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CONTENTS

PAPERS AND DISCUSSIONS

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UNDISTURBED CLAY SAMPLES AND UNDISTURBED CLAYS*

By Karl Terzaghi, Member†

(Presented at a meeting of the Boston Society of Civil Engineers held on January 8, 1941.)

Introduction

Since it is impossible to secure perfectly intact samples of clay (Terzaghi, 1936) the computation of the settlement of structures founded above beds of clay requires assumptions concerning those properties of the clay which cannot be determined by laboratory tests. The following paper deals with these assumptions. It also deals with the writer's conception regarding the physical causes of the difference between perfectly undisturbed clays and those of so-called undisturbed samples. For the sake of simplicity the discussion will be limited to clays which have never been under a pressure in excess of that produced by the weight of the existing overburden. Clays of this type are called normally consolidated clays.

The Virgin Compression Curve

Current conceptions regarding the consolidation of clay strata due to the weight of superimposed buildings are based on laboratory experience and on the results of settlement observations. In order to understand the origin of these conceptions let us assume that we have secured an undisturbed sample of a normally consolidated clay from

*This paper covers in general the subject of the paper entitled "The Critical Load on Strata of Clay beneath Foundations" presented at the January meeting of the Society.

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a certain depth below the surface of the ground. The water content of the sample is 45 per cent, corresponding to a void ratio e_0=1.21. The effective vertical pressure which acted upon a horizontal section through the sample, prior to its removal from the ground was p_0=2 tons per sq. ft. Since the deposit is normally consolidated the sample has never been acted upon by a higher pressure. In Fig. 1*a*

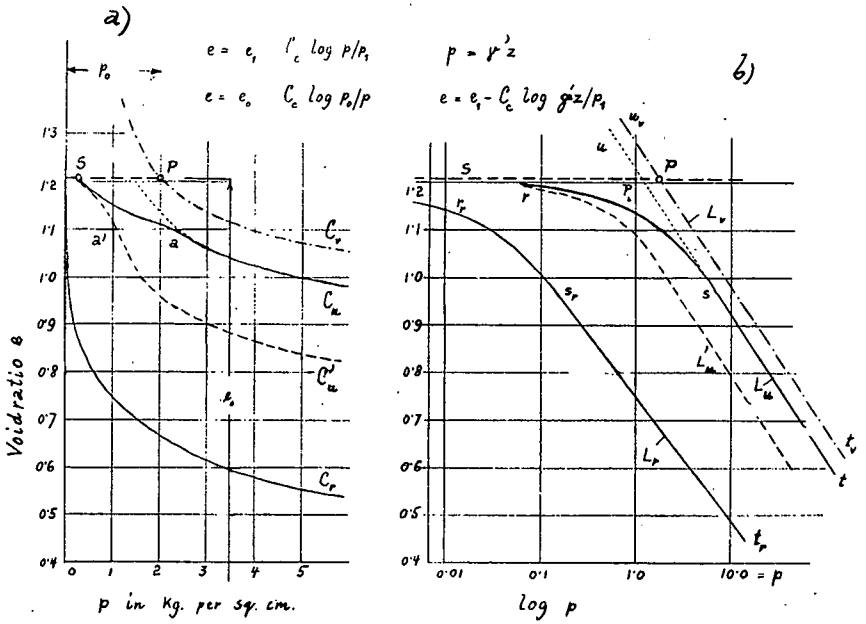

FIGURE 1*a*.

the abscissas represent the effective vertical pressure on the clay and the ordinates the corresponding void ratio. The original state of the clay is indicated by the point *P*. During the operation of sampling the void ratio remains practically unchanged, but the effective vertical pressure on the sample is considerably reduced. This state is shown by point *S*, the ordinate of which is equal to that of point *P*. The next step consists in submitting the sample to a consolidation test. The results of this test depend to a large extent on the degree to which the bond between the clay particles has been injured during the process of sampling. If the sample has been taken carefully, we obtain the curve C_u. At point *a* the curve shows a gentle break.

A. Casagrande (1932) has shown that the abscissa of point a is approximately equal to the pre-consolidation pressure, that is, to the greatest pressure to which the sample has ever been subjected in its previous history. For a normally consolidated clay this pressure is identical with p_0, Fig. 1a.

If the sample has been somewhat disturbed during the process of sampling a pressure void ratio Curve C'_u which is entirely located below C_u is obtained. The abscissa of the break a' in the curve is appreciably smaller than p_0. Finally, if the sample is tested after having been completely remolded at unaltered water content, the curve C_r is obtained. This curve has no break.

To show the characteristics of the test results more clearly the semi-logarithmic plot shown in Fig. 1b is useful. In this plot each of the curves L_u, L'_u and L_r, corresponding to the curves C_u, C'_u and C_r respectively, consists of a gently sloping, curved upper portion and a steep, straight lower portion. The straight line portion of each curve, as for instance the straight portion st of the curve L_u, can be described by the empirical equation

$$e = e_1 - C_c \log \frac{p}{p_1} \tag{1}$$

wherein p_1 is an arbitrary reference pressure, e_1 is the void ratio corresponding to the pressure p_1 on the straight line tu and C_c is an empirical coefficient, called the compression index. Rutledge (1939) has demonstrated that the value of C_c increases in a general way with increasing values of the natural water content of a clay. For a clay with a natural water content of 45 per cent such as that represented by Fig. 1 the value of C_c ranges between 0.3 and 0.6. The curve L_u represents a clay for which C_c=0.3. The value of C_c decreases slightly as the degree of remolding increases. For normally consolidated clays the extension su of the straight portion st of the pressure void ratio curve obtained from undisturbed samples always intersects the horizontal line PS at a point P_i which is located in the vicinity of the point P. The more the bond between the clay particles has been damaged during the process of sampling, the more the point P_i moves towards the left.

On the basis of observations similar to those shown in Fig. 1 it has become customary to assume that the decrease of the void ratio produced by increasing the load on a stratum of normally consolidated

clay from the overburden pressure p_o to a given pressure p could be computed by substituting $p_o = p_1$ and $e_o = e_1$ into Equation 1. Thus we obtain

$$e = e_o - C_c \log \frac{p}{p_o} \tag{2}$$

In Fig. 1*b* this equation is represented by the straight line $u_v t_v$ which is parallel to ut. The line $u_v t_v$ has been given the name *virgin compression* curve. The curve C_v in Fig. 1*a* represents the same curve in an arithmetic plot. If Equation 2 were justified, every increase of the load on the clay should initiate a normal process of consolidation. The settlement due to increasing the load on the clay by Δp should be greater than one-half of the settlement produced by an increase of $2\Delta p$ and the process of consolidation should take place in every bed of clay beneath the site covered by the surcharge, regardless of the depth at which such a bed is located.

The Sedimentation Compression Curve

The present state of every sedimentary, normally consolidated clay has been preceded by a gradual increase of the pressure from zero to the overburden pressure p_o. If Equation 1 were valid for both the process of loading in the laboratory and the process of loading by sedimentation in nature it should be possible to determine by means of this equation the relation between depth and void ratio. At any depth z below the surface of a submerged deposit of normally consolidated clay the overburden pressure p_o is equal to

$$p_o = z\zeta' \tag{3}$$

wherein ζ' is the submerged unit weight of the clay. For a soft clay which has never carried any surcharge the upper end of the straight line portion of the semi-logarithmic consolidation curve corresponds to a pressure of not more than about 200 lbs. per sq. ft. In a submerged bed of clay this overburden pressure exists at a depth of about $z_1 = 3$ ft. below the surface. Introducing the value 200 lbs. per sq. ft. for p_1 and the value of $z\zeta'$ for p into Equation 1, the following equation is obtained

$$e = e_1 - C_c \log \frac{z\zeta'}{200} \tag{4}$$

wherein e is the void ratio of the clay at a depth of 3 ft. below the surface of the deposit, e is the void ratio at a depth z meters below the

surface and ζ' is the submerged unit weight of the clay in lbs. per cu. ft.

The writer has knowledge of three reliable records concerning the relation between the depth and the void ratio of submerged deposits of normally consolidated uniform clays. Yet none of them shows any resemblance to what should be expected on the basis of the theory represented by Equation 4. Hence it is necessary to distinguish between the virgin compression curve derived from the results of laboratory tests, which is a hypothetical curve, and the *sedimentation compression curve* which represents the real relation between the void ratio and the very slowly increasing pressure due to the weight of the overburden in a clay deposit during the process of sedimentation.

The first information concerning the relation between depth and void ratio of a bed of clay in a state of very slow growth by sedimentation was secured by the author in 1924. Samples were taken in boreholes drilled to a depth of about 80 feet below the surface into the deposit of soft organic clay which covers the bottom of the Golden Horn near Istambul. The drill holes were located at a short distance off the shore near Aiwan Serai. The liquid limit of the samples ranged between 56 and 65 per cent. The water content of the uppermost samples which were obtained at a depth of about 5 feet, were approximately equal to the liquid limit. According to the results of consolidation tests on remolded samples the water content of the samples from a depth of 80 feet should have been about 20 per cent below the liquid limit. Yet, these water contents were almost as close to the liquid limit as those of the samples obtained near the surface. These findings as compared to the results of the consolidation tests confronted the author with what appeared at that time to be an insoluble puzzle.

A second record was obtained from borings made in 1939 at the author's request in a deposit of micaceous lacustrine clay at the southern end of the Lago di Resia in upper Italy. The clay originated from a terminal moraine and the surface of the deposit was covered with shallow water. Altogether thirty samples were secured from 7 drill holes having a maximum depth of 60 feet. The liquid limit of most of the samples ranged between 50 and 75 per cent. With few exceptions the water content of each sample was close to the liquid limit. Yet the difference between the liquid limit and the natural water content had no tendency to increase with depth. In this respect the

record is a duplicate of that obtained at the Golden Horn for a marine clay.

A third record was published by B. Fellenius (1936). It shows the relation between the water content and the depth below the surface of a remarkably uniform deposit of soft clay in the Gota River in southern Sweden. The surface of the deposit is covered by about ten feet of water. Curve C_s in Fig. 2 represents Fellenius' data, recalcu-

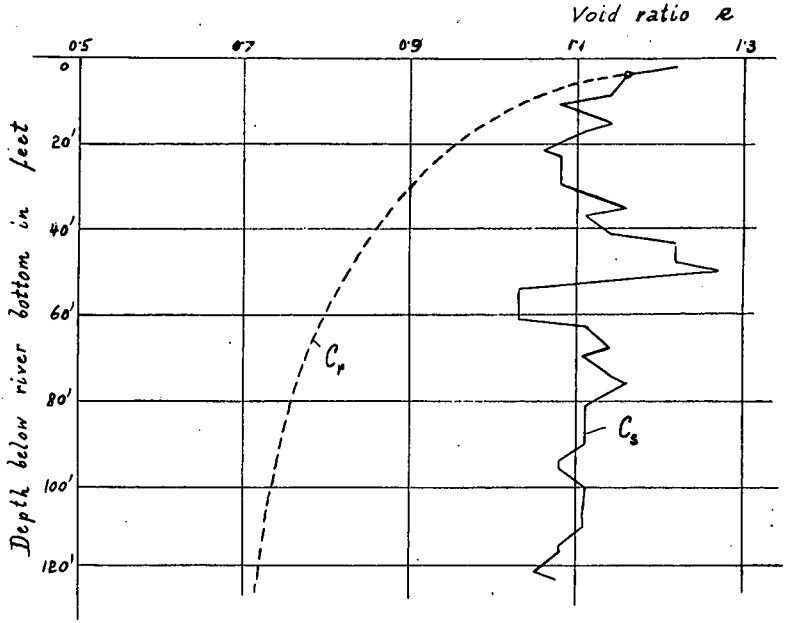

FIGURE 2.

lated on the assumption that the average unit weight of the solid soil particles is 2.7 gm. per cu. cm. As stated previously, the compression index C_c for a clay with a natural void ratio of 1.2 ranges between 0.3 and 0.6 (Rutledge, 1939). The average immersed weight of the clay is about 0.82 gm. per cu. cm. Assuming the lowest value, 0.3, for C_c we obtain by means of Equation 4 for the relation between the depth and the void ratio the dashed curve C_r in Fig. 2. In contrast to the empirical curve it indicates that the water content should decrease rapidly with depth. For higher values of C_c the difference between the theoretical and the empirical curves would be still more conspicuous.

In order to account for the records described above we are compelled to assume that the average slope of the sedimentation compression curve is extremely small compared to that of the curves obtained from laboratory experiments. Fig. 3 illustrates this state-

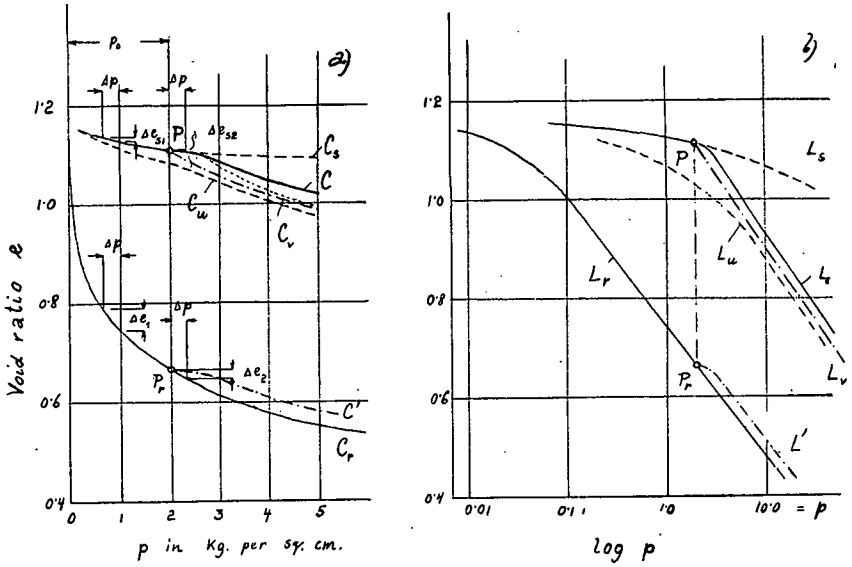

FIGURE 3.

ment. In this figure the curve C_r represents the pressure-void ratio curve for a remolded specimen of a clay having an initial water content, w_o=45 per cent, equal to the liquid limit. Assuming that the water content of the clay at a depth of 3 ft. below the surface is also equal to the liquid limit, the curve C_s is the steepest sedimentation compression curve which appears to be compatible with the aforementioned field records.

