeneous material. This left no doubt that in order to develop the strength of the cement in the concrete, the water must be prevented from escaping, as otherwise there will be loose cement present that is not doing any work.

Also it was desired to determine what effect the three day period of steaming had on the concrete in the pipes as compared to water curing. Standard cylinders from the same sample were cast and some placed in the pipes during steaming and others in the moist room. The results of these tests showed that the steamed concrete had a materially higher compressive strength in three days, but that its ultimate strength was not higher than the water cured.

This higher early compressive strength, obtained by steaming, is desirable by the contractor as the pipe can be moved earlier. However, care must be exercised that the temperature around the concrete is not raised over 120 degrees or the concrete will be dried out. It is doubtful if the steamed concrete is as durable as that which has been water cured. Blocks of both kinds of concrete were subjected to sand blasting and the mortar in the water cured blocks withstood this abrasive action much better. It is probable that when high early strength is desired, curing by hot water will produce better concrete than curing by steam.

One question that has been asked in the past is why the cement factor was not dropped below 6.6 sacks of cement per cubic yard, considering that the compressive strength was nearly 5,800 pounds per square inch, and if this strength was needed. From the standpoint of strength alone, this cement factor could have been lowered, but it is doubtful if it should have been from the standpoint of durability. Sand blast tests were made on blocks of concrete with cement factors varying from 5 to 8 sacks of cement per cubic yard. The concrete with higher cement factors showed greater resistance to the sand blast than did the lower, justifying the use of the greater amount of cement.

THE DEVELOPMENT OF THE DIGESTION TANK

By W. Allan Craig*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers held on November 1, 1939.)

The stabilization of organic matter by bacterial action is a natural and common phenomenon. In sanitary engineering, this stabilization is known as digestion. Digestion may be effected either by aerobic or anerobic bacteria. The bulk of the bacteria in a digestion tank is of the anerobic type although there is always some bacteria of the aerobic type present.

In order to obtain efficient digestion of sludge, it is necessary to provide favorable living conditions for the bacteria and other organisms active in the process.

1. Food must be provided at regular and rather short intervals.
2. Provision must be made to enable the bacteria to get rid of their waste products. This may be accomplished by some means of agitation or circulation.
3. An alkaline or near alkaline condition must be maintained as otherwise digestion will be replaced by fermentation.
4. A temperature of about 85° should be maintained.

Most of the above principles were recognized before separate sludge digestion tanks were built. In 1890, W. E. Adeney and W. Kaye Perry obtained the British patent No. 3312, which contains the following claim:

"The purification of acid or alkaline sewage and other waste liquors containing organic matters by rendering them neutral or slightly alkaline, adding an inorganic oxygen compound and maintaining them at a temperature of from 75° to 95° F., for a period of two hours or more, substantially as described."

Good digestion of sludge took place in the early settling tanks of the Fosses Mouras type, built in France from about 1860 to 1865.

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When the septic tanks, especially the Cameron tanks came into use, it was claimed that they digested practically all of the suspended solids; however, it was very soon discovered that such claims were entirely unfounded. The offensive organic material was digested, but a good part of the sludge was carried out with the effluent.

Engineers observing the processes taking place in septic tanks were thinking of ways and means to improve the operation of these tanks, and the most logical development was the separation of the functions of sedimentation and digestion. Clark, at the Lawrence experimental station of the Massachusetts State Board of Health, was the first to construct separate settling and digestion tanks, although he called the digestion tank the septic tank.

The Travis tank makes possible a partial separation of the settling and digestion compartment, but provision was made in these installations to divert part of the flow through the digestion compartment.

The Imhoff tank was an improvement on the Travis tank, although a separation of the settling and digestion compartments was not yet complete.

The separation of the settling tank from the digestion tank was complete in the experimental installation of a Kremer tank at Charlottenburg, Germany, and operated from 1907 to 1908. A gate separated the settling tank from the digestion tank.

Foerster also used a separate digestion tank in his installations in about 1910.

The installations mentioned above were all for small plants. The first really big installation, where separate digestion tanks were used, was in Birmingham, England, and was based on the experiments with the circulation of sludge by the late O'Shaughnessy from 1909 to 1911.

In tanks that were built at a later date, gas collection and means for heating were provided for. The first floating covers were made from concrete; later digestion tanks were rectangular, with the floating steel covers also rectangular. The supernatant liquor was collected in small open settling tanks where the solids in suspension were settled out. The clear liquor was pumped through a heater back into the digestion tanks for heating, and no heating coils were installed in these tanks.

The next progress in the design of tanks was the separation of the digestion process into two stages. This was first done by Dr.

Kusch in the so-called Kremer-Kusch tank. The supernatant liquor was discharged from the primary into the final digestor automatically over a weir and the sludge from the primary digestor was drawn into the final digestor through gates installed in the partition wall between the tanks.

Some of the first experiments made with separate digestion tanks in this country were not very satisfactory, principally because the sludge was dumped into a tank without any means for providing circulation or pH control, although Dr. Reichle and Dr. Thumm of the Landesanstalt published a paper describing all the conditions required for the successful design and operation of such tanks. A translation of this paper was published in the *Engineering News* and the *Engineering Record* in 1914.

A tank without any mechanical equipment was reconstructed from an old septic tank in 1924 at Merchantville, New Jersey, in which was made provision for sufficient movement of the sludge in the tank to insure good digestion. The original tank had only one baffle, and additional baffles were installed to provide a serpentine pass for the sludge, about 280 feet long. The digestion tank had a sludge capacity of 20,000 cu. ft. serving a connected population of 12,000. Good digestion was obtained, but after a number of years the baffles, which were held in place by wedges, turned over, and poor results followed. It was some time before the reason of the sudden failure of the tank to function properly was discovered. The tank was emptied, the baffles replaced, and the tank again placed in operation.

Later, due to increasing load, a new settling tank was added to the plant and one of the three existing settling tanks was used for a digestion tank in conjunction with the converted septic tank. Excellent results have been and are being continually obtained from these two tanks even though they are unheated.

The various digestion tanks at the Back River plant serving Baltimore, Maryland, are an illustration of the progress made in sludge digestion. Three rectangular tanks, each having a capacity of 203,000 cu. ft. were put in operation in 1912. These tanks were uncovered and no provision was made for heating or circulating the sludge. The supernatant liquor was pumped into the preliminary sedimentation tanks.

In 1915 sixteen round tanks, 38′-0″ in diameter by 24′-6″ deep

were built. From 1926 to 1933, a number of experiments were made using three of these tanks, two were provided with domes for collecting gas and another was equipped with a Pruess circulating pump.

Six additional rectangular tanks were built west of the circular tanks in 1924. These tanks have vertical baffles that provide a serpentine pass for the sludge. The sludge from the preliminary sedimentation tanks was removed about once a week, and pumped into one or more of the digestion tanks. Due to this method of operation (that is, the one tank receiving a large volume of sludge at infrequent intervals) digestion was very slow.

The latest type of tank at the Back River plant was designed by Mr. C. E. Keefer, who is in charge of the Division of Treatment Works of Baltimore. There were two of these tanks built in 1935, each 100 feet in diameter and with a side wall depth of 25 feet. Stirring apparatus is provided at the top and the bottom of each tank. An additional tank of the same size was completed this year. Gas is collected in a 200,000 cu. ft. waterless gas holder, and used for heating buildings and these three round digestion tanks. As much as 1,000,000 cu. ft. of gas is collected daily.

Fresh sludge is pumped to these three digestion tanks, after a digestion period of from two to three weeks. The partially digested sludge is pumped from these tanks, into the old unheated rectangular digestion tanks where it is permitted to settle and cool. The supernatant liquor from all of the digestion tanks is pumped into the four old primary settling tanks, the effluent from which is added to the effluent from the new mechanically-cleaned preliminary sedimentation tanks and treated on 30 acres of trickling filters.

In many Imhoff tanks a heavy scum blanket formed in the gas vents so that mechanical agitators were used to submerge the scum. When separate digestion tanks came in general use, mechanical equipment was installed in many of them, having a stirring device on the top for paddling down the scum and a collecting device at the bottom to push the digested sludge to the point of discharge. The advisability of installing such devices has been a subject of much controversy.

Digested sludge will flow readily in a digestion tank without mechanical aid. At the Merchantville tank with a total flow length of 280 ft. and a flat bottom, the sludge moves progressively forward and is readily withdrawn with a moisture content of 91% or less.

Gas bubbles evolved in the process of digestion will provide suffi-

cient agitation in the great majority of cases. The circulation in heated tanks due to the difference in temperature in the different strata of the tank is often more violent than desirable, and in most cases no additional means of agitation is required.

As a rule the scum blanket in covered tanks is about 12″ thick and does not interfere with digestion and gas collection. It is claimed for floating covers that the scum is kept submerged and will sink to the bottom. Many digestion tanks with and without stirring apparatus were built and with proper operation and normal sewage all gave good results. The arguments between the advocates of the different systems continued and in order to obtain first-hand information the Sanitary District of Chicago built two large experimental units side by side. One tank provided with a stirring mechanism was 40 ft. in diameter x 35 ft. deep with a sludge capacity of 275,000 gallons. The other tank without a stirring mechanism was 50 ft. in diameter x 35 ft. deep, and had sludge capacity of 325,000 gallons. The average results taken from the Chief Engineer's Budget Report were as follows:

Type of Digestor	Temp. Fahr.	Days Diges-tion	Cu. Ft. Gas Per Lb. Volatile Matter Added	B.T.U. Gas per Cu. Ft.	Raw Sludge % Mois-ture	Sludge Drawn % Mois-ture	% Diges-tion of Volatile Matter	Super-natant Liquor % Solids
Plain	94.4	28.4	8.9	738	96.2	96.2	42.1	0.35
Mechanical	94.8	32.0	8.9	748	96.1	95.5	41.9	1.37

A small amount of scum not over 1 ft. in depth accumulated in both tanks.

The high concentration of suspended solids in the supernatant liquor from the mechanical digestor was due, in part, to the fact that the sludge blanket was carried too near the supernatant draw-off port.

Occasionally where digestion tanks are handling sludge from industrial wastes such as wool washeries, etc. a heavy scum blanket forms of such a character that equipment must be installed that will break it up and submerge it, either by violent agitation and circulation as provided by the Pruess Pump, or by positive displacement and submergence as accomplished by the Link-Belt Scum Breaker.

The scum breaker consists essentially of a double strand of endless chain guided to travel along the underside of the roof of the tank and running over two sets of sprocket wheels. Attached to these

chains at approximately 5 ft. intervals are paddles which are designed to break up the scum. If the scum does not settle it is positively conveyed to one side of the tank, where it accumulates, and as the paddles travel around the head shaft the scum is necessarily carried along with the paddles and is, therefore, positively submerged.

In either round or rectangular tanks the paddles cut a swath through the scum blanket and the scum not originally in the path of the paddles floats into their line of action and is in turn paddled down and submerged.

The scum breaking mechanism is adaptable to both round and rectangular digestion tanks. Rectangular tanks lend themselves to compact layouts by the placing of digestion tanks in pairs or alongside of settling tank, and are frequently most economical, due to straight framing and form work and the use of common walls for adjacent structure. When round digestion tanks are indicated as being the desired shape then the use of Link-Belt scum breaking equipment permits an economical roof construction, as the roof may be supported by columns straddling the mechanism and the roof does not have to be designed to span the whole diameter of the tank.

The breaking up of the scum blanket is the most important function of the mechanism, and consequently normal design would be with the return strand hanging in a catenary or loop between the head and foot shafts. There are times, however, when it would seem advisable because of the possibility of sand or grit accumulating in a digestion tank to provide for the mechanical moving of such deposits along with the digested sludge to the discharge point. When such condition is indicated two lower turn shafts are added to the scum breaking mechanism and the return strand travels parallel to and along the floor towards the point of sludge withdrawal.

FLOOD CONTROL AND ITS RELATION TO PROBLEMS OF STREAM POLLUTION

By Donald F. Horton, Member*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers held on October 4, 1939.)

This subject is of particular importance in this part of the country because it has been said frequently that the principal water problems of New England are flood control and pollution abatement. The suggestion has been made with increasing frequency of late that flood control may be effected through multiple-purpose reservoirs in which flood waters may be collected and released later in relatively dry periods. Such operation, resulting in increased low-water flows, would yield benefits for many purposes, among them abatement of stream pollution through dilution.

Relation between Flood Control and Stream Pollution

Flood control is directly related to problems of stream pollution only in the sense that if adequate flood control measures prevent great floods, stream pollution caused by such floods is eliminated. Those who have not had personal experience with stream pollution and other sanitation problems which require emergency measures during floods probably recall Mr. E. Sherman Chase's description of his "Experiences at Louisville, Kentucky, during the Ohio River Flood of 1937", presented at a meeting of this Section on March 3, 1937. Mr. Chase's paper clearly illustrated the benefits which would result through avoidance of the repetition of such floods.

A type of stream pollution which does not commonly occur in New England except during floods results from silt. Unlike most other wastes, silt is the basic resource of the industry (agriculture) which produces it, and once in stream channels it rarely can be reclaimed. It is in part the product of agricultural practices and other soil disturbances and in part the product of natural processes of erosion, the relative importance of each source varying greatly among

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Statements and opinions are to be understood as individual expressions of the author and not those of the Engineer Department.

basins. It causes injury to aquatic life, impairs the quality of water for domestic, agricultural, and industrial use, impedes navigation by shoaling channels and harbors, and reduces the capacity of reservoirs. The damages to farm economy which it involves are so much greater, however, that its control is primarily a matter of concern to agriculture. The Department of Agriculture is authorized to undertake investigations of measures for run-off and water-flow retardation and soil erosion prevention.

Flood control is more frequently associated with problems of stream pollution in suggestions envisioning the abatement of stream pollution by dilution effected by increasing the low-water flow of polluted streams through operation of reservoirs. Reservoirs constructed and operated for flood control only do not increase low-water flows, because they are kept empty before floods and emptied as soon as practicable after floods. The best known examples of this type of reservoir are those which were built under the direction of Dr. Arthur Morgan for the Miami Conservancy District in Ohio. In New England we have several purely flood control reservoirs in the Winooski River Basin in Vermont.

Multiple-purpose reservoirs provide a means by which flood waters may be stored for later release to increase low-water flows. Collateral benefits from multiple-purpose reservoirs may include those from flood control, the development of hydroelectric power, improved recreational facilities, the storage of water for domestic, industrial, or irrigation use, or for pollution abatement by dilution of sewage and industrial wastes and the regulation of stream flow in the interests of navigation. Reservoirs of this type constructed, or under construction, by the Corps of Engineers, are Fort Peck, Conchas, Tygart, and Denison.

Jurisdiction of Engineer Department

In order that my viewpoint may be better understood, it will be desirable to consider the functions of the Engineer Department (as the organization charged with the civil duties of the Corps of Engineers, U. S. Army is called) with respect to flood control and stream pollution and the procedure followed in water-way investigations.

The present Federal flood control policy was established by the Flood Control Act of June 22, 1936, and amended and supplemented

by the Flood Control Acts of August 28, 1937, June 28, 1938, and August 11, 1939. These acts provide that Federal investigations and improvements of rivers and other waterways for flood control and allied purposes shall be under the jurisdiction of, and shall be prosecuted by, the War Department under the direction of the Secretary of War and supervision of the Chief of Engineers. The Act of 1938 authorizes and directs the Secretary of War to acquire in the name of the United States all lands, easements, and rights-of-way necessary for any dam and reservoir project or channel improvement or channel rectification project for flood control.