The conspicuous difference between the trend of the two curves, C_s and C_r in Fig. 3*a* becomes comprehensible if we consider the difference between the rate of loading which corresponds to these two curves and the influence of this difference on the compression produced by the load. It is known that every soil particle is surrounded with a very viscous film of adsorbed water the presence of which delays the establishment of a direct contact between the solid soil constituents (Terzaghi, 1925). Before this contact exists the films

of adsorbed water act as a lubricant. Therefore the rate at which the pressure on the clay is increased must have an influence on the resistance against intergranular slippage. This conclusion is in accordance with some laboratory tests which have been carried out by Langer (1936) on a stiff plastic clay from the vicinity of Paris, France. Fig. 4 is a semi-logarithmic plot of the test results. It shows that

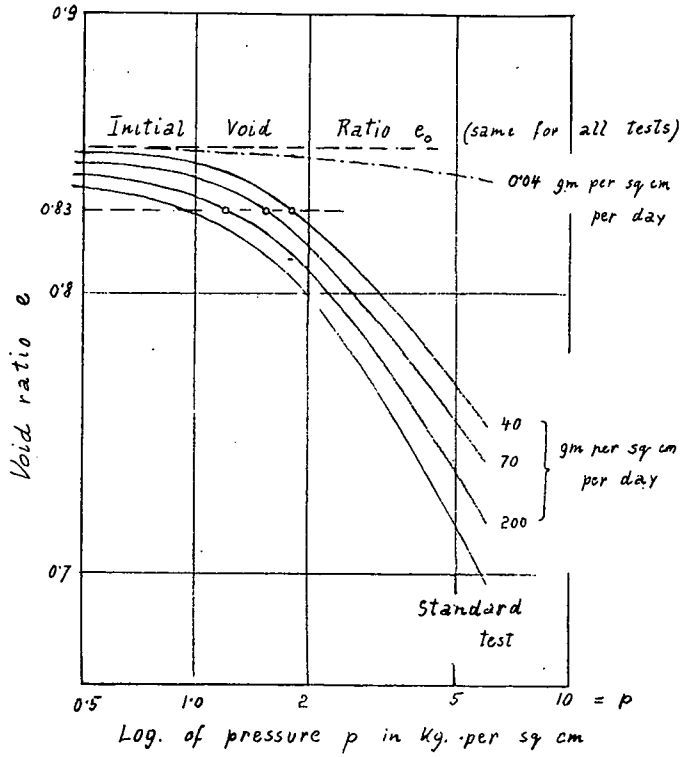

FIGURE 4.

the pressure p required for reducing the void ratio of the clay to a given value, for instance 0.83, increases with decreasing rate of loading. The lowest rate of loading, 40 gm. per sq. cm. a day, is about ten thousand times as rapid as the rate at which nature increases the pressure on a bed of clay during the process of sedimentation. The dash-dotted curve illustrates the writer's opinion concerning the shape and the trend of the curve which one would obtain if loading the

clay at a rate of 0.04 gm. per sq. cm. a day. This rate corresponds to the rate of loading on a clay during the process of sedimentation in nature. For two other clays which have also been tested, a reduction of the rate of loading from 200 to 40 gm. per sq. cm. a day had no measurable effect on the relation between pressure and void ratio. This seems to indicate that the effect of the rate of loading on the relation between pressure and void ratio is different for different clays. In order to investigate this effect in the laboratory, very much lower rates of loading than the lowest one in Langer's series should be used.

The following investigations will show that the existence of adsorbed films is likely to account not only for the difference between the general trend of the two curves, C_s and C_r in Fig. 3*a*, but also for the other differences between the properties of undisturbed and remolded clays.

Solid and Lubricated States of Clays

Fig. 5 is a graphic representation of the author's conception regarding the interaction between adsorbed layers. (Terzaghi, 1925, 1926.) It represents magnified ideal sections through the vicinity of a point of contact between two soil particles. In the immediate vicinity of the surface of the solid particles the adsorbed water is solid and its density is far above normal. With increasing distance from the surface of this solid film both the density and the viscosity of the water decrease and beyond a certain distance d the properties of the water are normal. The distance d depends both on the chemical properties of the solid and on those of the substances other than water which are present within the zone of adsorption. Thus, for instance, if the water in the voids of a bentonite specimen contain sodium salts in solution, the adsorbed layers are very much thicker than those in one saturated with pure water. The views regarding the nature of the effect of the solid on the water are still in a controversial state. Yet, the existence of the layers and the conspicuous deviation of their physical properties from those of normal water have been conclusively demonstrated by numerous and very different methods of investigation.

When two soil particles are pressed together with a force Q the outer zones of their adsorbed layers merge as shown in Fig. 5*a*. This

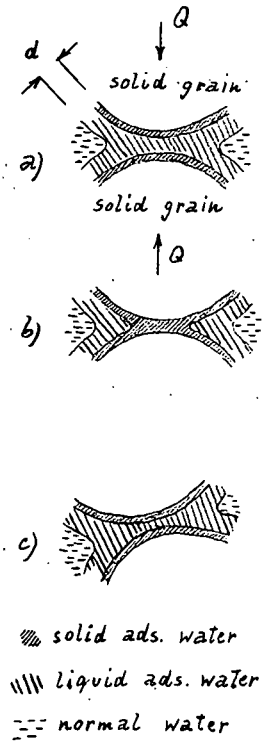

FIGURE 5.

event occurs instantaneously. However, in this state they are still separated from each other by a liquid though very viscous layer of adsorbed water. The further approach between the particles takes place at a rapidly decreasing rate, until finally the solid portions of the adsorbed films come into actual contact.

If p is the intensity of the effective vertical pressure per unit of area of a sample of clay, the average pressure Q per point of contact between two clay particles is equal to p divided by the number of points of contact per unit of area. The value Q represents the statistical average of the real contact pressures. The individual values may be greater or smaller than mean value. Hence at some points of contact the rigid bond will be established much more rapidly than at others. The gradual development of a solid bond between the clay particles will be called the *process of solidification.* Before the clay is com-

pletely solidified, some of the particles are already connected with each other by a solid bond, due to direct contact between the solid parts of the adsorbed layers. Others are held together by the highly viscous, yet liquid portion of the adsorbed layers. Both types of bond participate in the transmission of the pressure from grain to grain. Hence it is necessary to subdivide the effective stress in a clay into two parts. One part, the *solid bond stress,* is entirely carried by the bond between the solid portions of the adsorbed layers. The second part, the *film bond stress,* is carried by the viscous resistance of the adsorbed films. As long as part of an effective stress consists of a film bond stress, the stress produces a slow, viscous intergranular movement. If the entire stresses in a mass of clay are carried by the solid bond the clay is in a *solid state.* This state is preceded by the *lubricated state* in which part of the stresses are carried by the film bond. As the solid state is approached, the *degree of lubrication* of the clay decreases.

The process of solidification involves the lateral displacement of adsorbed water from the zones of potential solid bond between the soil particles towards the intergranular voids. The rate at which solidification at a given load per unit of area proceeds depends undoubtedly on the quantity of highly viscous, adsorbed water which needs to be displaced in order to establish the solid bond and on the pressure per unit of area of the adsorbed films. If we make an approximately plane horizontal section through a loaded layer of sand in such a manner that it does not intersect any grain, practically the entire section will pass through normal water and the pressure produced by the load per unit of area of the section through the adsorbed water will be extremely high. On the other hand, if we make a similar section through a layer of clay which carries the same load per unit of area as the sand, the major part of the section will be located within adsorbed water and the pressure due to the load per unit of area of the section through the adsorbed water will be relatively low. Therefore one should expect, that the time required for a given pressure to establish a given degree of solidification should for a clay be very much longer than for a sand.

The influence of grain size on the rate of solidification may account for the known fact, that a powder consisting of scale-like mineral particles such as mica or hematite flakes cannot be transformed into a plastic substance by mixing it with water, unless the

size of the particles is smaller than about 0.002 mm. A coarser powder solidifies during the test, while the finer powder remains in a lubricated state.

The same influence may also account for the well-known statistical relation between grain size and the angle of shearing resistance. In a general way, the smaller the effective grain size the smaller is the angle of shearing resistance determined by means of slow laboratory tests, such as slow shear tests or slow triaxial compression tests. In a slow test the neutral stress in the clay at the instant of failure is practically equal to zero. However, the degree of solidification in a highly colloidal clay is likely to be far less advanced at the instant of failure than in a leaner clay. If this is the case the increase of the shearing resistance due to a given increase of the normal stress on the surface of sliding should decrease with decreasing grain size.

As the state of failure in a shear test on clay is approached, the presence of films of liquid water between the clay particles should cause the clay to flow at constant stress like a viscous liquid. This conclusion has been repeatedly confirmed by experiment. (Terzaghi, 1931, 1932; Hvorslev, 1936).

Consolidation Due to Increasing the Load on a Normally Consolidated, Sedimentary Clay

Thus far our investigation has been of a purely theoretical nature. In order to make the step towards phenomena of practical importance we must analyze the effect of a transition from the slow process of loading applied by nature to the relatively rapid process associated with the construction of a building. In order to visualize the mechanical consequence of this transition we make one real and one imaginary consolidation test. Both tests are made on a remolded clay, the initial water content of which is equal to 45 per cent and equal to the liquid limit. In the real experiment the load is applied in equal increments Δp at the rate of 1 ton per sq. ft. per week and in the imaginary experiment the load is increased at a rate of 1 ton per sq. ft. per 500 years. The two tests will be referred to as rapid and slow tests respectively. Tests by Langer (1936) have shown that a process of loading by increments leads to practically the same pressure-void ratio curve as a continuous process of loading, provided the average rate of load application is the same in both tests. Therefore we are justified in assuming

that the compression curve which corresponds to the slow application of the load, is identical with the sedimentation compression curve C_s in Fig. 3*a*. The curve obtained from the rapid test is represented by the curve C_r.

In the rapid test an increase of the pressure from p to $p+\Delta p$ reduces the void ratio by Δe_1 and in the slow test by Δe_{s1}, which is very much smaller than Δe_1. The probable reason for this difference has already been explained. During the slow test every load increment acts on the clay for a period of many decades. As a consequence, contact is established between the solid portions of the adsorbed layers and the clay assumes the characteristics of a cemented material like concrete. This bond represents a true cohesion. It accounts for the high modulus of elasticity of slowly sedimented clays and for the absence of consolidation under very low excess pressures.

After the load on the specimen represented by the curve C_s in Fig. 3*a* becomes equal to p_0 the rate of loading in this test is changed from 1 ton per sq. ft. in 500 years to 1 ton per sq. ft. in one week. When the first increment in excess of p_0 is applied the clay cannot possibly know whether this increment will be followed by another one in a day or after 20 years. Hence the decrease Δe_{s2} caused by this increment cannot be greater than the decrease determined by the slope of the curve C_s at point P. The next increment which is added to $p_0+\Delta p$ after a day or two causes a heavier taxation of the strength of the clay. Yet the bond which has been built up during centuries of intimate contact between the clay particles cannot be expected to break down abruptly. Hence the effect of the second load increment will be intermediate between that for a perfectly solidified clay, determined by the slope of the curve C_s and that for a lubricated clay, determined by the slope of C_r. Finally after several load increments have followed each other in relatively rather rapid succession, all the clay particles will be separated from each other by films of liquid adsorbed water whereupon the clay particles are able to slide along each other with the same ease as those in the clay subjected to a rapid test. Hence the pressure-void ratio curve representing the imaginary test should ultimately become at least as steep as the curve C_r. Thus we arrive at the following conclusion. When changing the rate of loading at the state represented by point P in Fig. 3*a* from very slow to rapid the slope of the corresponding void ratio-

pressure curve C should consist of two distinctly different parts. The lower one, corresponding to the higher pressure, should have the same general trend as the curve C_r and the upper one should represent a gradual transition from C_s into C as shown in the figure.

In reality the first part of our imaginary experiment indicated by the curve C_s takes place during the deposition of a bed of clay. Point P represents the clay in its natural state. The subsequent rapid process of loading is performed by man, when constructing a building above the bed of clay. In order to get information regarding the decrease of the void ratio of the clay due to the weight of the building we proceed in the following manner. We first remove the clay from its original location whereby we reduce the load on the clay from p_o to a small value. Then we transfer the specimen into a consolidation device and increase the load on the clay again from zero to p_o. During this process the void ratio of the clay decreases by several per cent. Hence, the void ratio-pressure curve C_u which we obtain while increasing the pressure to values of more than p_o cannot possibly be identical with the curve C. It must be located at a lower level as shown in Fig. 3*a*. Hence, no matter how careful the sample has been secured and how conscientious the test in the laboratory has been made, the test result does not inform us on the character of the transition between the sedimentation consolidation curve C_s and the surcharge consolidation curve C, nor does it permit any definite conclusion regarding the position of the curve C with reference to the theoretical curve C_v. The plain line in Fig. 3*a* merely represents the result of a guess. The curve may as well occupy the position indicated by the dotted line.

As a final step in our investigation let us assume that a remolded clay has been consolidated rapidly to the state corresponding to P_r on the curve C_r in Fig. 3*a*. Then we leave the clay under the pressure p_o for a period of say 500 years, whereupon we resume the process of loading at the original rate of 1 ton per sq. ft. per week. During the intermission the contact between the clay particles changes from the state illustrated by Fig. 5*a* into that shown in Fig. 5*b*. Furthermore during the same period the bond between the adsorbed layers is strengthened by molecular adjustments within the zone of contact, known as thixotropic processes. Hence, if we continue our experiment at the original rate of loading, the initial slope of the following section of the pressure-void ratio curve should be as small as that of

the tangent to the curve C_s at point P. The trend of the curve cannot become identical with that of curve C_s until the clay has passed from the solid into the lubricated state. Thus it appears most likely that the pressure-void ratio curve will be similar to C' in Fig. 3*a* and to L' in Fig. 3*b*.

Secondary Time Effect

After the solid bond between the clay particles has been destroyed, for instance, by a load in excess of the critical load, the clay particles are temporarily separated from each other by films of very viscous adsorbed water. Hence the clay does not pass again into the solid state until the solid bond between the clay particles, illustrated by Fig. 5*b*, is re-established. In this connection we must consider two possibilities. Either the solidification of the clay occurs simultaneously with the consolidation or else the solidification continues after the consolidation is complete. In the first case the time-settlement curve should be in agreement with the theory of consolidation while in the second case the settlement due to consolidation should be followed by a supplementary settlement which cannot be accounted for by a process of consolidation. Both field and laboratory experience demonstrates that we have to deal almost exclusively with the second case. Figs. 6*a* and *b* show the relationship between per-

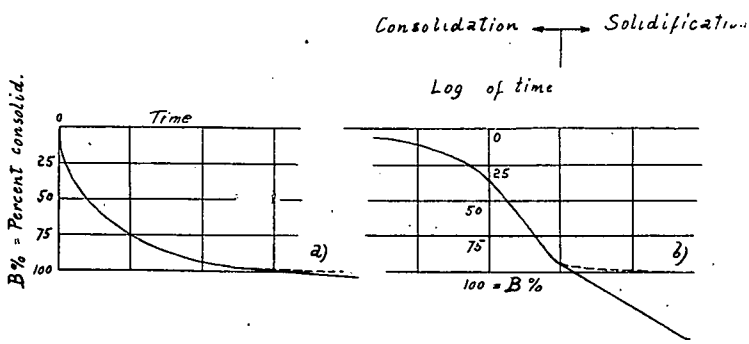

Figure 6.

centage consolidation and time for a given load increment which is applied in performing a consolidation test on clay. In Fig. 6*a* the relationship is shown on an arithmetic diagram and in Fig. 6*b* on a semi-logarithmic diagram. In both diagrams the plain curve repre-

sents the test results and the dashed curve, that part of the corresponding theoretical time-consolidation curve which does not coincide with the test curve. The difference between the ordinates of the dashed and the plain curves indicates the decrease of the void ratio associated with the process of solidification. As long as consolidation proceeds, part of the load on the clay is carried by an excess hydrostatic pressure in the water and the balance chiefly by film-bond stresses, because the relative movement between the clay particles maintains a state of lubrication. After the excess hydrostatic pressure in the water has become practically equal to zero the film bond is gradually replaced by the grain bond. This is the process of solidification which follows the process of consolidation. During the process of consolidation the effective stresses increase at the expense of the neutral stresses until the neutral stresses become equal to zero. During the period of solidification the grain-bond stresses increase at the expense of the film-bond stresses, until the film-bond stresses disappear. As long as part of the load is carried by film-bond stresses, the decrease of the void ratio of the clay continues. The settlement associated with this decrease is called secondary compression.

Thus far the only attempt to formulate a theory of the secondary compression has been made by Taylor and Merchant (1940). The theory has been worked out for the purpose of predicting the secondary time effect in the field on the basis of data obtained from consolidation tests in the laboratory. The theory is based on assuming that the rate of secondary compression follows a simple law, resembling that for viscous flow or for creep. In reality this law is likely to be very complicated, for two reasons. First of all, the viscosity of the liquid part of the adsorbed layers decreases with increasing distance from the surface of the solid particles. Second, as the rate of slippage at the points of contact decreases the resistance due to the viscosity of the adsorbed films combines with a supplementary resistance due to thixotropic stiffening, resulting from the building up of molecular bonds within the adsorbed layers. Hence, before accepting the theory for practical usage it would be advisable to investigate the importance of the error due to the basic assumption by comparing the computed and the real secondary time effect in several typical cases. This could even be done for existing buildings for which reliable settlement records are available.

Settlement under Small Loads

Whatever the position of the real compression curve C in Fig. 3 with reference to the theoretical compression curve C_v may be, the percentage difference between the amount of compression determined by these two curves rapidly decreases with increasing values of p-p_o. This may account for the satisfactory agreement between the computed and the measured amount of settlement of heavy structures, covering large areas. On the other hand, if the increase of the pressure on a bed of clay due to the weight of a superimposed building is small, the settlement is likely to be considerably smaller than what the computation based on the theoretical curve C_v in Fig. 3 would lead to expect. However, quantitative information regarding this difference could only be obtained by combining laboratory tests with settlement observations on light buildings and the available data do not yet permit any definite conclusions.

With increasing depth beneath the base of a building, the excess pressure produced by the weight of the building on horizontal sections decreases with increasing depth. At a certain depth the excess pressure enters the range within which the slope of the real compression curve C, Fig. 3*a* is insignificant, while that of the theoretical curve C_v is steep. Hence, below this depth, the compression of the clay due to the weight of the building should be very much smaller than the compression computed on the basis of the curve C_v. Since the shape and the extent of the flat part of the curve C in Fig. 3*a* cannot be determined by laboratory tests, information regarding the depth to which consolidation proceeds can only be obtained by means of settlement observations on underground reference points. In this connection the following observation is of interest. At the author's request, three underground reference points were established at depth of 15, 45 and 100 feet below the level of the pile points of the Charity Hospital in New Orleans. The subsoil consists of a succession of strata of sand and of normally consolidated clay which has been explored by test borings to a depth of 150 feet below the pile points. The natural water content of the clay did not decrease with depth and according to the customary method of computing the settlement due to the consolidation of the beds of clay all the underground reference points should have moved down. Yet, the lowest reference point did not move at all and the intermediate reference point descended much less than it should have, according to theory.

At the present state of our knowledge, the estimate of the depth limit to the compression of clay due to the weight of superimposed buildings can only be based on judgment, which may be very misleading.

In many cases the construction of a building is preceded by the excavation for subbasements. The amount by which this operation reduces the pressure on the beds of clay located beneath the site decreases with increasing vertical distance between the clay and the bottom of the excavation. Fig. 7 illustrates the author's conception

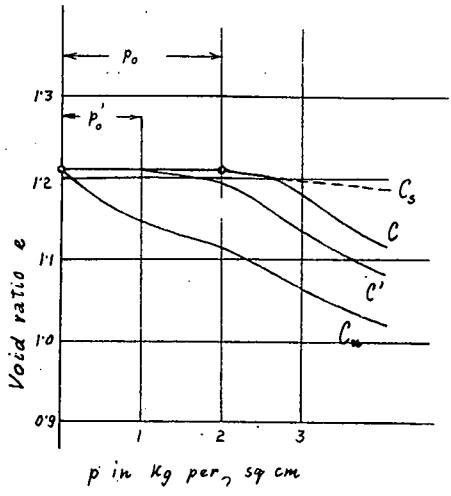

FIGURE 7.

of the effect of a temporary reduction of the pressure on the compressibility of a bed of clay. In this figure the curve C_u represents the results of a consolidation test on an undisturbed sample. Curve C shows the relation between a steadily increasing pressure such as the pressure produced by the construction of a building without subbasements and the corresponding void ratio. The curve C' represents the same relation on the assumption that the application of the pressure has been preceded by a temporary reduction of the pressure to a value $p'_o < p_o$. Since neither of the curves C and C' can be obtained by means of laboratory tests, the conception expressed by these curves can only be confirmed or modified on the basis of field experience.

Conclusions

The preceding analysis has led to the following tentative conclusions:

(1) If the state of stress in a clay has remained constant at least for several decades, or if the state of stress changes at a very low rate, the effective pressure on the clay is transmitted from grain to grain exclusively by a bond between the solid part of the adsorbed films which surround the clay particles. As long as this solid bond is intact the clay is in a solid state. In this state it has the physical characteristics of a porous, cemented material such as a typical loess. The modulus of elasticity is high and its volume compressibility is smaller than that of a dense sand.

(2) When a bed of clay in a solid state is loaded beyond a certain limit, for instance by constructing a heavy building above the clay, the rigid bond between the clay particles breaks and the particles become temporarily separated from each other by liquid films of adsorbed water. This is the lubricated state of the clay. In this state the clay is highly compressible. The increase of the load on a clay in a lubricated state is first of all followed by a period of consolidation the length of which depends both on the physical properties of the clay and on the thickness of the bed of clay. During this period the building located above the clay undergoes settlement due to consolidation. Yet, after the major part of the excess water has drained out of the clay, the clay is still in a lubricated state. Therefore the sliding of the clay particles along each other continues until finally the solid parts of the adsorbed layers come into contact with each other, whereupon the clay has again attained a solid state. The corresponding settlement of the structure is called settlement due to secondary compression. The relation between the time and the secondary compression per unit of depth depends on the grain size, on the chemical composition of the clay and of the adsorbed layers and on the length of the period of consolidation which preceded the period of secondary compression. However, in contrast to the rate of consolidation it is independent of the thickness of the bed of clay. The transition of a clay from the lubricated into the solid state may require years, decades or centuries, depending on the nature of the clay.

(3) After the consolidation of a clay is practically complete all the stresses in the clay are effective stresses. Yet these stresses

are partly carried by the highly viscous liquid films of adsorbed water which surround the clay particles and partly by a direct bond between the solid portions of the adsorbed layers. Therefore it is necessary to subdivide the effective stresses in the clay into two parts each having different characteristics: the film bond and the grain bond stress. A constant film bond stress is associated with a viscous flow which occurs at a constant rate while a constant grain bond stress does not produce any movement.

(4) During the process of consolidation the stresses in the clay consist mainly of neutral and of film bond stresses, while the grain bond stresses are negligible. During the subsequent period of solidification the neutral stresses are negligible while the grain bond stresses steadily increase at the expense of the film bond stresses. Finally the film bond stresses disappear, whereupon the clay is again in a solid state.

(5) The rate of settlement due to consolidation can be estimated by means of the theory of consolidation. This theory is based on the simplifying assumption that the entire effective stresses represent grain bond stresses. An attempt to establish a theory of the secondary compression associated with the process of solidification has been made by Taylor and Merchant. The data required for evaluating the error associated with this theory are not yet available.

(6) Prior to the beginning of construction operations practically every bed of clay in the field is in a solid state unless the structure of the clay has been disturbed by recent soil movements or by artificial changes in its state of stress. As the load on a bed of clay due to an artificial surcharge increases, the clay gradually passes from the solid into the lubricated state, whereby the permanent compression produced by a given increase of the pressure first increases and then decreases.

(7) During the sampling operations every clay passes from the solid into a partially lubricated state. Hence, information regarding the physical properties of clays in a solid state can only be obtained by means of field observations, for instance by measuring the settlement of structures the weight of which increases the pressure on underlying beds of clay by not more than a fraction of the unconfined compressive strength of the clay, or by observing the movement of underground reference points beneath the foundation of heavy build-

ings and by comparing the results of the observations with those of the computed settlements. Up to this time the available data do not yet justify expressing a definite opinion regarding the relation between pressure and compression within the range of pressure required for transferring the clay from the solid into the lubricated state.

Acknowledgment

The author wishes to express his gratitude to Professor A. Casagrande for careful perusal of the manuscript and for valuable suggestions.

References

Casagrande, A., (1932). The Structure of Clay and Its Importance in Foundation Engineering. Journ. Boston Soc. C. E., Vol. XIX, No. 4.

Fellenius, B., (1936). Proc. Int. Comp. Soil Mech. Cambridge, Mass. Vol. III.

Hvorslev, M. J., (1939). Torsion Shear Tests and Their Place in the Determination of the Shearing Resistance of Soils. Proc. Am. Soc. T. M., Vol. 39.

Langer, K., (1936). "The Influence of the Speed of Loading Increment on the Pressure-void Ratio Diagram of Undisturbed Soil Samples." Proc. Int. Comp. Soil Mech. Cambridge, Mass. Vol. II.

Rutledge, Ph. C., (1939). Compression Characteristics of Clays and Application to Settlement Analysis. Doctor's thesis, Harvard University.

Taylor, D. W. and Merchant W., (1940). A Theory of Clay Consolidation Accounting for Secondary Compression. Journ. Math. Phys., Vol. XIX, No. 3.

Terzaghi, K., (1925). Erdbaumechanik.

Terzaghi, K., (1926). The Mechanics of Adsorbtion and the Swelling of Gels. Fourth Colloid Symposium Monograph. Chemical Catalogue Co., Inc., New York.

Terzaghi, K., (1931). The Static Rigidity of Clays. Journal of Rheology, Vol. 2, No. 3.

Terzaghi, K., (1932). Tragfähigkeit der Flachgründungen. First Congress Int. Assoc. for Bridge and Structural Eng. Paris.

Terzaghi, K., and Fröhlich, O., (1936). Theorie der Setzung von Tonschichten. Vienna.

THE FLOOD HYDROGRAPH

By Howard M. Turner* and Allen J. Burdoin,** Members

(Presented at a meeting of the Hydraulics Section of the Boston Society of Civil Engineers held on May 7, 1941)

A part of the material of this paper was contributed to the Committee on Floods and appears as the report of the Sub-Committee on the Flood Hydrograph, in the preliminary Flood Report presented to the Society at the annual meeting March 19, 1941.

The flood hydrograph should be of special interest to the members of this Society, for its significance in the analysis and prediction of flood runoff and peak flows was first noted in the report of this Society's Committee on Floods, September 1930. This called attention to the fact that flood hydrographs at any point on a stream for storms within the "concentration period" have substantially the same base length regardless of the amount of runoff, so that the flood peak varies generally with the total flood runoff. A flood characteristic curve or hydrograph on a unit basis was also developed. This was largely the work of the secretary of that committee, S. Stanley Kent, of Lowell. About two years later the method of the unit hydrograph[1] was published. This is the hydrograph of a flood with a runoff of 1 inch from a rainfall of unit length, generally one day. Its use has become one of the standard methods of analyzing flood flows. This paper describes a different method of analyzing the flood hydrograph.

The flood hydrograph (Fig. 1) is made up of a rising limb starting at the base flow at point "A", with an increasing slope up to point "B". From this point up to the crest, point "C", the flow increases but at a decreasing rate. From the crest the flow recedes, slowly at first, and then more rapidly until the time when no further flow is contributed by the rainfall at point "D". From this point, which is a point of inflection in the curve, the hydrograph consists of a recession or drainage curve with a decreasing slope which, as

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[1]Stream Flow from Rainfall by the Unit-Graph Method, L. K. Sherman, Engineering News-Record, April 7, 1932, p. 501.

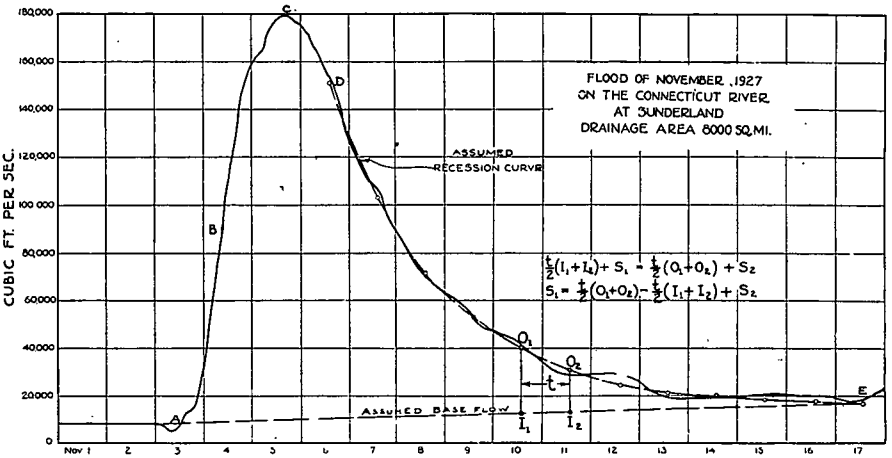

FIG. 1.—FLOOD OF NOVEMBER, 1927, ON THE CONNECTICUT RIVER AT SUNDERLAND.

has previously been described,[2] is merely draining off of the flood water held in storage in the area.

RECESSION AND STORAGE CURVES

Since this recession or drainage curve beyond the point of inflection, "D", is the outflow from the flood storage in the whole river system without any flow increment from the storm rainfall, for any given point on a stream it should be a similar curve for all floods. It is also possible to use this curve to derive a curve of volume of total storage vs. flow at that point.[3] It will generally be found, it is believed, that, except in some cases in the lower stages, this curve will be approximately a straight line, as has been found to be the case generally in the storage in a given reach of a river channel.

This storage curve has been figured for the Connecticut River at Sunderland. The curve on Fig. 1 shows the 1927 flood hydrograph and the recession curve and base flow assumed. Based on this, the "storage curve", i.e., the curve of volume of total storage vs. river

[2]Surface Runoff Phenomena, Part I—Analysis of the Hydrograph by Robert E. Horton. Horton Hydrological Laboratory, Voorheesville, N. Y., Publication 101, Feb. 1, 1935.

Since this paper was presented the authors have had occasion to reread the above publication of Mr. Horton's in which storage curves were derived from the recession curve of the hydrograph and applied to it to correct it for the effect of "channel storage." This reference should have been given at the time the paper was presented.