The Federal Government now exercises direct regulatory control over two types of water pollution. First, under the Oil Pollution Act of 1924 the discharge of oil from vessels into tidal navigable waters is prohibited. Second, under earlier River and Harbor Acts, the deposit of "any refuse matter of any kind or description whatever other than that flowing from streets and sewers and passing therefrom in a liquid state, into any navigable water of the United States or into any tributary of any water from which the same shall float or be washed into such navigable water" is prohibited. The Secretary of War is authorized to enforce both of these regulations, which are designed to prevent pollution resulting from discharge of oil from vessels, and to maintain the navigable channels in coastal and inland waterways.

An indirect control of water pollution is administered by the U. S. Public Health Service under its authority to determine the quality or safety of drinking and culinary water used on interstate carriers. Likewise, under a voluntary agreement with the several states, the Service is in a position to regulate indirectly conditions in shellfishing areas by giving or withholding its endorsements to the certifications issued by the producing states to interstate shippers. With these few exceptions, the regulation of water pollution now is exercised almost wholly by state governments.

Engineer Department Procedure in Waterway Investigations

The procedure of the Engineer Department, as directed by Congress, in the study and prosecution of projects connected with waterways provides a splendid illustration of democratic methods. Such projects are not initiated by the Government, but by the people of the locality affected, who, through their representatives in Congress,

bring the need for Federal improvement of a waterway before the Congress. In the event the evidence of need is favorably received, a study and report by the Engineer Department is directed by an act of Congress. Upon receipt of this directive from Congress, the Chief of Engineers directs the District Engineer of the appropriate district to undertake the study and report.

The District Engineer usually begins his investigation with public hearings, at which all parties concerned are requested to present their views. The information so obtained is combined with all other information bearing upon the question at issue, of whatever nature and from whatever source, including all necessary field investigations, computations and estimates, and the whole then subjected to a final engineering and economic study which results in the District Engineer's finding and recommendation. This report is submitted to the Division Engineer, and after review by him, to the Board of Engineers for Rivers and Harbors in Washington. Upon action by the Board, the report then is transmitted by the Chief of Engineers to Congress through the Secretary of War. The project is subsequently acted upon by Congress, and favorable action results in its inclusion in an act authorizing the construction of the project. Subsequently, construction is carried on from appropriations made from time to time to the War Department for the prosecution of the most urgent of the projects under its jurisdiction which were previously authorized.

The action described above is not necessarily final. Whenever any interests affected request and obtain further Congressional acts or resolutions of a proper committee directing a review of previous reports upon the project, such a review is undertaken by the Department in the same manner as an original examination or survey.

The studies undertaken by the Engineer Department upon a waterway improvement proposal include all phases of a complete engineering analysis of the project. They comprise not only design, but the economics of justification, and they include consideration of all incidental effects as well as the direct purpose for which the improvement is intended, as outlined by Congress in the act directing the report.

Multiple-Purpose Reservoirs

Storage capacity provided in flood control reservoirs for purposes other than flood control may be of some effect in reducing floods,

but such effect is unreliable in that this storage space may be full when a flood occurs. Since the effect of such storage is not positive and reliable, the same flood protective measures are required whether such storage is available or not. It is therefore normally considered improper to charge to flood control any of the costs of providing storage solely for purposes other than flood control.

In the case of a reservoir where storage is to be used jointly for flood control and for other purposes, it is essential that the storage capacity required for flood control be available when needed. Such a condition can exist only where floods always occur during a definite season of the year and where holding the required flood storage capacity empty during the flood season will not affect its availability for other purposes during the season when floods do not occur.

In studying projects for multiple-purpose reservoirs from which certain benefits are found to be purely local, it is essential to determine the ability and willingness of local interests to participate in the costs of the improvement.

Sanitation Benefits from Multiple-Purpose Reservoirs

It has been generally considered by proponents of multiple-purpose reservoirs and engineers directly concerned with sanitation and pollution abatement that large benefits may be obtained through provision of storage for the purpose of increasing the flow of the rivers during the dry season, at which time pollution is most critical, and by this increase to alleviate an otherwise objectionable condition. On the face of it, it appears that the construction of multiple-purpose reservoirs containing storage capacity assigned to the job of increasing low-water flow may be of considerable assistance in the solution of the pollution problem. However, the benefit to be derived from increased dilution of polluted waters is dependent upon the use of the river. Some streams considered of an industrial nature may be grossly polluted and may need very expensive treatment in order to completely purify them to the extent necessary for recreational or water-supply uses, while it might be possible, through partial treatment or dilution, or a combination of both, to clean up the stream sufficiently to fulfill public health and aesthetic requirements.

The National Resources Committee Special Advisory Committee on Water Pollution, in its 1938 report, "Water Pollution in the United States", has outlined the basic considerations as follows:

"The key problem in planning for pollution abatement is to find the standards in each section of a stream which express the best balance between the stream's use for receiving and assimilating sewage and other waste and its use for other purposes, aesthetic and economic.

"Every drainage basin has a complex of human uses of land and water. Each basin is a patchwork of economic activities, some of which show little regard for physical conditions and others of which represent delicate adjustments to advantages and disadvantages of the natural environment. In a given basin, one tributary may traverse a wilderness used chiefly for recreation, and one may be the axis of an urban area devoted in large measure to commerce. Present and future needs for water and for the use of the stream as a means of waste disposal may require different standards of quality for each tributary area. In the wilderness area the sole issue may be maintenance of the fishing streams in their natural conditions, whereas in the urban area the problem may be to determine the maximum treatment which can be installed without impairing the economic life of the area."

Practical application of this principle has been made by the Interstate Commission of the Delaware Basin. It divided the basin into four zones. The principal uses of the waters of the Delaware River are expected to be as follows: (1) Zone 1—for water supply after such treatment or purification as may be necessary, and for recreation, bathing, maintenance of fish and aquatic life, agriculture, and for other related purposes; (2) Zone 2—for water supply, after treatment or purification, and for recreation, navigation, maintenance of fish and aquatic life, agricultural, industrial, and other purposes; (3) Zone 3—for navigation, industrial water supply, and other purposes; (4) Zone 4—for navigation, industrial water supplies, commercial fishing, shellfish culture, recreation, and other purposes. On the basis of expected use in each zone as outlined, minimum requirements for the waters of each zone were drawn up and approved by representatives of the interested states.

A comparable set of standards was established under the New York, New Jersey, Connecticut Tri-State Compact approved in 1936, under which the tidal waters of the New York metropolitan area were classified as follows: (1) Class "A", in which the designated water areas are expected to be used primarily for recreational purposes, shellfish culture, or the development of fish life; (2) Class "B", in

which the designated water areas are not expected to be used primarily for recreational purposes, shellfish culture, or the development of fish life. Standards of content of solids, coliform organisms, and oxygen demands for the effluent were set up for treatment of sewage in Class "A" waters, and standards of content of solids and oxygen demand for Class "B" waters.

Because of the great variation in water use and standards, not only within each river basin but also between basins, it is evident that generalizations as to sanitation benefits from multiple-purpose reservoirs are of little value. For this reason it is more profitable to confine discussion of this point to specific drainage basins or portions of drainage basins.

Tygart Reservoir

The Tygart Reservoir, recently completed by the U. S. Engineer Office at Pittsburgh, affords an illustration of a multiple-purpose reservoir from which actual benefits have been secured in abatement of stream pollution. The Tygart River joins the West Fork River to form the Monongahela, which is an acid stream except for a few days during high-water periods, when it is slightly alkaline. Pollution consists of sewage, industrial acid wastes, and large amounts of acid mine drainage, a condition considerably different from that encountered in New England. The Tygart Reservoir was designed to have a storage capacity of about 300,000 acre-feet, of which 100,000 acre-feet would be available for storage to augment the low-water flow of the Monongahela for navigation and other purposes.

The water stored during high-water periods, being alkaline, has a neutralizing effect on the acidity prevalent during the greater part of the year. Studies indicated that water stored in the reservoir would have an average alkalinity of about 6 parts per million and an average hardness of about 50 parts per million. During the period in 1938 when the reservoir storage was used to supplement the natural flow, the outflow averaged approximately 315 c.f.s., or about 25% of the total flow of the lower Monongahela River at McKeesport. The release of this amount of water having the characteristics described above, into a river constantly acid and with a hardness of 150 to 250 parts per million during the summer months is a material aid to navigation, and, at the same time, has a marked influence upon the quality of the water. Estimates of savings for the

Monongahela River as a whole, based upon the reduction of hardness of 23 parts per million and the reduction of acidity of about 4 parts per million totalled nearly $50,000 for the year 1938.

Merrimack River Basin

The Boston District of the Engineer Department is interested in the Merrimack River Basin. The comprehensive plan for flood control and other purposes, as approved by the Chief of Engineers pursuant to a preliminary examination and survey (H. Doc. No. 689, 75th Congress, 3d Session), was authorized by the Flood Control Act of June 28, 1938. The total estimated cost of this plan is $21,000,000. Construction under this project has been initiated at the Franklin Falls Dam, near the city of Franklin, N. H. Final selection of the other reservoirs which will complete the authorized project is now in progress.

In connection with a survey of the river for navigation, flood control, and water power, required by the River and Harbor Act approved June 20, 1938, the following considerations are being investigated:

(1) Could not multiple-purpose reservoirs for flood control and other purposes be so designed and operated as to increase the low-water flow of the stream?

(2) If so, could not the degree of required sewage and waste treatment be reduced at various places and part of the multiple-purpose reservoir be charged to pollution abatement, since the increased low-water flow would provide dilution requiring less expensive sewage or waste treatment plants?

Because of the density of population and inadequate disposal facilities for sewage and industrial wastes, gross pollution of the Merrimack River results between the New Hampshire-Massachusetts boundary and the sea. The river, in its course throughout the state of New Hampshire, is considerably polluted by domestic sewage and industrial wastes, but analyses of river samples collected over a number of years under the direction of the Massachusetts Department of Public Health show that the river as it enters Massachusetts is not objectionable for recreational uses, although it may at times contain large number of bacteria which make it an unsafe source from which to take drinking water, even after treatment. Through its course in

Massachusetts the river receives extremely heavy pollution of domestic sewage and industrial wastes.

The need for abatement of the pollution of the Merrimack River in Massachusetts is locally recognized. The sources of pollution are recognized and measurable. The effects of pollution are understood in large part. Techniques for abating pollution are developed to a point where effects and remedies are understood to a degree which makes it possible to estimate the cost of bringing the river to any standards of purity which may be set with proper regard for the most economic use of the water and the adjacent land. Most persons familiar with the problem can agree on these basic facts and technical judgments, although there are differences of opinion as to the practicability of some treatment processes and as to the economic and social need for complete treatment. Major disagreement arises over the means which should be taken to insure the speediest practicable abatement.

The proposal for construction of a trunk sewer in the Merrimack Valley has received considerable support and has been discussed for a number of years. The Commonwealth of Massachusetts has sponsored a number of investigations of this proposal and of other methods of solving the problem. The results of the most recent studies were published in Massachusetts Senate No. 100, December 1, 1937, from which the following description of proposed remedial measures has been taken.

Treatment of the pollution problem of the Merrimack River has been studied from two angles. The ultimate goal, depending upon increased development of the Merrimack Valley, unforeseen at this time, includes complete purification of the river to the extent where it would be satisfactory as a water supply or for recreation and all industrial uses. This could be achieved by construction of a trunk line sewer with local connections and sewage treatment works located near the mouth, at an estimated construction cost of $27,000,000 and an estimated annual cost of maintenance and operation of about $300,000, or with similar results by an alternate plan involving local treatment works at an estimated construction cost of $20,000,000 with an estimated annual cost of maintenance and operation of $470,000.

The other suggested solution of the problem provides that only partial treatment of the worst sources of pollution be planned for and

that no attempt be made to provide sufficient purification of the river for recreation and water supply uses. It is considered that recreation and water supply requirements can be provided by other means at less cost by substituting a new joint water supply, releasing desirable lakes and ponds for recreational use. This partial treatment, including construction of local trunk line sewers, pumping stations, and appurtenances, would involve an expenditure of about 9 million dollars.

Because we are aware of no rational method for determining the annual damages attributable to pollution, an arbitrary determination of sanitation benefits from multiple-purpose reservoirs becomes necessary. This may be made by assuming these benefits to be equal to the saving over the cost of economically justifiable treatment works which would otherwise be required to provide comparable results. Under exceptionally favorable circumstances it might be possible to increase the low-water flow sufficiently to obviate the necessity for treatment works. In other cases the cost of required treatment works could be reduced.

Enough has been said about conditions in the particular case of the Merrimack Valley to indicate the difficulties inherent in the evaluation of benefits from abatement of stream pollution through increased dilution provided by multiple-purpose reservoirs. Such benefits are dependent on the planned use of the river, which necessarily involves plans for abatement of pollution. As no standards such as those which have been set for the Delaware River and for the New York Metropolitan Area have yet been adopted for acceptable water use in the various portions of the Merrimack River Basin, these benefits can only be determined from consideration of probable future action by local authorities with respect to the problem.

As earlier stated, the need for abatement of the pollution of the Merrimack River is largely accepted. However, the direct and indirect effects of water pollution on the complex human and economic activities in the basin are not satisfactorily expressed in monetary terms for the determination of a benefit-cost ratio. The extent of the provision of increased low-water flow would be predicated on what system of works and treatment should be undertaken to abate pollution in the Merrimack. The problem would then resolve itself into evaluating the effectiveness of increased low-water flow on the same engineering basis that sewage treatment systems are rated and determining the

savings in cost for each increment of increased low-water flow from a multiple-purpose reservoir. On this basis, monetary benefits can be obtained from such a reservoir. In the case of any recommended multiple-purpose reservoir, it is required, of course, that the benefit-to-cost ratio be favorable.

Summary

Reservoirs constructed and operated for flood control purposes alone yield sanitation benefits only to the extent to which the reduction in flood heights reduces stream pollution and other unsanitary conditions caused by great floods. Multiple-purpose reservoirs provide a means by which flood waters may be stored for later release to increase low-water flows, thereby abating pollution by dilution. The benefits from multiple-purpose reservoirs may thus include those from flood control, water power, sanitation and other sources.

Studies by the Engineer Department of waterway improvements include consideration of incidental effects as well as the direct purposes for which the improvements, as directed by Congress, are intended. For example, studies of the Merrimack River Basin for navigation, flood control and water power include studies of possibilities of incidental sanitation benefits from suggested multiple-purpose reservoirs.

Acknowledgment

Investigations of waterways in the Boston U. S. Engineer District are under the supervision of Colonel A. K. B. Lyman, District Engineer, Captain Donald G. White, Executive Officer, and Captain Emil J. Peterson, Chief, Engineering Division. Data concerning sanitation benefits from Tygart Reservoir were furnished by the Pittsburgh U. S. Engineer District of which Colonel W. E. R. Covell is District Engineer.

Mr. John R. Gardner, Assistant Engineer, provided valuable assistance in the preparation of this paper.

OF GENERAL INTEREST

THE JOHN R. FREEMAN LECTURES ON HYDRAULICS

The John R. Freeman Lectures on Hydraulics afforded the members of the Society a particularly fine opportunity to obtain up-to-date principles and methods in dealing with hydraulic problems and particularly the modern developments in the value and use of model analyses for hydraulic problems.

President Gordon M. Fair, in proposing this series of lectures, believed that the members of the Boston Society of Civil Engineers would take full advantage of this opportunity. The John R. Freeman Fund Committee, Mr. Charles T. Main, Chairman, and Robert Spurr Weston, Charles M. Allen and Howard M. Turner were in accord with this plan, and are gratified with the success of this series of lectures, the expense of which was met by the income of that fund.