[3]Supra, note 2.

flow or outflow has been computed. This curve was computed, as were all the curves following, by the general storage or routing formula:

$$\frac{t}{2}(I_1 + I_2) + S_1 = \frac{t}{2}(O_1 + O_2) + S_2$$

Where I_1 and I_2 = inflow, cu. ft. per sec., at the beginning and end of each interval of t seconds

O_1 and O_2 = outflow, cu. ft. per sec., at the beginning and end of each interval of t seconds

S_1 and S_2 = volume of storage in cu. ft. corresponding to outflow O_1 and O_2 respectively.

The 1927 storage curve was figured starting at point "E", the end of the flood runoff at its intersection with the assumed base flow, taking the base flow as the "inflow", the flood flow as the "outflow" and solving for the storage volume. The resulting storage curve is shown on Fig. 2. This curve was figured starting at 17,000 c.f.s. the

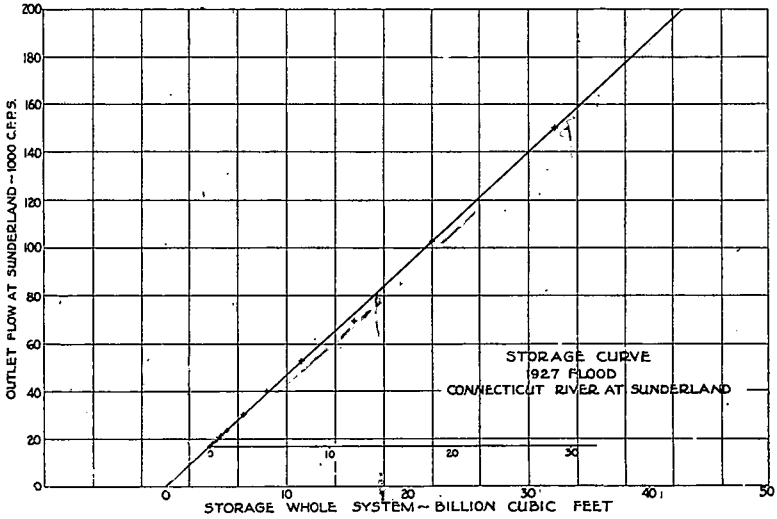

FIG. 2.—STORAGE CURVE, 1927 FLOOD, CONNECTICUT RIVER AT SUNDERLAND.

base flow at the end of the flood. It was extended down to 0 in a straight line as shown.

It is interesting to note how the total storage shown on this curve compares with the measured storage on the Connecticut River. The

U. S. Army Engineers have measured the storage in the main river channel during the 1936 flood from Waterford, Vt., to Sunderland, Mass., a stretch of the lower 60% of the total length of the main river, and give the total as 36.3 billion cu. ft. The 1936 flood at Sunderland was 238,000 c.f.s. which, from the storage curve of Fig. 2, would give a total storage volume of 54 billion cu. ft. The storage in the lower part of the main river valley apparently contributes the greater part of the total storage in the whole river system.

This 1927 storage curve for the Connecticut River at Sunderland has been tested out on flood hydrographs of other floods as follows: the base flow was assumed as a straight line drawn between the flow existing before the rise and an assumed point where the flow from the flood appears to have run out. The point of inflection, beginning the recession curve, was picked out by eye. Starting at the point of inflection, taking the base flow as "inflow", the "outflow" was computed using the storage curve of Fig. 2. These figured curves compared to the actual hydrographs are shown for the floods of 1913, 1933, 1936, and 1938 on Fig. 3. In only one case are these curves the results of more than one trial.

In two of these, the 1936 flood and to a lesser degree the 1933 flood, additional rain after the peak of the flood produced a rise which clearly appears on the hydrograph. In general, however, these curves present a fit that is as close as could be expected when the difference in the character of floods shown is considered. This is particularly interesting when it is considered that this storage curve must represent not only the storage in the main river valley but over the whole system including that in the main river, the tributaries, the small brooks and even the storage accumulated on the ground which can drain off over the surface.

The Flood Hydrograph

The flood hydrograph at any point in a stream is the outflow from the river above and should be the "inflow" from the storm runoff on the drainage area above as modified by the storage in the whole system. It will thus be affected by (1) the shape of the drainage area, (2) the length of time that the storm rainfall lasts, (3) the length of time it requires for the runoff from this rainfall from different parts of the watershed to reach the point of measurement, and (4) the

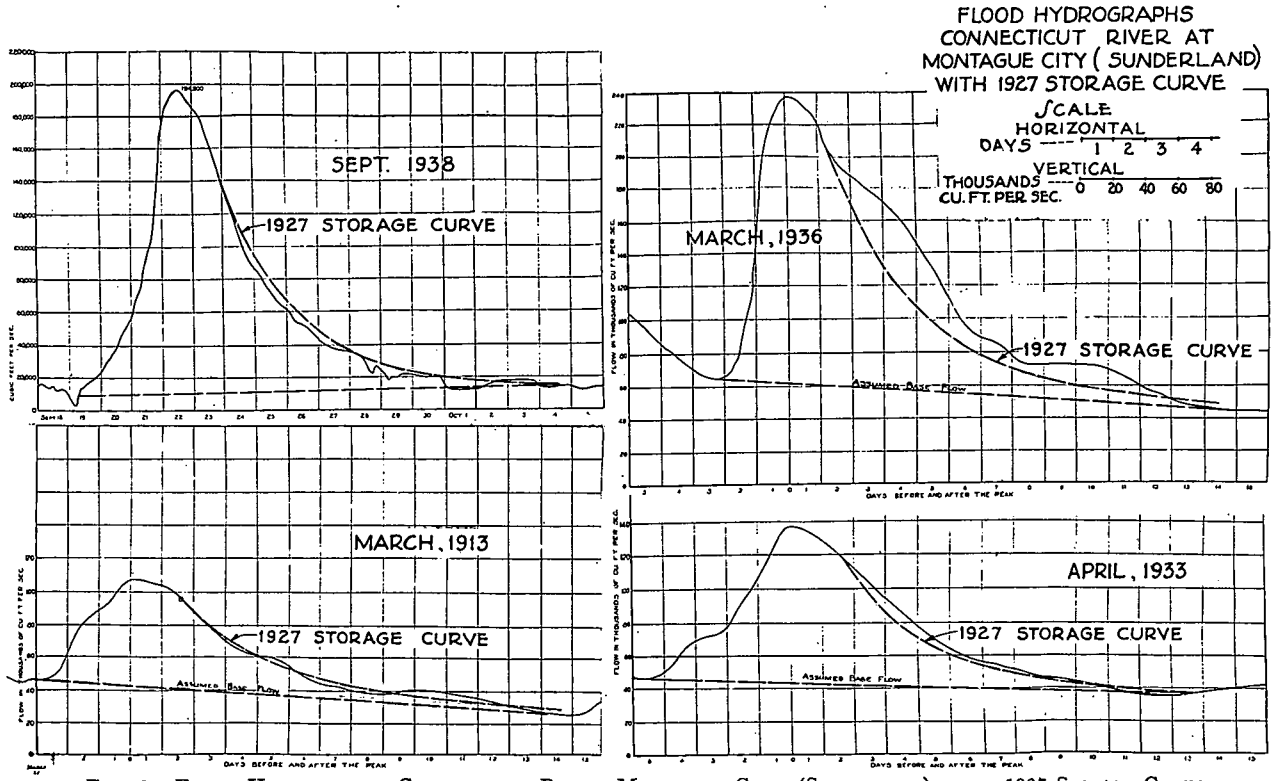

FIG. 3.—FLOOD HYDROGRAPHS, CONNECTICUT RIVER, MONTAGUE CITY (SUNDERLAND) WITH 1927 STORAGE CURVE.

storage in the river system. A simple method of analysis of the flood hydrograph reflecting all these elements is described below.

This analysis is based on flood runoff, not rainfall. The base flow is assumed as a straight line from the flow at the beginning of the flood rise, ending when the flood hydrograph approaches the horizontal. The flood runoff from a given storm is considered to be the runoff above this base flow.

The flood flow at the point of measurement, assuming no flood storage at all, will depend on the shape of the drainage area, the time of the rainfall t_o, and the length of time for water from the furthest point of the river to reach the point of measurement, here called the "concentration time" $\frac{L}{v}$. This "concentration time" is not necessarily the same as the time from the beginning of the flood to the time of the peak, as has sometimes been assumed as the "concentration period" in previous analyses.

The hydrograph of the flow from the area such as would exist were there no storage is called the flood "inflow hydrograph". Fig. 4 shows the derivation of inflow hydrographs for a rectangular drainage area assuming uniform rainfall over the whole area for storm periods, t_o, less than, equal to, and greater than, the concentration time, $\frac{L}{v}$. Fig. 5 shows the same hydrographs for a triangular drainage area and for a diamond shaped area. The general shapes of these curves represent the hydrographs of the areas which would have existed at the point of measurement under the assumed conditions were there no storage. Excessive width may have a modifying effect on some of these shapes.

If the "storage curve" for the whole river system is known, it should be possible to determine the "outflow" hydrograph, i.e., the actual flood hydrograph at the point of measurement, by routing the inflow through the storage and thus determining the outflow. This has been done for the Connecticut River at Sunderland, considered as a narrow rectangular area, first taking the 1927 flood.

The rainfall at certain points on the Connecticut River is shown on Fig. 6 and appears to come at a rate fairly close to uniform during most of the period. Based on this curve, the period of rainfall, t_o,

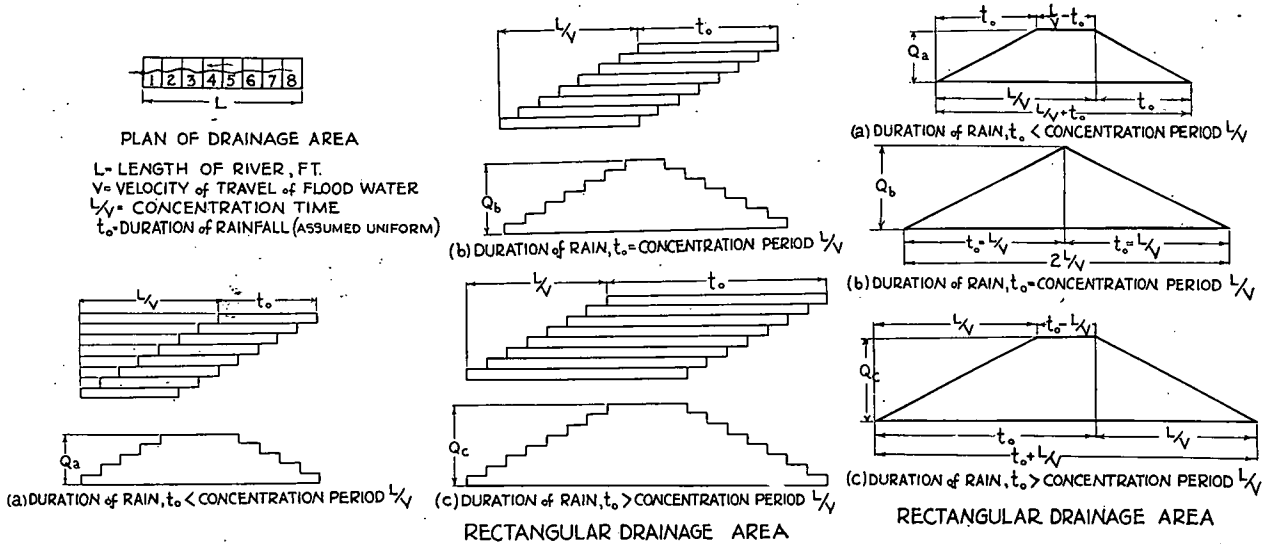

FIG. 4.—DIAGRAMMATIC FLOOD RUNOFF-INFLOW HYDROGRAPHS.

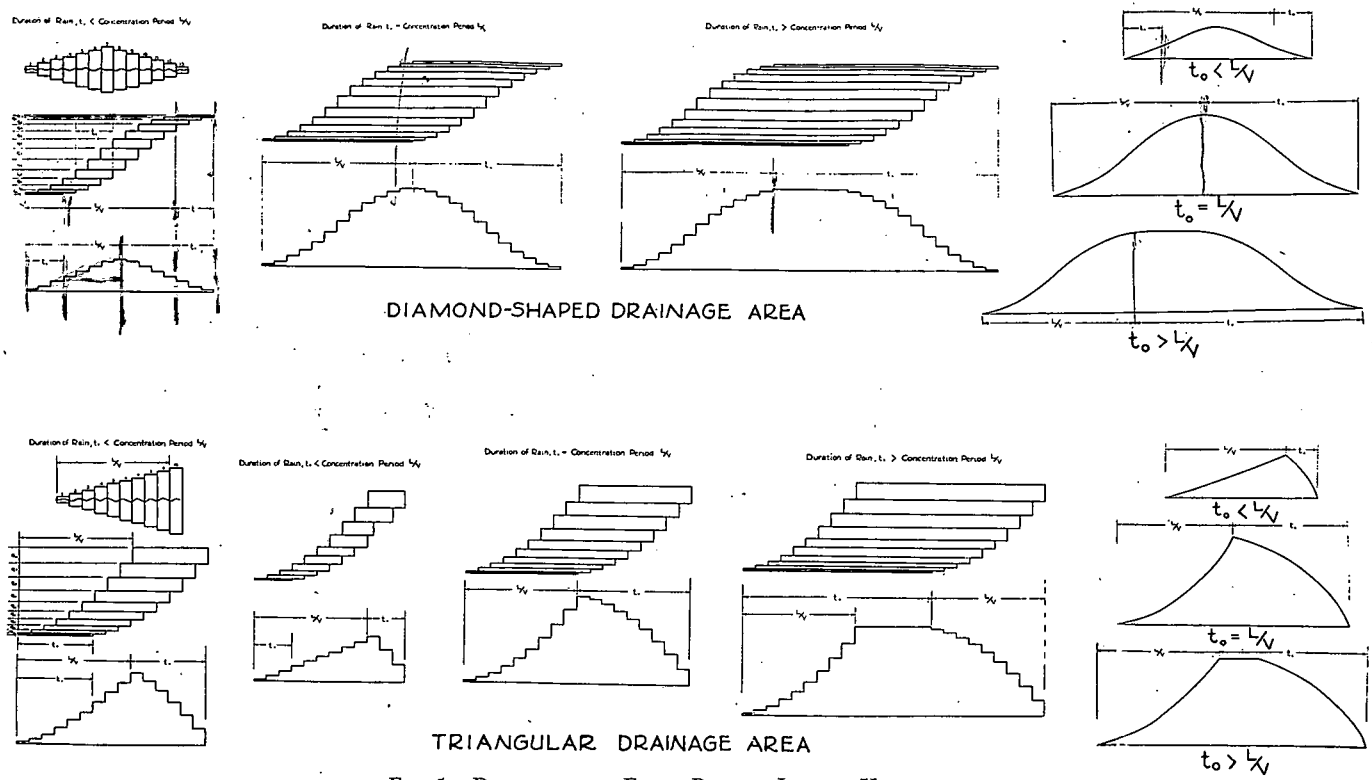

FIG. 5.—DIAGRAMMATIC FLOOD RUNOFF INFLOW HYDROGRAPHS.

has been taken as 30 hours. The concentration time, $\frac{L}{v}$, has been taken at 50 hours; t_o being less than $\frac{L}{v}$, the inflow hydrograph will be a trapezoid of the form shown on the figure. The area of this trapezoid represents the total flood runoff, taken as the flood runoff above the base flow, and in the computation it has been so figured. Using this inflow hydrograph and the storage curve previously determined, the inflow is routed through the storage to give the outflow, using the same general storage formula but in this case, starting at the beginning of the flood and solving for $\frac{t}{2} O_2 + S_2$ from which, knowing the storage relation, O_2 can be found. The outflow curve thus obtained is shown on Fig. 6 compared to the actual flood hydrograph.