The announcement concerning this series of lectures in the October, 1939, JOURNAL of the Society, mentioned that the lectures would be given principally by Prof. Kenneth C. Reynolds, Professor of Hydraulics at Massachusetts Institute of Technology. In addition, the final lecture of the series was given by Prof. Harold A. Thomas, Professor of Hydraulics and Sanitary Engineering, Carnegie Institute of Technology, Pittsburgh, Pennsylvania, whose special talk covered "The Propagation of Stable Wave Configurations in Steep Channels."

These lectures, available only to members of the Boston Society of Civil Engineers, were especially well attended. The maximum registered was 90; the lowest attendance at lectures was 69; the average attendance was 86. The series began October 17 and ended with the last lecture on December 19, 1939.

Supplementing the final lecture in the series Prof. Thomas gave a paper on "Recent Developments in Hydraulic Model Studies," illustrated, at the meeting of the Society held on December 20, 1939. This meeting was designated the John R. Freeman Night and was attended by 100 members and guests.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

OCTOBER 20, 1939.—A regular meeting of the Boston Society of Civil Engineers was held this evening at North Hall, Walker Memorial, Massachusetts Institute of Technology, and was called to order by the President, Gordon M. Fair, at 7:20 P.M.

This meeting was the annual joint meeting with the Student Chapters of the American Society of Civil Engineers, at Harvard University, Massachusetts Institute of Technology, Tufts, Rhode Island State College, New Hampshire State University, Worcester Polytechnic Institute, Brown University, and the Northeastern University Section of the Boston Society of Civil Engineers. About 300 members and guests attended the meeting and 290 persons attended the supper.

The President extended a cordial welcome to the more than 260 students present and expressed appreciation of the cooperation of the officers of the student organizations and of the faculty members in making this event so successful.

The Secretary reported the election of the following members on this date:

Grade of Student: Irving T. Berkland, Quinton Carlson, H. S. Church, Jr., Richard J. Ford, George W. Hankinson, Richard P. Heffernan, George E. Russell, Jr., William C. Peterson, Leon Piatelli, Harold C. Thomas.

The President stated that there is a vacancy on the Board of Government, due to the death of Prof. Henry B. Alvord, of Northeastern University, who had been elected at the annual meeting last March, and who died on April 19, 1939. In consequence of this the Nominating Committee, in accordance with the provisions of the By-Laws, presented its report, which was read, nominating Richard S. Holmgren, Chief Engineer, New Hampshire Water Resources Board, for Director, to fill the vacancy, until March, 1941.

The President stated that the election to fill this vacancy would be held by ballot at this meeting, due notice of which was given in the notice for this meeting. The ballots having been distributed, the President appointed Messrs. H. B. Kinnison and F. N. Weaver, as Tellers, to collect the ballots. The Tellers subsequently canvassed the ballots and their report will be presented at the next meeting of the Society.

VOTED—On Motion of Prof. F. N. Weaver, seconded by Prof. Haertlein that the Society express its appreciation of the courtesy of Massachusetts Institute of Technology in providing the facilities of Walker Memorial for this meeting.

The President then introduced the speaker of the evening, Prof. Hardy Cross, Professor of Civil Engineering at Yale University, who gave an inspiring and entertaining talk, entitled "Tredgold Said It".

The members gave a rising vote of thanks to Prof. Cross for his very unusual talk.

The meeting adjourned at 8:30 P.M.

EVERETT N. HUTCHINS, *Secretary.*

NOVEMBER 15, 1939.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Boston City Club. This was a joint meeting with the Northeastern Section

of the American Society of Civil Engineers. The meeting was called to order by Samuel M. Ellsworth, President of the Northeastern Section, American Society of Civil Engineers, who turned the meeting over to President Gordon M. Fair, of the Boston Society of Civil Engineers for matters of business to be acted upon by the Society.

The list of new members elected today will be announced at the next meeting of the Society on December 20, 1939.

President Fair stated that the Board of Government had voted to recommend that an amendment to the By-Laws be adopted, to provide for the waiving of the entrance fee in the cases of applicants for membership who are commissioned officers in the Armed forces of the United States and for employees of the permanent agencies of the Federal Government.

Prof. John B. Babcock, Chairman of the Committee on Membership, which advocated this amendment, stated that the committee believed that such an amendment to the By-Laws would make it advantageous, for engineers in Federal Services whose work is such that they are likely to remain in Boston and its vicinity only a few years, to become members of the Society.

On motion of Prof. John B. Babcock and seconded by Prof. Frederic N. Weaver *VOTED* that Section 8, of the By-Laws, relating to Fees and Dues, be amended in the first sentence so as to read as follows: *Section 8.* Fees and Dues. The entrance fee for Members and Associates shall be ten dollars; for Juniors, five dollars, and for Students, three dollars; except that for Commissioned Officers in the Armed forces of the United States and for employees of other permanent agencies of the Federal Government the entrance fee shall be waived after September 1, 1939.

The vote was unanimous. This matter will be presented for final action at the next meeting of the Society on December 20, 1939.

The following Resolution was passed:

Whereas the Boston Society of Civil Engineers and the Northeastern Section of the American Society of Civil Engineers wish to sponsor the erection of a memorial to the late Frank E. Winsor, Chief Engineer of the Metropolitan District Water Supply Commission, who died January 30, 1939, at the height of a distinguished career, leaving many friends in both Societies, and whereas said Commission has named the main dam of its Quabbin Reservoir, Winsor Dam, in his memory and in recognition of his services, and is willing to cooperate with these Societies in furnishing and preparing a suitable site for such a memorial, be it resolved that:

The Boston Society of Civil Engineers and the Northeastern Section of the American Society of Civil Engineers in their joint meeting on Wednesday, November 15, 1939, at the Boston City Club, appoint a committee to be known as the Frank E. Winsor Memorial Committee to consist of: Arthur D. Weston, Frank A. Barbour, Harrison P. Eddy, Jr., Samuel M. Ellsworth, Gordon M. Fair, Frederic H. Fay, Frank M. Gunby, Karl R. Kennison, and Robert Spurr Weston, for the purpose of receiving funds to erect a suitable memorial to be located alongside the Administration Road at Quabbin Reservoir overlooking Winsor Dam, and that Francis H. Kingsbury, Treasurer of the Northeastern Section of the American Society of Civil Engineers, be designated to receive such funds and act as Treasurer for the Committee.

President Ellsworth introduced the speaker of the evening, Col. William J. Wilgus, Honorary Member, American Society of Civil Engineers, who gave an extremely interesting talk on "America's Problem No. 1—Transportation".

Informal discussion and question period followed the talk.

The meeting adjourned at 8:45 P.M.

EVERETT N. HUTCHINS, *Secretary.*

DECEMBER 20, 1939.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club, and was called to order by Vice President Frank B. Walker. One hundred members and guests were present; ninety persons attended the dinner preceding the meeting.

The Vice President announced the death of Stephen deM. Gage, who died on October 2, 1939, and had been a member since May 19, 1915.

The Secretary reported the result of the special election held on October 20, 1939, when Mr. Richard S. Holmgren was elected Director, until March, 1941, to fill the vacancy on the Board of Government caused by the death of Prof. Henry B. Alvord, who died April 19, 1939.

The Secretary reported that the following 51 members were elected on November 15, 1939:

Grade of Member: Robert W. Anderson, George N. Bailey, Manuel A. Benson, Allen J. Burdoin, John J. Devine, Kenneth Eff, Francis J. Flynn, John W. Gurry, Daniel A. Howe, Karl Jetter, Victor H. Kjellman, Charles E. Knox, John P. Luby, John B. McAleer, George H. Mittendorf, Walter E. Merrill, Clarence N. Morang, Clarence L. Muntz, William C. Murray, Ralph C. Palange, Marshall A. Patch, Ralph F. Reinhardt, Chester A. Richardson, Harold A. Scott, Herbert B. Shumway, Harry M. Solomos, Harold A. Thomas, Jr., Medford T. Thomson, Freeman W. Towers,* Gardner K. Wood.