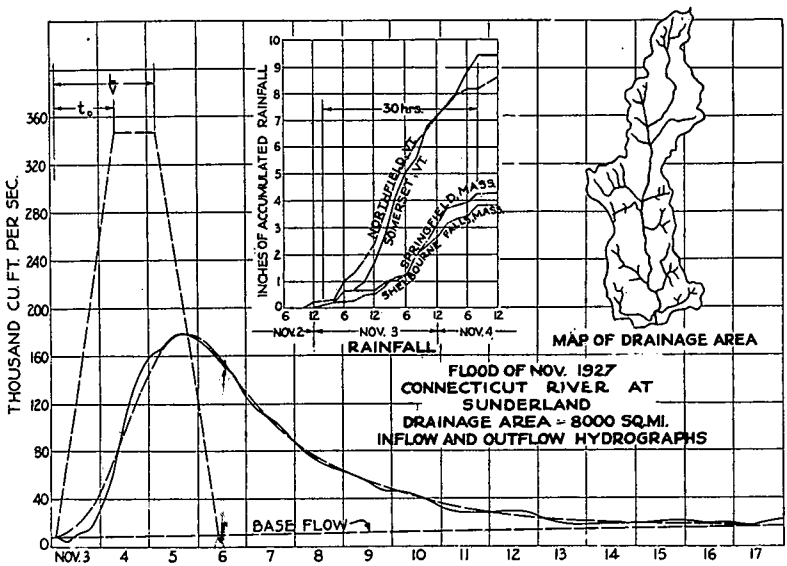

FIG. 6.—CONNECTICUT RIVER AT SUNDERLAND. FLOOD OF NOVEMBER, 1927.

For another example, where the length of storm, t_o, is greater than the time of concentration, the flood of 1913 was selected. The rainfall (including flood runoff from melting snow) was assumed to be uniform for three days, the actual length of the heavy rainfall.

The inflow hydrograph is as shown. The flood outflow was computed using the same storage curve and the same $\frac{L}{v}$.[4] Fig. 7 shows the result compared to the actual hydrograph.

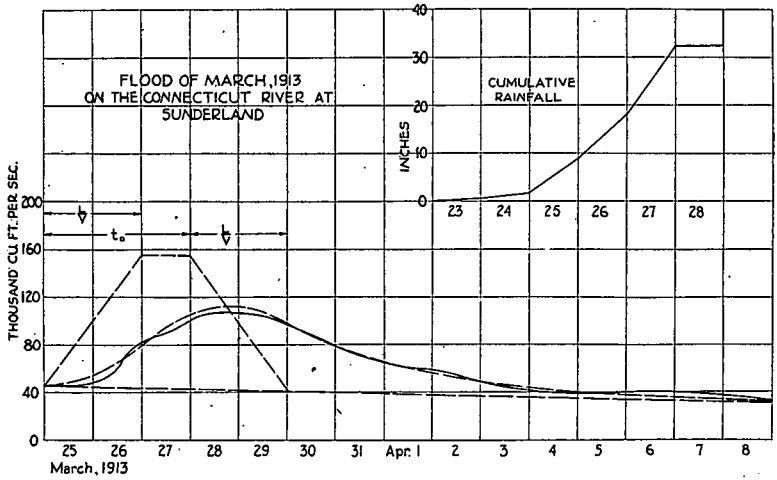

FIG. 7.—CONNECTICUT RIVER AT SUNDERLAND. FLOOD OF MARCH, 1913.

The Merrimack River at Lowell has been similarly analyzed. Fig. 8 shows the points on the 1938 flood hydrograph used to obtain

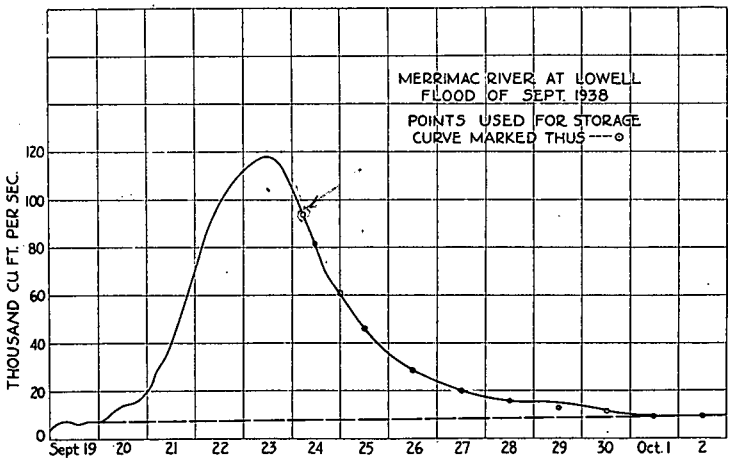

FIG. 8.—MERRIMACK RIVER AT LOWELL. FLOOD OF SEPTEMBER, 1938.

[4]Within 4% for ease of computation.

the storage curve which is shown on Fig. 9. In this case it is not a

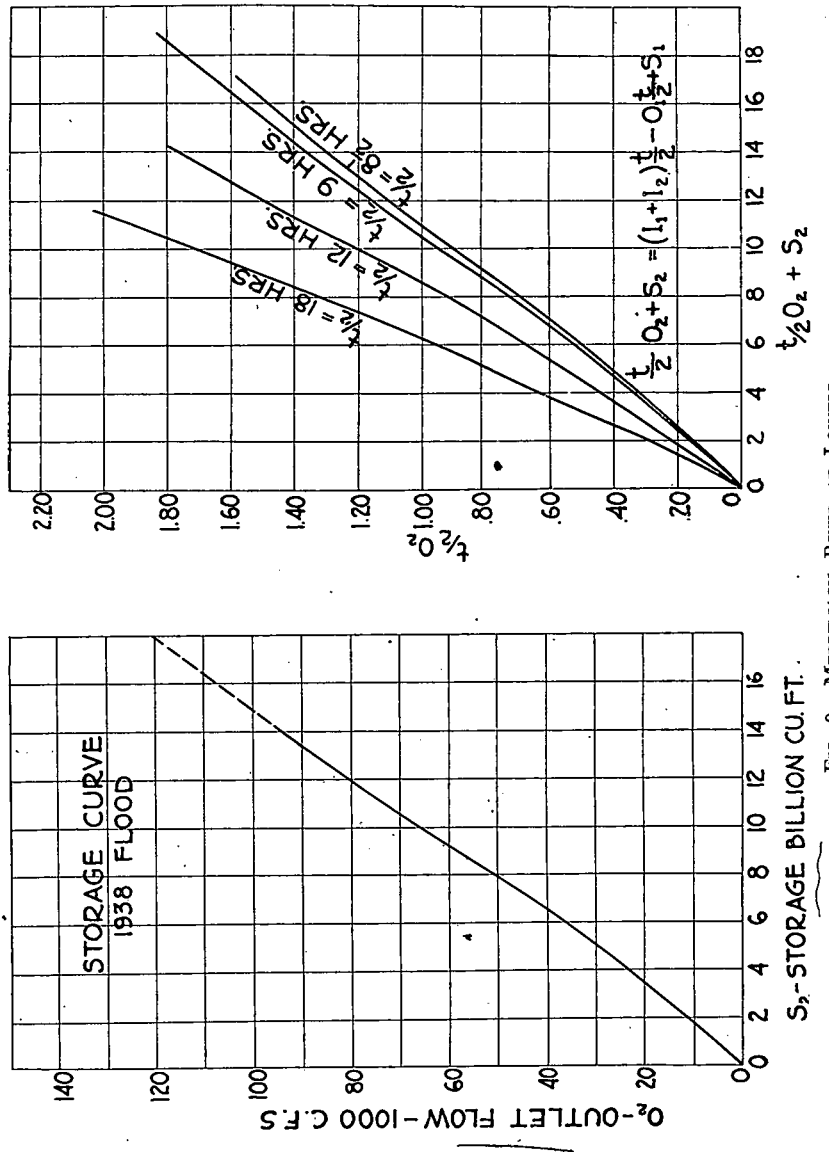

FIG. 9.—MERRIMACK RIVER AT LOWELL.

straight line in the lower stages. The figure also shows a set of curves used in computation of the outflow hydrograph. These are of assistance in the case where the storage curve is not a straight line to obtain $\frac{t}{2} O_2$ from $\frac{t}{2} O_2 + S_2$ obtained from the general storage equation.

Fig. 10 shows for the 1927 flood similar inflow and figured out-

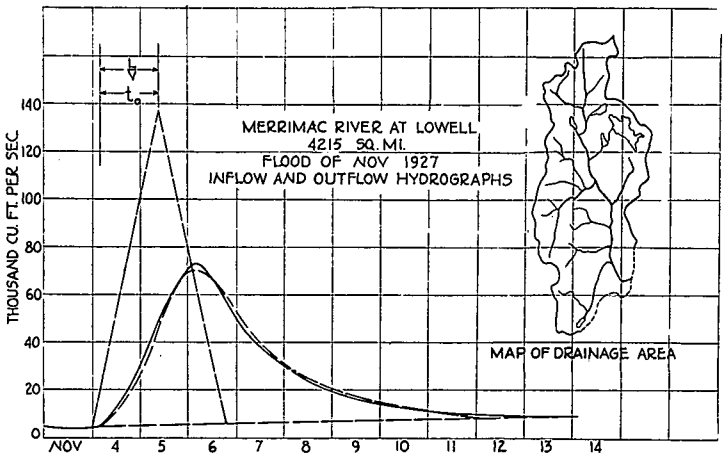

FIG. 10.—MERRIMACK RIVER AT LOWELL. FLOOD OF NOVEMBER, 1927.

flow hydrographs and the actual flood hydrograph. There were some further assumptions necessary in this case. The rainfall in the Merrimac River valley was apparently later than in the Connecticut River valley, particularly in the North part. This is indicated in the rainfall records which unfortunately are not of the recording type. The $\frac{L}{v}$ taken is somewhat less than for other floods.

Fig. 11 shows the same curves for the 1936 flood. In this case it was necessary to estimate what the flood hydrograph would have been without the rainfall which occurred after the peak. This was done by figuring a recession curve using the storage curve as shown. The hydrograph thus amended is what is used to obtain the area of the inflow hydrograph. The figured outflow hydrograph is plotted compared to the actual hydrograph amended. Fig. 12 shows the same hydrographs for the flood at Lowell of May, 1923.

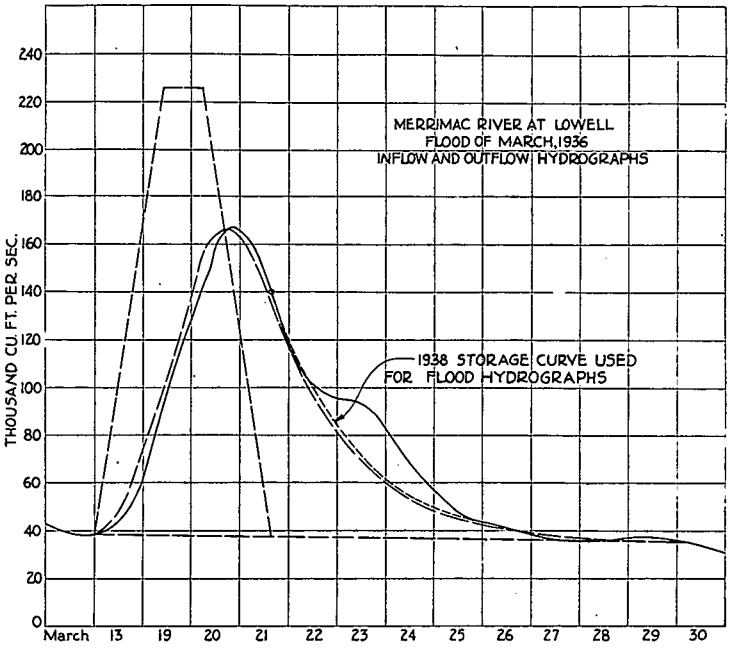

FIG. 11.—MERRIMACK RIVER AT LOWELL. FLOOD OF MARCH, 1936.

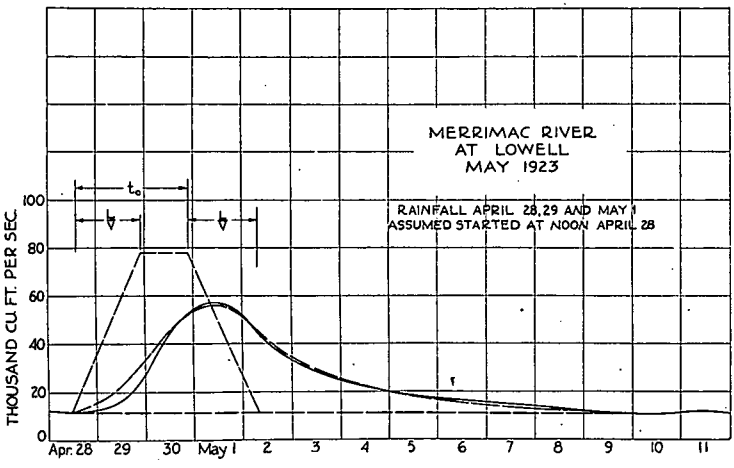

FIG. 12.—MERRIMACK RIVER AT LOWELL. FLOOD OF MAY, 1923.

When it is considered what assumptions have to be made, as to the shape of the drainage area, the uniformity of the rainfall, and that the storage of the whole river system is constant for every flood, the closeness with which these computations fit the actual hydrographs seems quite remarkable. It has been found that there may be several possible inflow hydrographs which will give an outflow hydrograph very close to the actual hydrograph. The reason for the results would seem to be the fact that the storage is such a large factor in the flood hydrograph, as will be shown later. It apparently smooths out the various differences and discrepancies to give similar shaped hydrographs for various different conditions.

In certain cases tried it has not been possible to secure a satisfactory fit between the computed and actual hydrographs. For example, in the case of the 1938 flood on the Merrimac River at Lowell the peak came later than it figured with this method of analysis. An inspection of the rainfall records on the recording gauges shows, however, that the storm was moving up the drainage area so that at the upper end it was about 12 hours later than at the lower. This should have the effect, reflected in the hydrograph, of delaying the peak beyond what would have been the case if the rainfall had been more uniform over the whole area during the length of the storm.

The trapezoidal inflow hydrograph assumed is undoubtedly an oversimplification. This may well be one of the reasons why it has generally been found that at the very start of the flood the figured hydrograph is somewhat outside the actual hydrograph. The result, however, seems to justify the use of such an inflow hydrograph as being sufficiently close for drainage areas which are nearly rectangular. Other shaped drainage areas have not yet been analyzed.