Grade of Junior: George Anthony, Russell H. Brown, Isaac Chase, Jr., Charles G. Edson, Charles E. Downe, John A. Hames, Wilbar M. Hoxie, Robert J. Greer, William L. Isherwood, Jr., Joseph M. Kennedy, Donald L. Milliken, Russell S. Pease, Arthur F. McVarish, W. Robert Williams, Arthur J. Yardley.

Grade of Student: Robert H. Brown, Robert D. Carpenter, George O. Huse, Paul D. Killam, Alan Levy, Malcolm J. Noonan.

*Transfer from Grade of Junior.

The Vice President referred to the series of lectures on Hydraulics which were concluded on Tuesday, December 19, and stated that these lectures had given an impetus to the increase in membership. This meeting is designated as John R. Freeman Night, and all new members were especially urged to attend this meeting.

Vice President Walker referring to the vote by the Society at the last meeting November 15, relative to an amendment to the By-Laws, stated that final action on this matter is to be taken at this meeting. *VOTED* that Section 8, of the By-Laws, relating to Fees and Dues, be amended in the first sentence so as to read as follows: Section 8. Fees and Dues. The entrance fee for Members and Associates shall be ten dollars; for Juniors, five dollars, and for Students, three dollars; except that for Commissioned Officers in the Armed forces of the United States and for employees of other permanent agencies of the Federal Government the entrance fee shall be waived after September 1, 1939.

Vice President Walker stated that the Board of Government has made a recommendation relative to the use of the income of the Permanent Fund.

VOTED that the Board of Government be authorized to use as much as may be necessary of the current income of the Permanent Fund for current expenses. Final action on this will be taken at the meeting of the Society on January 24, 1940.

Vice President Walker then introduced the speaker of the evening, Prof. Harold A. Thomas, Professor of Hydraulic and Sanitary Engineering at Carnegie Institute of Technology, Pittsburgh, Pa., who gave an extremely interesting talk on "Recent Developments in Hydraulic Model Studies", which was illustrated with stereopticon slides.

Prof. Thomas, is a leader in the application of model technique to the solution of hydraulic problems. During the past few years he has conducted a number of interesting and unique investigations for the Corps of Engineers and for various private companies.

By a rising vote of thanks the members of the Society expressed their appreciation of the talk.

Adjourned 9:30 P.M.

EVERETT N. HUTCHINS, *Secretary.*

DESIGNERS' SECTION

NOVEMBER 8, 1939.—The Designers Section of the Boston Society of Civil Engineers held a regular meeting in the Society Rooms this evening.

The meeting was called to order by the Chairman, Professor J. D. Mitsch, at 6:40 P.M. The minutes of the previous meeting were read by the Clerk and were approved.

The Chairman then introduced Mr. Preston M. Putnam, Senior Civil Engineer of the Metropolitan District Water Supply Commission, who spoke on the subject "Design of Concrete Mixtures for Concrete Pipe Construction." Mr. Preston described the concrete being used in the large concrete pipes now under construction for the Metropolitan Water Supply. Following his talk, he showed some excellent moving pictures which presented in detail the various construction phases of this project.

The meeting adjourned at 8:00 P.M.

Forty-two members and guests were present.

JOHN B. WILBUR, *Clerk.*

DECEMBER 13, 1939.—The Designers Section held a meeting in the Society Rooms this evening. The Secretary's Report of the November meeting was read and approved.

The Chairman then introduced the speaker of the evening, Mr. Howard Williams, Designing Engineer, Fay, Spofford and Thorndike. Mr. Williams spoke on the design of the Skating Rink for the Boston Skating Club. He described the problems involved in both the structural design of the building and the methods of producing and maintaining a smooth ice surface.

A question period followed the talk and the meeting adjourned at 8:15 P.M.

The talk was illustrated by lantern slides and the attendance was 33.

JOHN B. WILBUR, *Clerk.*

JANUARY 10, 1940.—The January meeting of the Designers Section was held in the Society Rooms on January 10, 1940. The meeting was called to order by the Chairman at 7:10 P. M. The Secretary's Report was read and accepted. It was voted that the Chairman appoint a Nominating Committee for the purpose of bringing in a slate of officers for the next year at the February meeting.

The Chairman then introduced the speaker of the evening, Commander J. P. Searles, Civil Engineer Corps, U. S. Navy, Public Works Officer of the First Naval District. Commander Searles described the general organization of the Navy and the part of the Civil Engineer Corps in this organization. He then described the South Boston Drydock and the new construction which is being carried out in connection with this project. He described in particular the new quay walls. The talk was illustrated with slides and was followed by a discussion.

The meeting adjourned at 8:30 P. M. Fifty-six members and guests were present.

JOHN B. WILBUR, *Clerk.*

HIGHWAY SECTION

OCTOBER 25, 1939.—A regular meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening in room 200, Northeastern University (new building), Boston, Mass.

The meeting was called to order by the Chairman, Mr. Arthur E. Harding, who introduced the speaker of the evening Mr. Vincent K. Cates, Technical Service Engineer of the Universal Atlas Cement Company, who spoke on the subject "Manufacture of Portland Cement".

The speaker, with the help of a blackboard diagram, very interestingly traced the various processes in the manufacture of cement from the raw ingredients in the quarry to the finished cement product.

Two illustrated motion picture films entitled "From Mountain to Cement Sack" and "Construction That Endures" were shown, at the completion of which there was a brief question period.

The speaker was accorded a rising vote of thanks and the meeting adjourned at 9:35 P.M. with an attendance of 57 members and guests.

PARKER HOLBROOK, *Clerk.*

NOVEMBER 22, 1939.—A regular meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening in the Society's rooms, 715 Tremont Temple.

The meeting was called to order by the Chairman, Mr. Arthur E. Harding.

The reading of the minutes of the previous meeting was omitted.

A motion to appoint a nominating committee made and voted.

The Chairman appointed: Prof. E. A. Gramstorff, Prof. A. J. Bone, and Mr. T. C. Coleman.

The Chairman then introduced the speaker of the evening, Mr. H. R. Swartz, Editor, "New England Construction", who spoke on the subject "Developments in Methods and Equipment of Road Construction".

The speaker interestingly traced the developments in methods and equipment used in moving large quantities of earth and rock on various construction projects from the heavy, cumbersome steam shovel and horse-drawn bottom dump wagons, through the developments in the use of gasoline-driven equipments, to the present trend to diesel power.

Stereopticon pictures showing the use of equipment on various construction work throughout New England were shown with explanatory remarks by the speaker, and a brief question period closed a most interesting evening.

The speaker was accorded a rising vote of thanks and the meeting adjourned at 9:05 P.M. with an attendance of 12 members and guests.

PARKER HOLBROOK, *Clerk.*

NORTHEASTERN UNIVERSITY SECTION

NOVEMBER 3, 1939.—A meeting of the Northeastern University Section of B.S.C.E. was held this evening in Room 440W.

The meeting was called to order at 6:45 P.M., by the Chairman Louis G. Reiniger. The minutes of the last meeting, held May 18, 1939, by Division B, were read and approved, after which the Chairman outlined the program that was being carried out in Division A as follows:

This year's activities opened with a talk on Thursday afternoon, October 19, 1939, by Joseph Higgins, N.U. '39, on the construction of the North District Sewer of the Metropolitan District Commission. On the following day, October 20, seventeen juniors and senior members went to Chelsea and inspected the job.

On the same night, approximately seventy Northeastern students of both divisions attended the B.S.C.E. Student Night held at Walker Memorial, M.I.T., and heard Professor Hardy Cross, of Yale University speak on the topic, "Tredgold Said It."