The treatment of the base flow may also be too simple. Unless it is the result of surface runoff from a previous flood or melting snow this base flow should represent the accumulated flow from the ground into the river and its tributaries above. At the start of a rainfall which produces additional surface runoff it is possible that the base flow may be reduced due to the rapid rise of water in the channel choking it off. As against this, there is more water going into the ground from the flood rainfall which may cause more water to flow out from the ground during the flood itself which might offset possible reduction along the river channel. Whatever the assumption

as to base flow, the storage curve and both the inflow and outflow hydrographs are all made on the same basis so that they are consistent, and the results seem to justify the simple assumption that the base flow is a straight line across the bottom of the flood hydrograph.

Certain information regarding the inflow hydrograph for any case can be obtained from the actual flood hydrograph. The point of inflection, that is the beginning of the recession or drainage curve must occur at the time the flood inflow ceases, that is, at the end of the inflow hydrograph. The peak of the flood must occur at a point where the inflow and outflow hydrographs intersect. A straight line drawn up through these two points will give the descending side of a triangular or trapezoidal inflow hydrograph. Knowing the flood runoff, i.e., the total area under the curve, from the actual hydrograph, and the time of rainfall, with an assumed $\frac{L}{v}$ the inflow hydrograph can be plotted for trial routing through the storage.

In all these cases, the method of procedure is as follows: The storage curve is first obtained from the recession curve of an actual flood hydrograph, using an assumed base flow determined from the hydrograph itself, and checked using the hydrographs of other floods. These storage curves extended to zero should be very similar for all the floods considered. The hydrograph analysis of any given flood is then undertaken as follows: The flood runoff above the base flow is measured, to find the total outflow runoff. The total inflow runoff must equal this. The total inflow hydrograph is developed taking into account the length of the rainfall, assumed uniform, the shape of the drainage area and the assumed time of concentration. This is then routed through the storage starting at the beginning of the flood. It is believed that with a certain number of trials, the inflow hydrographs for any stream, on which there are several flood hydrographs available, can be determined which combined with the storage curve will enable the flood hydrograph of that stream to be determined for any storm where the rate of rainfall can be assumed as uniform throughout its length.

It may be questioned why it is not better using the general storage equation and the storage curve to figure back from the actual outflow hydrograph to the inflow hydrograph instead of assuming, as is done

here, an inflow hydrograph and then computing the outflow hydrograph. It may be that this can be done but difficulty has been found in cases tried because small differences in slope of the actual hydrograph may throw off the result. This should be the subject of further study.

Effect of Varying Elements

This method of analysis permits assessing the effect on the shape of the flood hydrograph of the three elements that will determine it, i.e., the length of the storm, t_o, the amount of storage, and the time of concentration, $\frac{L}{v}$.

The effect of the length of the storm on the flood hydrograph is shown on Fig. 13. A rectangular drainage area is assumed of 5,000

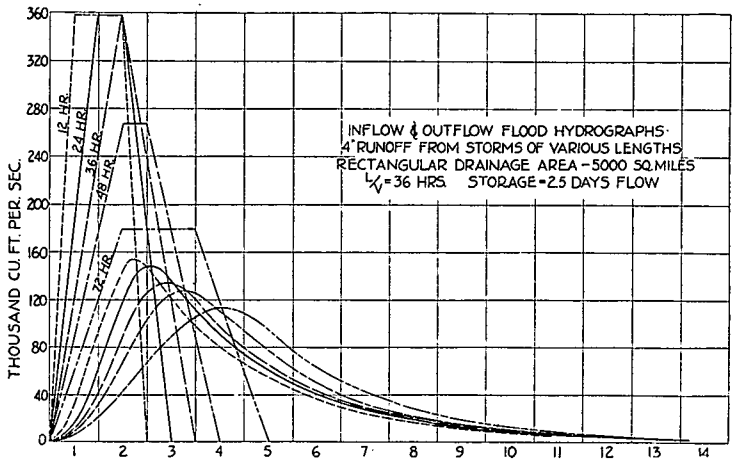

Fig. 13.—Inflow and Outflow Flood Hydrographs, Storms of Various Lengths.

square miles, with a runoff of 4″ above the base flow, a time of concentration of 36 hours, and a straight line storage curve with a storage volume for each flow of 2½ days times that flow. The inflow and outflow hydrographs are shown for storms of 12, 24, 36, 48, and 72 hours. It is interesting to note that even a storm lasting twice as long as the concentration time does not produce a flood hydrograph with a constant flat top such as has been assumed in some theories of the hy-

drograph. Fig. 14 shows a storm of 12 days duration for the same

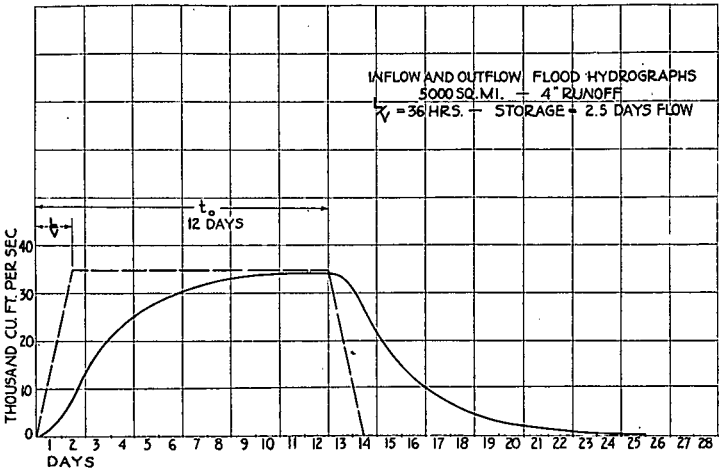

FIG. 14.—INFLOW AND OUTFLOW FLOOD HYDROGRAPHS, 12 DAY STORM.

drainage area. It will be noted that even with a storm as long as this, the outflow does not equal the inflow though it approaches it very closely. It can be shown that if the storage acts in the way assumed here, no matter what the length of the storm, the outflow will never reach the inflow though it will be asymptotic to it.

The amount of storage appears to be the most powerful factor in the shape of the flood hydrograph. Fig. 15 shows hydrographs for

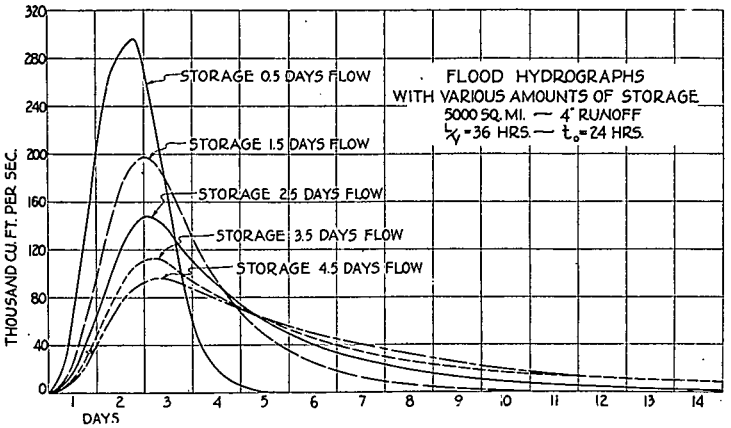

FIG. 15.—FLOOD HYDROGRAPHS WITH VARIOUS AMOUNTS OF STORAGE.

the same drainage area described above, assuming a time of rainfall of 24 hours and storage curves assumed as straight lines, with the amount of storage varying from one half to four and a half days flow. The peak in the case of the former figures about three times the latter.

The time of concentration, $\frac{L}{v}$, however, does not seem to be such an important factor. Fig. 16 shows the effect for the 24 hour storm

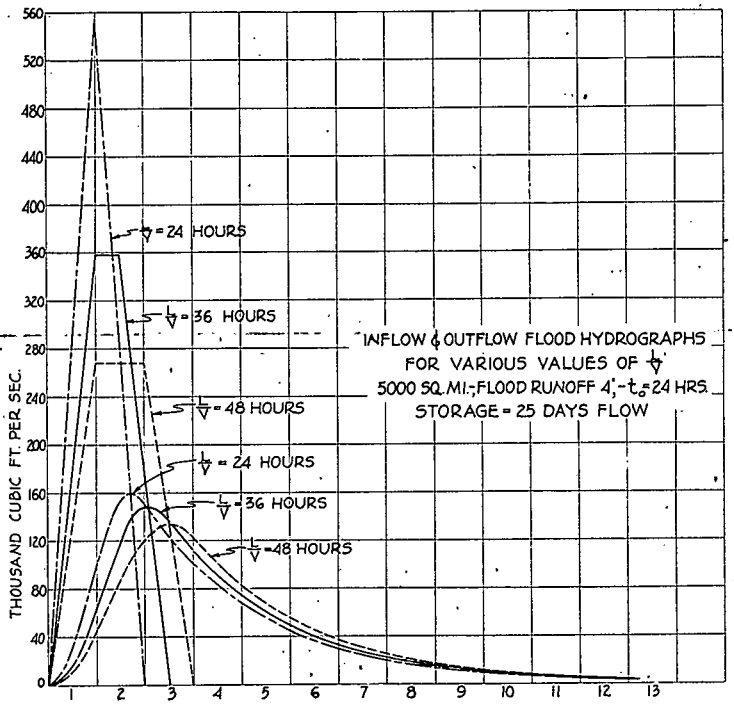

FIG. 16.—INFLOW AND OUTFLOW HYDROGRAPHS FOR VARIOUS VALUES OF $\frac{L}{v}$.

on the same drainage area of varying the time of concentration from 24 to 48 hours.

Comparison with Another Method

General equations for the flood hydrograph have been derived by R. T. Zoch in three papers published in the *Monthly Weather Review*.[5] Since these equations, with certain changes in application and the insertion of proper constants, give results similar to the analysis described above, it is interesting to compare them.

Mr. Zoch made the following assumptions to simplify the mathematics of the problem: All the rainfall appears as runoff during the storm period or, as he expresses it, "there is no evaporation"; the rain falls at a constant rate over the whole area; the soil conditions (or temporary retention) are uniform; there is no snow cover; the drainage area is rectangular; the stream velocity is constant; the base flow is unaffected by the storm flow and is to be added to the latter. All his equations were also based on the fundamental assumption that "The rate of runoff at any given time is proportional to the rainfall remaining with the soil at that time." In other words, the amount withheld from the rainfall to give the surface runoff including the amount held in storage is proportional to the discharge.

The fundamental equation[6] is: Volume remaining = Total rainfall — total runoff volume or, in mathematical symbols,

$$cR = \int_0^t Ai\,dt - \int_0^t R\,dt$$

Where i = rainfall rate—inches per hour
R = the runoff rate—inches on the drainage area per hour
t = time in hours
A = drainage area
c = a constant, expressed in hours, covering the proportion that the amount withheld from running off bears to the runoff; in other words the amount withheld at any time, "t", is equal to c hours of flow at the runoff rate R.

Based on these assumptions, the runoff from a small area was first analyzed. The runoff from such an area will increase as the

[5]On the Relation between Rainfall and Stream Flow, I, II and III, by R. T. Zoch, The Monthly Weather Review, I, Vol. 62, Sept. 1934; II, Vol. 64, April, 1936; III, Vol. 65, April, 1937.

[6]The notation used is slightly different from that used by Mr. Zoch; it is believed to be more in accord with the usual notation used by hydraulic engineers.

quantity of rain retained as infiltration and storage increases until the runoff will practically equal the rainfall if the rain lasts long enough. After the rain stops the accumulated water in storage must drain away. A flood hydrograph for a small area, plotted from these equations, is shown on Fig. 17. The equations given were obtained

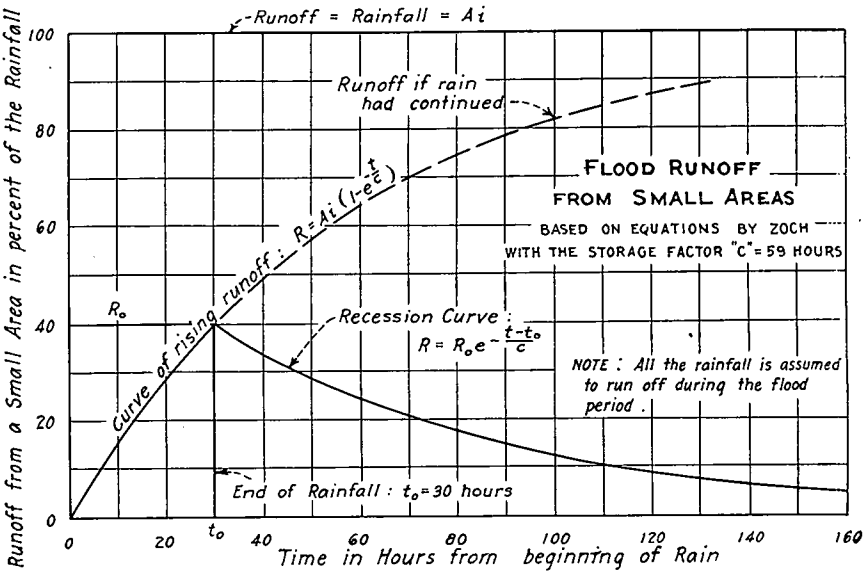

FIG. 17.—FLOOD RUNOFF FROM SMALL AREAS.

by solution of the fundamental equation given above. The area is assumed small enough so that water from all parts of the area arrives at the outlet simultaneously without time lag.

He then considered a drainage area to be made up of an infinite number of such small areas, each having a flood hydrograph as shown in Fig. 17. These hydrographs will then follow each other downstream, so that for a rectangular area of length "L", width "W", and velocity "v", the flood hydrograph for a gaging station at the outlet will be made up of a succession of the above hydrographs superimposed on each other.

The results obtained from his equations may be compared to the analysis developed by the authors, by considering that the rainfall, "i", corresponds to the authors' inflow (runoff), the runoff "R" representing the actual flood flow in both cases. Mr. Zoch's "c" then becomes a

factor covering storage only and, being assumed constant, corresponds to the authors' storage curve, when a straight line. This has been done in the computation of a hydrograph above the base flow of the Connecticut River at Sunderland shown on Fig. 18. In this the area has been assumed rectangular and divided for purposes of illustration into five equal areas. The velocity of flow is assumed uniform so that the abscissa represents time of flow in hours for the water to reach the gaging station, as well as distance. The individual flood hydrographs for each area are shown starting at a time equal to the time required for water from the center of each small area to reach the outlet. Since the abscissa represents both time and distance, this brings the beginning of each hydrograph under the center of the area producing it. The flood hydrograph of the river at the gaging station will be the sum of the individual flood hydrographs from the small areas. These hydrographs have been summed up mathematically as done by Mr. Zoch, assuming the total area divided into an infinite number of infinitesimal contributing areas. The resultant hydrograph has been plotted on Fig. 18.[7]

The resulting hydrograph consists of four parts, each with its characteristic equation as shown, except in the special case where the length of rain equals the concentration period, in which case there are only three parts. Each of the first three parts corresponds to one of the straight lines of the trapezoid of "flood inflow" developed by the authors, and the last part consists of the recession curve representing depletion of storage following the end of the inflow.

In case the rain lasts longer than the concentration period, the hydrograph still consists of four parts, but a different equation must be used for the second part. This equation is:

$$Q = Wi \left[L - cve^{\frac{-t}{c}} \left(e^{\frac{L}{cv}} - 1\right) \right]$$

In the example shown on Fig. 18 the proper constants have been used to give a hydrograph of the 1927 flood at Sunderland with base flow deducted. The length of rainfall, t_o, was taken the same as used in the authors' analysis, and quantity of runoff equal to the actual runoff.