On Tuesday noon, October 31, 1939, the Section sponsored a lecture and moving picture on the Metropolitan

pressure aqueduct and the construction of the pipe for it. Mr. Albert S. Genaske, assistant civil engineer for the Metropolitan District Water Commission, was the lecturer. This lecture will be followed up by a visit to the Lock Joint Pipe Company in Natick, and the aqueduct, on Saturday, November 4, 1939.

Friday, November 10, 1939, the juniors and seniors are planning a trip to view the foundation construction at the new building of the New England Mutual Insurance Company on Boylston Street. This trip will be preceded by an explanation of the work in a talk to be given Thursday noon at the School by Mr. R. C. Pitcher, owner's and architects' representative.

Following the program outlined, the Chairman explained the essay contest that is held each year in the Section, and the acting clerk read the rules as set up by the B.S.C.E. The lecture of the evening was part of this contest.

The remainder of the meeting was taken up by the lecture, "How Combat Engineers Span a River," given by Mr. Louis G. Reiniger, senior in Civil Engineering at Northeastern, and Private in "D" Company, 101st Combat Engineers, M.N.G. He first explained the need for temporary bridges in war time, the kinds of conditions met in building them, and the types used for these varying conditions. He explained in detail how his Company spanned a river during maneuvers at Plattsburg, N. Y. The lecture was concluded by the showing of slides of the construction.

There were twenty-seven present at the meeting, which adjourned at 8:30 P.M.

WINFIELD B. KNIGHT,
Acting Clerk.

APPLICATIONS FOR MEMBERSHIP

[January 20, 1940]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

KIRKWOOD B. BROWN, Waban, Mass. (b. February 5, 1912, Dorchester, Mass.) Graduated from Northeastern University, 1936, with B.S. degree in Civil Engineering. Experience, December 1936, to December 1937, with Stone & Webster as structural draftsman. October 1938, to date, with Metropolitan District Water Supply Commission as senior engineering aide. Refers to *E. S. Averell, A. E. Everett, E. A. Gramstorff, C. C. McCully.*

JOSUE GOLLAN, Santa Fe, Republica Argentina. (b. August 13, 1891, Santa Fe, Rep. Argentina.)) Doctor in Chemistry graduated from the Facultad de Ciencias Exactas, Fisicas y Naturales of the Universidad Nacional de Buenos Aires, on November 1913. President of the Consejo Directivo of the Instituto Experimental, de Investigacion y Fomento Agricola Ganadero, Santa Fe. Rector of the Universidad Nacional del Litoral, Santa Fe. Professor of Agri-

cultural Chemistry at the Facultad de Quimica Industrial y Agricola of the Universidad Nacional del Litoral, Santa Fe.

LAURENCE G. LEACH, Providence, Rhode Island, (b. December 16, 1910, Cleveland, Ohio.) Graduated from Shaw High School, June 1928. Received B.S. degree at Case School of Applied Science, 1932. Experience, one year Municipal Engineering work, City of South Euclid, Ohio. Six months Construction work, National Air Races, Cleveland, Ohio. Four months Surveying work, Cuyahoga County Ohio. Seven months, U. S. Engineer Office, Zanesville, Ohio. Hydraulic Studies and Hydraulic Design, including Hydraulic Model Studies at Case School of Applied Science at Cleveland. For sixteen months stationed at the Leesville Dam, Ohio, in charge of Concrete Design and Control and Tunnel Operations. Since September 1936, stationed at the Providence U. S. Engineer Office, duties include Economic Studies, Hydraulic Design and Hydrology Studies. Rated as Assistant Engineer. A Junior member of the American Society of Civil Engineers, also registered as Professional Engineer and Surveyor in the State of Ohio. Refers to *J. C. Dingwall, H. B. Kinnison, G. H. Mittendorf, A. J. Ober, H. A. Scott.*

PETER A. MURPHY, Arlington, Mass. (b. April 19, 1909, Medford, Mass.) Lowell Institute, M.I.T.-1929-1932, Bldgs Course; Advanced Reinf. and Advanced Testing Material Courses. Mass. Institute of Technology, 1932-1933, Civil Engineering. Experience, Stone & Webster, Inc., Boston, Mass., Draftsman, Drafting for hydro-electric plants and industrial buildings (1929-1931). Boston Elevated Ry., Boston, Mass., 1931-1932, Designer-Investigation of foundations and design of steel and reinforced concrete for steam power plants. 1932-1933, student at Mass. Institute of Technology. U. S. Coast and Geodetic Survey, Boston, Mass., 1933-1935, Observer, Instrument man with second order triangulation and traverse party. Corps of Engineers, U. S. Army, 1935-1937, Lieutenant, detailed as engineer officer for CCC hospital and barrack construction at Fort Devens and Fort Adams. U. S. Engineer Department, Boston, Mass., 1937 to date, Junior Engineer, preliminary estimates and design for flood control structures in New England. Also final design of structures for flood control dam. Refers to *J. D. Guertin, J. H. Harding, D. F. Horton, E. F. Kelley.*

For Transfer from Grade of Student

CHARLES G. HUNT, JR., Newton Center, Mass. (b. Sept. 6, 1915, Newton Highlands, Mass.) Graduated from Northeastern University, June, 1939, with B.S. degree in Civil Engineering. Experience, April, 1935, to November, 1938, with the City of Newton Engineering Department, as Rodman and also as acting Transitman. February, 1939, to April, 1939, with the Metropolitan District Commission, Sewer Division, as Junior Engineering Aid on the North Metropolitan District Relief Sewer in Everett. June, 1939 to present with the Metropolitan District Water Supply Commission as Junior Engineering Aid on the Southboro Tunnel section of the new pressure aqueduct in Southboro. Refers to *C. O. Baird, A. E. Everett, E. A. Gramstorff, J. J. Vertic.*

BURRITT F. LEIGHTON, Brockton, Mass. (b. July 9, 1916, Brockton, Mass.) Graduated from Northeastern University, June, 1939, with B.S. degree in Civil Engineering. Experience, Cooperative work at Northeastern, October 1935 to May 1936, City Engineering Department, Brockton, Mass., as Rodman; May 1936 to June 1939, Hayward & Hayward, Civil Engineers, Brockton, Mass., as instrumentman and blueprinter. At present employed by the Metropolitan District Water Supply Commission, working on the cut and

cover aqueduct located in Southboro. Duties have included office work, inspection of concrete, and instrumentman. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, Jr., E. A. Gramstorff.*

JOHN H. MANNING, Newton, Mass. (b. February 27, 1916, Boston, Mass.). Graduated from Northeastern University in 1939, with B.S. degree and honor in Civil Engineering. Experience, Cooperative work student with City of Newton Engineering Department from April, 1936, to June, 1939, as a rodman. Employed from June, 1939, to present by City of Newton Engineering Department as a Transitman. Refers to C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff, A. Q. Robinson.

MEMBERS ELECTED NOVEMBER 15, 1939.

The following were elected to membership, with others, at the meeting held on November 15, 1939. Their applications, having been received subsequent to the printing of the October, 1939, JOURNAL, are printed here for the purpose of record.

For Admission

GEORGE N. BAILEY, Pembroke, Mass. (b. October 26, 1886, Wells River, Vt.) Received A.B. degree at Middlebury College, 1908. B.S. and E.E. at Worcester Polytechnic Institute, 1913. Experience, 1913 to 1919, private practice. 1919 to 1921, New England Power Company, assistant to Supt. of Steam Power. 1921 to 1928, private electrical, mechanical, and civil engineering practice. 1928 to date, New England Power Company, assistant engineer in hydraulic department. Refers to *C. M. Allen, C. R. Bliss, H. B. Kinnison, E. G. Lee, T. H. Safford.*

JOHN A. HAMER, Providence, Rhode Island. (b. December 16, 1913, Manchester, N. H.). Graduated from Trinity College, Hartford, Conn., 1935, with B.S. (honors in Civil Engineering). Experience, July, 1935 to Aug., 1936, rodman with Metropolitan Water Bureau, Hartford, Conn. Field Office work connected with construction of Saville Dam. August, 1936, to date, U. S. Engineer Office, Providence, R. I., flood hydraulics, compilation of rainfall and stream flow data, distribution unite graph determination and their relation to water shed characteristics, flood routing, determination of benefits from various reservoir and dike plans by the damage, frequency method, determination of most economical reservoir capacity allowing for natural valley storage, and storm and flood frequency studies. Spent 8 months at the hydraulic laboratory of Worcester Polytechnic Institute constructing and testing models of hydraulic features of proposed flood control dams. At present have charge of sub-section studying economics of flood control system for Connecticut River Watershed. Refers to *C. M. Allen, J. C. Dingwall, L. J. Hooper, W. I. Kenerson, H. B. Kinnison.*

WILBAR M. HOXIE, Northfield, N. H. (b. February 9, 1917, Somerville, Mass.). From September 1933, to December 1935, School of Architecture, University of New Hampshire. December 1935, to June 1938, Civil Engineering, University of New Hampshire, graduating June, 1938, with B.S. degree. Experience, June and July, 1938, Fort Wright, New York, U. S. Army, CAC. 2nd Lt. August 1938 to December 1938, Junior Engineering Aide in Metropolitan Water Supply Commission on aqueduct, field party and office. December 1938, to present date, Assistant Engineering Aide in U. S. Engineer Office, Hydraulics Section, Providence, R. I., on unit graph studies, etc. 1938 and 1939, Massachusetts Extension School at M.I.T. Junior member of the American Society of Civil Engineers and member of American Society of Military Engineers. Refers to *H. B. Kinnison, A. T. Safford, G. A. Sampson, C. W. Sherman, R. S. Weston.*

WALTER E. MERRILL, West Medford, Mass. (b. March 17, 1899, Somerville, Mass.). Harvard College, A.B. *cum laude,* 1911. Mass. Institute of Technology, B.S. degree, 1913. Experience, June, 1913, to March, 1917, rodman, transitman, assistant civil engineer, Metropolitan District Commission. March, 1917, to March, 1918, to date, assistant Sanitary Engineer and Senior Sanitary Engineer, Massachusetts Department of Public Health. Refers to *G. G. Bogren, F. L. Flood, F. H. Kingsbury, A. D. Weston, E. Wright.*

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*Transfer from Grade of Junior.

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DEATHS

FREDERICK A. LOVEJOY, December 21, 1939
STEPHEN DE M. GAGE, October 2, 1939

BOOK REVIEWS

Engineering's Part in the Development of Civilization, 1939, by DR. DUGALD C. JACKSON, Professor Emeritus of Electrical Engineering, Massachusetts Institute of Technology. Published by American Society of Mechanical Engineers, New York, from "Mechanical Engineering", July to December, 1938.*

Engineers, as a rule, are the Marthas of applied science. They are absorbed and engrossed in the daily task to such an extent that they frequently fail to understand their place and the place of their work in the world's scheme, and frequently lack all inclination to understand. This book, the result of six lectures delivered by Professor Jackson at the University of North Carolina State College of Agriculture and Engineering in 1938, will go a long way to develop the point of view of the Marys, and should offer the engineer and his work a raison d'etre.

The narrative opens before the dawn of recorded history, and, with the inseparable relation between engineering and civilization as the thread, traces the development of man from the brute stage of tooth and claw to the present stage of airplane and tank; and while the author seems to be optimistic, and speaks of wars and other evils in the past tense, the reader, with more recent developments in mind, may feel that little real progress has been made.

The book obviously is the result of much reading, and more thinking. In addition to the picture of the past, scenes such as England in the 18th Century, and sketches of the misery of past ages; and in addition to the exposition of the advance of humanity through engineering, the author, ever the individualist, inserts his opinions as to such vital matters as war and peace, preparedness, political situations and politicians, democracies and dictatorships, and various abuses and their remedies; and proposes to transform the dismal science of economics into a radiant and useful science through the application of engineering principles.

*By Frederic N. Weaver, Professor of Civil Engineering, Tufts College, Tufts College, Mass.

In short, this little book of somewhat over one hundred pages gives the engineer a comprehensive picture of his history and activities, relates him to the past and to the present, gives him a background which he sadly needs, and does it in such brief compass that even the busy engineer should find time in which to read it, and then find more time in which to think over the narrative, and analyze the conclusions.

"Flash!", 1939, by DR. HAROLD E. EDGERTON. Hale, Cushman & Flint, Boston. 203 pages. Price, $3.00.*

For the past fifteen years, Dr. Harold E. Edgerton of the Massachusetts Institute of Technology, has been perfecting the stroboscope. During this period he has acquainted the public with his work, so that today there are few who do not know what it is all about, or have not seen his ultrahigh-speed photographs. Who has not coveted a set of his single and multiple exposure photographs—particularly the latter? Now you can own them, for Dr. Edgerton and his colleague, James R. Killian, have collaborated in the production of the book, *"Flash!"* (Hale, Cushman & Flint, Boston. 203 pp. $3.00).

"Flash!" embodies all that one could wish for and it brings together for the first time, in book form, a most fascinating series of more than 200 photographs "seeing the unseen". Anyone acquainted with Dr. Edgerton's work will want this book, for here is the cream of many thousands of photographs of everyday objects, motions, and details that no man has ever seen before. In this respect the book speaks for itself. If you are not already familiar with this type of photography, now is the time to get acquainted.

As one thumbs through the pages he sees motion "frozen"; pictures taken at one-millionth of a second, startling in their clarity of detail; also many spectacular multiple exposure high-speed photographs. You can study and see the action of a humming bird as it hovers over a vial of sweet water; see bullets in action; and see what happens to your club when you make that long drive. Archery, football, baseball, tennis and others sports receive their share of attention. Such homely things as a falling drop of milk, water running out of a faucet, bubbles, and many other actions are shown which contain rare beauty when photographed Edgerton's way. The use of the stroboscope and the photographs is clearly explained in simple terms.

Dr. Edgerton has probably done more than any other individual to show the man on the street that out of the laboratories of research comes much that is practical and many things that affect him immeasurably in everyday life.

Multiple exposure photographs may be for scientific purposes, but I would rather hang one on my wall than some of the so-called modern art. Edgerton has caught all the beauty inherent in the action of man, animal, and machine. He has brought to our attention a new field of art. It may not be long before our museums are displaying the rare and beautiful — shall I say — Edgertonographs.

Stars and Men, by STEPHEN A. IONIDES and MARGARET L. IONIDES. Bobbs Merrill, New York. 460 pages. Price, $4.00.*

When a group of friends seat themselves around a crackling fireplace in a log cabin, on a mountainside, anything can happen. Conversation (not chatter) in such surroundings often becomes so stimulating that those present are imbued with the desire to continue it and

*By R. Newton Mayall, Landscape Architect and Engineer, 115 Newbury Street, Boston, Mass.

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carry it farther at some later date, even under other circumstances. Stephen A. Ionides and Margaret L. Ionides tell us their book, *Stars and Men* is the result of such a conversation, which finally settled down to business on the subject of astronomy.

Stars and Men is an interesting and entertaining story of the influence of astronomy on man, written by a consulting engineer and his daughter. The treatment simulates a discussion among friends, of an evening. The completeness of the book and variety of subjects is evident by a glance at its contents. There are chapters on time, the calendar, navigation, geography, etc., which are a direct result of astronomy and man's application of its mechanics—these are combined with other subjects such as eclipses, the sun, planets, stars, and constellations, which are usually found in textbooks.

The Ionides have made their book more pleasurable by the skillful insertion of anecdotes, superstitions, and historic incidents, together with illustrations and diagrams. *Stars and Men* does not require any previous knowledge of astronomy for the enjoyment of its contents. It may well be the mentor for many readers.

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