[7]Zoch's equations give the runoff in sq. mile inches per hour. To obtain c.f.s., multiply by 645.

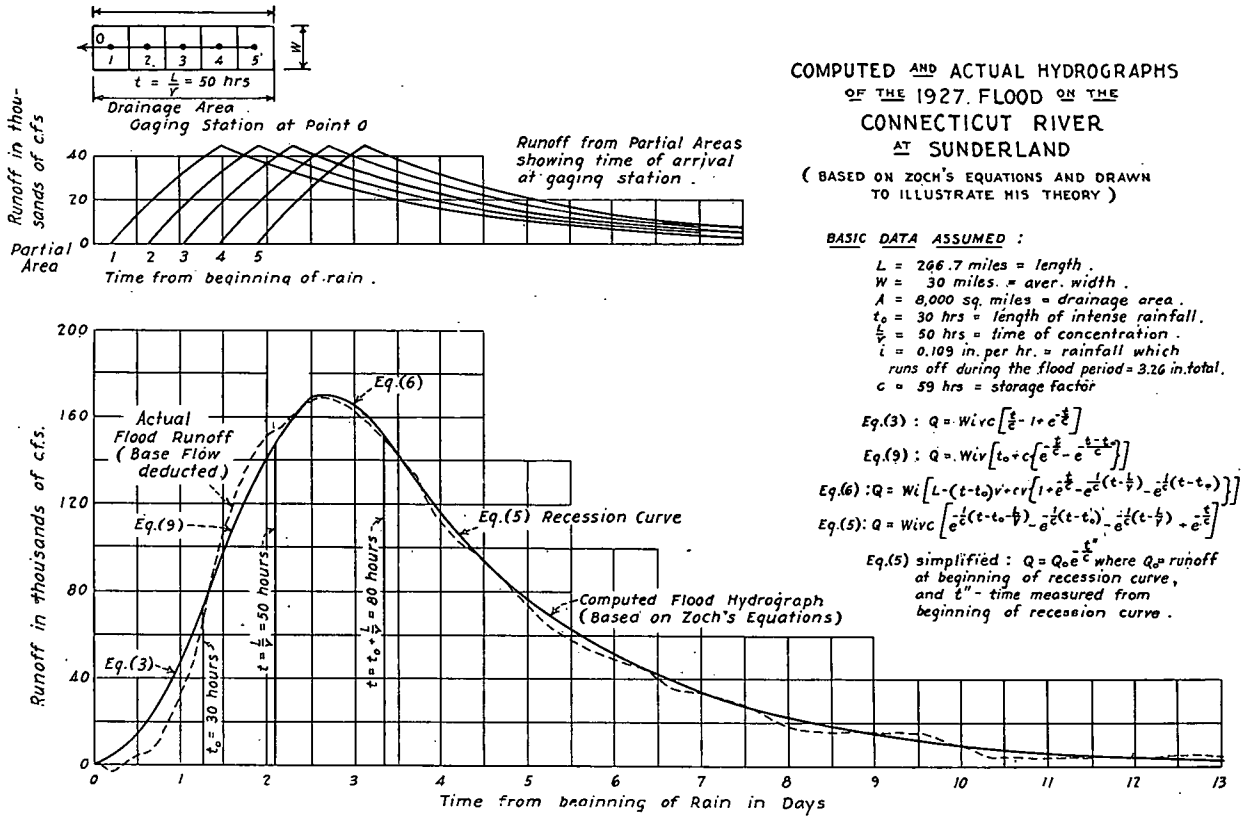

FIG. 18.—COMPUTED AND ACTUAL HYDROGRAPHS OF THE 1927 FLOOD ON THE CONNECTICUT RIVER AT SUNDERLAND USING R. T. ZOCH'S EQUATIONS.

The factor "*c*" was obtained from the storage-discharge curve for the Connecticut River shown in Fig. 2, and the same $\frac{L}{v}$ was used. The resulting curve is the same as the authors' shown on Fig. 6 with the base flow deducted. This must be so because, assuming that the storage-discharge relationship is a straight line, it makes no difference mathematically whether the runoff from each small area is first put through storage and then moved down the river, or whether the runoff is first moved down the river and then put through storage. Neither should it make any difference if the runoff passes through storage part way down the river, and then continues to the gaging station. In a river basin, during a flood, the flow is passing through storage in all three ways. Conceivably the flow could be separated into its component parts, passed through storage, and recombined without affecting the accuracy of the result. Perhaps this fact accounts in part for the close fit with the actual hydrographs of all the curves computed by the authors' method.

All Mr. Zoch's equations given here apply only to a rectangular drainage area. He has also worked out equations for triangular and elliptical drainage areas, and has presented a method for dealing with drainage areas of any shape. The method is complicated, involving the expression of the shape of the drainage area as a power series and the integration of the resulting series term by term. He also developed equations for the magnitude and time of occurrence of the flood crest.[8]

These equations involve a number of terms which in themselves are hard to estimate, in a complicated relationship which makes the proper selection of the constants difficult in any particular case. The same assumptions as required in the authors' analysis must be made as to the time of the rainfall, t_o, and the $\frac{L}{v}$, and also "L" and "W", and the amount of rainfall, "*i*", that appears as total runoff must also be estimated. The factor "*c*", applying as it does to all water withheld from the rainfall and not just the surface and channel storage, may be expected to vary with different floods depending on the condition of the ground. As far as the authors are aware, Mr. Zoch does not present any method of determining the important factor "*c*".

If "c" is made to apply to the surface and channel storage only "c" can be determined from the recession curve of the actual hydrograph. Mr. Zoch's equation (5), which is the equation of the recession curve, may be written in the following simplified form:

$$Q = Q_o e^{\frac{-t''}{c}}$$

where t'' = time from beginning of the recession curve and Q_o = runoff above the base flow at the beginning of the recession curve. This equation plots as a straight line on semi-log paper which allows one to determine "c".

The method presented by the authors thus gives results practically identical with those obtained from Mr. Zoch's general equations if it is assumed that his storage factor "c" applies to surface and channel storage only and the base flow is omitted. The authors' method of taking the whole runoff to the point of measurement first, and then routing it through storage all at one point by use of the storage equation, requires but simple arithmetical computations for its solution. It makes it easy to handle the case in which the storage-discharge relation is not a straight line, as for example on the Merrimack River at Lowell. The method also permits the determination from actual flood hydrographs for any station of the various elements which enter into the flood hydrograph and which appear as the constants in Mr. Zoch's formulae.

$$^{8}t_c = c \log_e \left(e^{\frac{L}{cv}} + e^{\frac{t_o}{c}} - 1\right)$$

$$Q_{max} = Wi\,[L - (t_c - t_o)\,v]$$

Further Adaptations

This paper covers only cases where simple assumptions could be made. The method should prove capable of modification to take care of different shaped drainage areas, in the construction of the flood inflow hydrograph, as indicated previously, though it may be found that even these may be reducible to the trapezoidal and triangular form without too great error. It is believed that it may be possible to allow for varying rates and distribution of rainfall. By the insertion of a time lag factor, it should be possible to fit the case where a storm

has moved up or down the drainage area. Other possible refinements to cover other variations in conditions may also prove to be possible. Very little work has actually been done on these modifications; they are mentioned as possible within the scope of the method presented herein.

It is hoped that this analysis will prove of some value in the use of the flood hydrograph for estimating and predicting floods as it gives a method of determining and using the flood characteristic factors of the drainage area,—the shape, the concentration time, and the total storage, and permits the determining of the effect of storm rainfalls of different intensities and lengths. It should also assist in classifying streams as to their flood producing characteristics. More flood hydrographs with varying characteristics should be analyzed in this way than has been possible in this study to date, particularly on different shaped drainage areas.

DISCUSSION

By G. N. Bailey*

This paper emphasizes the greater importance of the factor of valley storage, while relegating the "concentration period" to much less importance, in their influence on the flood hydrograph. The little experience of my own would support these conclusions, but there is still in my mind some question as to just what this "concentration period" really is. It would seem to me that the downward progress of the flood, after it has entered the real waterways, is completely handled by the valley storage treatment, employing an equation similar to that of the paper, and the basic data of waterway characteristic derived from the recession curve; and that the "concentration period" applies only to the progress of the rainfall down to the point of entering the waterways.

There is no application of "concentration period" to progress of the flood in the waterways simultaneously with the storage treatment; because without storage that part of the concentration period would be zero. If one considers the waterways of a river replaced by a pipe with side branches, constructed of unstretchable material, and carrying an incompressible water with a given regimen of flow; and then introduces at any point in the system a given increment of flow, that increment of flow will appear downstream at the measuring point instantly. Actually of course in any river system it does not so appear instantly, and the reason it does not is storage.

Acceptance of the principle that the concentration period is concerned only with flood progress above the waterways might lead to the conclusion that in country of the same character the concentration period would not vary much with amount of drainage area.

I have attempted to apply the storage treatment in reverse to an actual hydrograph at a measuring point, using the c.f.s.h.[1] ÷ c.f.s. constant derived from the recession curve, in an effort to derive the hydrograph of inflow to the waterways; but ran into such computation

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[1]c.f.s. hours.

difficulties (probably the same as the authors have mentioned) as to make the results valueless. However, if a method could be devised to avoid such difficulties and actually derive a faithful inflow hydrograph, then that inflow hydrograph could be compared to the rainfall record and provide a more scientific approach to understanding of the relation between rainfall and the inflow hydrograph than the more or less arbitrary assumption of a concentration period.

A workable method of deriving the inflow hydrograph from the discharge hydrograph is suggested, that avoids the computation difficulties just referred to; which would be strictly valid only in case the c.f.s.h. vs. c.f.s. relation is a straight line as the equations assume. Because increment c.f.s.h. divided by increment c.f.s. is a constant, one can divide any hydrograph into several, treat each fractional inflow hydrograph by the equation, and then add the resultant fractional discharge hydrographs to obtain a total, identical with that which would result from treatment of the total inflow hydrograph.

To apply this to the problem at hand. Assume a particular discharge hydrograph. Take an inflow hydrograph of simple shape, and by use of the valley storage equation, obtain the corresponding discharge hydrograph. Subtract the latter from the discharge hydrograph started with. Then continue with this residual discharge hydrograph the same treatment, i.e., again pick out an inflow hydrograph, apply the equation to it and subtract the resulting discharge hydrograph from the first residual discharge hydrograph to get a second residual discharge hydrograph. Repeat this process as far as desirable, the criterion being to reduce the final residual discharge hydrograph to unimportant magnitude, and then add the several fractional inflow hydrographs to get the total inflow hydrograph. With a little practice and judicious selection of very simple forms of inflow hydrographs—possibly rectangular by choice—the calculation would not be laborious.

One other point might be mentioned. A modification of the storage equation used in the paper reads

$$T \left\{ \frac{(i_1 + i_2)}{2} - \frac{(d_1 + d_2)}{2} \right\} = K \left\{ d_2 - d_1 + X \left[(i_2 - i_1) - (d_2 - d_1) \right] \right\}$$

in which "X" is a factor of the "wedge" portion of the valley storage increment, which is positive with rising and negative with falling stage.

This is the formula used, I believe, by the Army Engineers in routing floods in the Connecticut River and tributaries, for which they used one common value of $X = .3$; while it ignores factors which might under certain conditions invalidate it, yet it is, so far as I know, the best one in use for such streams as the Connecticut, and furthermore seems to be sufficiently accurate for the purpose of routing floods through successive reaches when well supported by physical data.

In routing through a single reach there is no question that during changing stage the valley storage increment in a given period is made up of two parts: a "prism", which is a function of discharge, and a wedge, plus or minus whether rising or falling, which is a function of the excess or deficiency of inflow compared with discharge. Nor is there any question in my mind that the same principle applies to the river system above any measuring point, because it is simply made up of numerous reaches in various arrangements, in series and in parallel. There is, however, the element of diversity, which is more important the greater the river system, due particularly to its extent and the direction of storms, causing one part of the river to rise while perhaps another part falls; the result being probably to bring the value of "X" nearer to zero. We can form some judgment about the value of "X" from the nature of a reach, but the effect of the "X" of a whole river system demands study and until the results of such a study are available it may be safer merely to accept for the moment the value of $X = 0$, as used in the paper. I have little doubt that even for Sunderland it would be of significant amount, and I suspect that it may properly be varied with different rates of flood rise. Statistical data is necessary to support any such study, and possibly might be available from the Army's Connecticut River Data. Incidentally if "X" be used in routing the flood it should also be used in deriving the storage characteristic "K" from the recession curve.

DISCUSSION

By Prof. H. K. Barrows,* Member

In their study of the flood hydrograph the authors' analysis is novel especially in calling attention to the fact that for a given station upon a river the recession curve of the flood hydrograph is the same, at and below any given elevation of water surface, because as they put it, the flow at any time in this part of the flood hydrograph is "purely a matter of drainage of water from storage," which is always the same at any given water elevation, provided of course that no added rainfall occurs during the recession period.

Hence the recession curve once established can be used for any flood within the limits of observation. Also water in storage in the drainage area above the location point of the flood hydrograph can be determined as shown by the authors and a curve prepared showing storage in terms of outlet flow.

Strictly, the storage between two assumed water levels will be the outflow of water during the time required for this change of level to take place, less a relatively small amount of base inflow, chiefly from ground water which is going on at all times. No great error will, however, usually be involved by ignoring this small inflow, which commonly varies but little in amount.

The above is with reference to a single point upon the river, while the usual routing formula is for a given reach of river *between* two stations or points. In this latter case inflow at the upper end of the reach during a given period of time equals outflow at the lower end during this time plus or minus storage, while the recession portion of the flood hydrograph, less a usually small and relatively constant amount of base flow represents wholly storage in the stream drainage above the location point of the hydrograph.

For the entire recession curve total storage is thus approximately the average flow multiplied by the time during which this occurs—the result being in second-foot days. It is more accurate, however, to construct a storage curve by increments, as the authors have done.

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It is obvious that the storage represented by the falling limb of the flood hydrograph must have been filling up at the time of the rising limb of the hydrograph.

Referring to Fig. 6, viz., Connecticut River at Sunderland, November 1927, an approximation of the total inflow or flow without valley storage effect at the time of the rising limb of the hydrograph has been made by superposing on the latter the fill in of storage during the time that the hydrograph is rising. This has been done in the following manner:

Taking for example flow between 20,000 and 40,000 c.f.s., it will be noted that the period of drop on the falling limb of the hydrograph is about 3.4 days, whereas the period of rise on the rising limb of the hydrograph is about 0.2 day. Consequently for this portion of the rising curve there would be added, as representing storage, an amount of $(30{,}000 - 12{,}000) \times \frac{3.4}{0.2} = 306{,}000$ c.f.s. To this would be added the 30,000 which represents the average flow for this segment of the curve, making a total of 336,000 cfs. to be plotted. For other flows, points are computed in Table 1 and plotted approximately on Fig. 19. The curve is approximate only due to its scale and shape.

It will be noted that the flood hydrograph *without storage effect* would be practically triangular in shape, with the peak flow at the end of time t_0. The curve would pass through the crest point of the actual hydrograph and reduce to base flow at the time of the point of inflection of the descending limb of the hydrograph. Its total area should approximate that of the actual flood hydrograph (above the base flow line).

Obviously, valley storage on a river like the Connecticut is of great effect in reducing flood peaks. In 1927 (see Fig. 6) the maximum inflow to the valley was about 650,000 cfs. whereas valley storage reduced this peak to 180,000 cfs. or about one-fourth of the former.

The flood hydrograph without storage as shown on Fig. 6 for the Connecticut River at Sunderland, is similar in form to actual flood hydrographs of such stations as Pemigewasset at Plymouth, Winooski at Montpelier, and other smaller rivers with steep slopes and relatively little valley storage during the November 1927 flood.

It may be fairly said that the change in form of the flood hydro-

graph between the above forms is due to the resulting typical increase in the amount of valley storage which commonly accompanies increase of drainage area. The shape of the actual flood hydrograph in fact reflects the character of the drainage area as to river slope and valley storage.

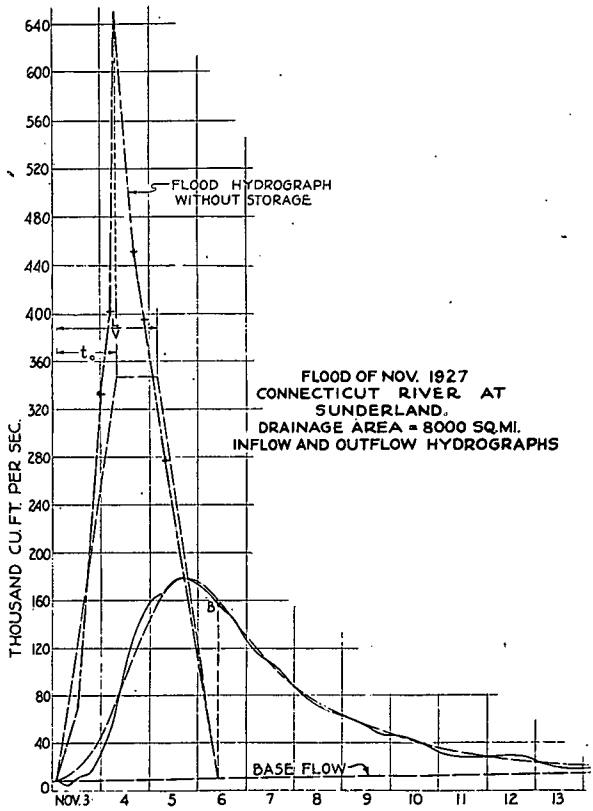

FIG. 19.—FLOOD HYDROGRAPH WITHOUT STORAGE COMPARED TO AUTHORS' INFLOW HYDROGRAPH.

TABLE 1.—SUNDERLAND—NOVEMBER 1927—FLOW WITHOUT STORAGE—COMPUTATIONS

Range of Flow cfs.	Mean cfs.	Base Flow cfs.	Storage Flow cfs.	Time Days from graph Descending	Rising	Ratio	Plotted flow-rising limb cfs.
8,000-20,000	14,000	8,000	6,000	4.0±	0.4	10.±	60,000+ 14,000 = 74,000
20,000-40,000	30,000	12,000	18,000	3.4	0.2	17.	306,000+ 30,000 = 336,000
40,000-60,000	50,000	12,000	38,000	1.4	0.15	9.3	354,000+ 50,000 = 404,000
60,000-80,000	70,000	12,000	58,000	1.0	0.1	10.0	580,000+ 70,000 = 650,000
80,000-100,000	90,000	11,000	79,000	0.6	0.1	6.0	474,000+ 90,000 = 564,000
100,000-120,000	110,000	11,000	99,000	0.5	0.13	3.85	380,000+110,000 = 490,000
120,000-140,000	130,000	11,000	119,000	0.4	0.15	2.67	320,000+130,000 = 450,000
140,000-160,000	150,000	10,000	140,000	0.4	0.23	1.75	246,000+150,000 = 396,000
160,000-180,000	170,000	10,000	160,000	0.4	0.6	0.67	108,000+170,000 = 278,000

DISCUSSION

By John B. Drisko*

The authors have presented a thorough study of the flood hydrograph, and have suggested a very practical method of synthesis. In the cases shown, which have been selected to cover a wide range of conditions, the method gives an hydrograph which agrees with the observed hydrographs of past floods surprisingly well. The method is based on numerous assumptions, some of which seem quite arbitrary, and yet the results are quite satisfactory. The question immediately arises whether this is an indication of the insensitivity of a river to varying conditions over its watershed in producing a flood, or whether it is a reflection on the sensitivity of the method. The paper deserves special credit for its recognition of the importance of the recession curve; also for its clear demonstration of the effect upon the resulting hydrograph of various factors which may affect a flood, such as valley storage, duration of rainfall, and shape of watershed.

The method may be compared with the combined unit hydrograph-flood routing method, in which a downriver flood is derived by progressively adding together the contributions from the several subareas, each determined with the aid of the unit hydrograph, and transferred downstream by flood routing methods The authors' method in some respects seems better suited as a method for deriving flood hydrographs for intermediate areas over which the conditions are more nearly uniform—in competition with the unit hydrograph method. One of the basic assumptions necessary in utilizing the unit hydrograph is that of uniform rainfall over the watershed, and the lack of compliance with this assumption is one reason why unit hydrographs are not satisfactory, in general, for large areas where variations of rainfall, either in amounts or in time of arrival, are to be expected. Furthermore, the unit hydrograph is more valuable in expressing the individuality of small, quick, areas where the delays and "smoothing" due to large valley storage are not present. The unit hydrograph,

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practical as it is, has a weakness which is suggested in the paper, namely, that use of the unit hydrograph to reproduce storms of varying intensities will yield varying recession curves for the final hydrographs. As the authors have so well set forth, however, all floods at a certain point should have the same recession curve, after cessation of rainfall and inflow to the collecting channels. Since the recession curve is a reflection of the rate of draining of the valley storage, and not of the surface run-off and concentration characteristics of an area, the logical conclusion is that the unit hydrograph is suited only to areas where the valley storage is negligible, and that the authors' method is well suited to areas with appreciable valley storage.

The unit hydrograph has the great merit that it recognizes the individuality of a watershed,—and watersheds, especially small ones, do vary greatly. Incidentally, it was before the Boston Society that the first paper on the unit hydrograph method was presented. I refer to a paper, "An Analysis of the Storm of November 1927 by the Rational Method", by Cedric H. MacDougall, presented on November 19, 1930, and published in the December 1933 issue of the Society's Journal. This paper recognizes the individual characteristics of different streams and shows how a flood hydrograph may be derived by adding together successive unit curves, each of which is assumed to result from a unit run-off. This is essentially the unit hydrograph method, and it is unfortunate that the paper presented in 1930 was not published until late 1933, after other treatises on the subject had been published and had claimed the credit. MacDougall's unit curves are derived from a study of topography and stream slope, rather than from an analysis of past floods.

One of the advantages of the combined unit hydrograph and flood routing method is that it suits each storm distribution and timing, and is equally applicable to storms covering the entire drainage area uniformly, and to those covering only certain portions of the area. It is suggested that the authors' method may be combined with flood routing, to give a prediction method using larger sub-areas and involving less detail than the unit hydrograph-flood-routing method.

The authors' study demonstrates the futility of attempting any maximum stage prediction until a storm is complete and there is reasonable assurance that no additional water will fall. Prior to such a time it is only possible to make a qualified prediction.

One additional fact may be gleaned from the paper, and that is the importance of base flow. The slope of the tail end of the recession curve is so flat that the base flow cannot be closely determined. In the examples shown the authors have assumed the base flow in such fashion as to yield close agreement between the computed hydrograph and the observed hydrograph. In actual practice, of course, an assumption of the base flow would have to be made without the knowledge of whether or not it was right, and without the opportunity of checking it in any way. The following table shows the variable assumptions which may be made with regard to the base flow, and also gives figures which do not entirely agree with the statement made in the first paragraph of the paper, that "flood hydrographs . . . had substantially the same length regardless of the amount of run-off, so that the flood peak varied with the total flood run-off".

FLOODS ON CONNECTICUT RIVER AT THOMPSONVILLE, CONN.

Flood	Peak discharge c.f.s.	Duration, days	Volume of run-off, inches	Base flow assumption
1927	190,000	9	3.8	Increase from 20,000 c.f.s. to 30,000 c.f.s.
1936	282,000	10½	3.9	Uniform at 85,000 c.f.s.
		10½	7.4	Including base flow.
		14½	4.6	Decrease from 85,000 c.f.s. to 56,000 c.f.s.
1938	236,000	10½	4.4	Increase from 14,000 c.f.s. to 29,000 c.f.s.

The study represented by Plate 11 is of especial value in demonstrating the profound effect of valley storage upon the peak discharge and general shape of the flood hydrograph. Similarly, Plate 10 shows the much smaller effect upon the flood hydrograph of minor variations in the duration of rainfall contributing to a flood.

The authors have presented a real contribution to the field of hydrology by their close scrutiny of the flood hydrograph, their analysis of the basic elements which go to make it up, and their ingenious method for deriving the flood hydrograph from simple basic data.

DISCUSSION

By Karl Jetter,* Member

The paper on the Flood Hydrograph presented by Mr. Turner and Mr. Burdoin has been studied with much interest. It presents a unique and sound approach to an elusive problem of general concern to engineers dealing with river projects.

After the first reference by the Boston Society of Civil Engineers in 1930 to the significance of the flood hydrograph in analyzing and predicting flood run-offs, and through the later researches and contributions from many sources, there has been a wide range of thought from the approach of the mathematical physicist to that of the practical hydrologist. The former takes the hydrograph as a known integration of many diverse factors giving the particular flood. With an eye to such curves, the three points of inflection and corresponding limbs of the hydrograph are recognized to have definite meanings and equations that might be worked backwards, even into the hinterland of the originating rainfall. The practical hydrologist started from the other end, the originating rainfall, and with various assumptions and approximations, reproduced therefrom the hydrograph of outflow. During this period, the need has been recognized, and studies carried on to evolve the hydrograph characteristics from more general values to those more adequate for basic design of a given river control works. Such basic design may be considered as having the following principal objectives (a) evaluating river flow conditions to determine optimum operation of the project, (b) evaluating basin characteristics for determining the maximum possible flood against which the hydraulic structures would have to be protected. In the latter case, studies made showed that the maximum possible flood from the basins considered could have a peak value greatly in excess of those of actual occurrence as given by the limited period of record. Even for streams having a number of years of recorded discharge, the hydrograph to be used for maximum flood conditions must usually be extrapolated from data based on floods of much lesser magnitude. It is found that for like

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run-off conditions, flood peaks tend to increase in greater proportion than increase in rainfall intensities, so that straight line extrapolations are not safe. Hence it is believed that one of the most useful applications of the methods described in the paper will be to assist in extrapolating known flood hydrographs to the one giving the maximum flood peak inherently possible for a given river basin.

In this connection the paper evaluated the effect of total basin storage. Starting with an actual recorded hydrograph at a given stream measuring point, chosen for certain characteristics, and used as a river basin outflow hydrograph, there is developed an inflow hydrograph and the relation of total storage over the whole basin to the recorded outflow discharge. This so-called inflow hydrograph is considered as one which would have existed at the point of measurement under the assumed conditions were there no storage. Hence from the storage equation, inflow would equal outflow and the derived curve is both an inflow and outflow hydrograph. Considering the developed hydrograph as an inflow hydrograph over the basin appears indefinite. It can equally be considered an outflow hydrograph at the measuring point as modified by total basin storage. As such it is more tangible. It represents the modifying effect of total basin storage. It is an intermediate between a basin inflow hydrograph based on the pluviograph, and the basin outflow as given by the recorded hydrograph at the outflow point of the basin.

In the prediction of floods the pluviograph covering a basin is the first index of expected flood magnitude. Similarly, in evaluating the maximum possible flood, the rainfall-runoff conditions are given, and the hydrograph applicable to these maximum conditions must be determined. The pluviograph, recorded or assumed, corrected for quantities that do not appear as surface runoff, is therefore the initial inflow hydrograph on the basin. For similar basins, the larger the basin the greater the relative reduction in peak value between that supplied by the rainfall inflow and that recorded by the basin outflow. Conversely, the smaller the area, the more nearly the outflow hydrograph approaches in shape and peak value the rainfall inflow graph. It was believed that much of this reduction could be tied to channel and total basin storage, analogous to a large storage reservoir, using the equation: outflow = inflow ± storage. Application of the method given in the report to a drainage area of 622 square miles showed that

only about half the modification or reduction between inflow pluviograph and recorded outflow could be attributed to total basin storage. In order to better represent the quantities, the inflow pluviograph, outflow hydrograph without storage, and the recorded outflow hydrograph were plotted as mass curves. The outflow without storage mass curve plotted about midway between the remaining two curves. However, the effect of basin storage is a large item and its evaluation is a long step toward solution of maximum basin discharge characteristics. There remains then to evaluate the factors causing the modification between inflow pluviograph and the outflow graph corrected for storage, one such factor undoubtedly is the timing of tributary inflows to the main channel and outflow measuring point.

The relationship of total basin storage to recorded surface discharge rate is obtained in the paper by reservoir routing. It appears that a clearer picture could be obtained of the recession side of a hydrograph, representing only flow out of storage, by considering the following: Let (a) be the point of inflection on the hydrograph representing the beginning of the discharge from storage, (b) some other point on the hydrograph between (a) and a point (c) taken at the intersection of the recorded hydrograph and the hydrograph of estimated base flow. Let q be the discharge given at point (a) and q′ the discharge at point (b). The hydrograph is a (time)—(rate of discharge) relation and the area under the hydrograph between two ordinates represents a volume. Hence the volume represented by the area under the hydrograph to the line of base flow and between the ordinate through point (a) and the point (c) gives the volume of total runoff from storage for the discharge rate, q. Similarly the volume may be determined for any intermediate point (b) and corresponding discharge q′. If, as stated, in the paper to be likely, basin storage values (except for the low discharges) are generally found directly proportional to discharge, such a simple relationship would be of great value and permit extrapolating the effect of basin storage on flood peaks greatly in excess of those of record.

The writer does not know what type or shape of recession curve has the characteristic wherein the ratio of areas under the curve are directly proportional to the limiting ordinates for the given areas. However, simple exponential curves were tried of a shape similar to a hydrograph recession curve. Let the hydrograph curve be a simple

parabola with the vertex corresponding to the point represented by the intersection of the hydrograph and base flow. The absisca x represents the time coordinate, and the ordinate y the rate of discharge. The curve then is $y = x^2$. If the ordinates are taken at small and equal intervals represented by a unit of time, the volume for each interval will be approximately the value of the ordinate. The summation, Σy of these ordinates from the vertex or zero ordinate up to a given ordinate will be the total storage corresponding to the given ordinate. In tabular form,

x	1	2	3	4	5	6	7	8	9	10
y	1	4	9	16	25	36	49	64	81	100
Σy	1	5	14	30	55	91	140	204	285	385

Plotting x against y gives the parabolic curve assumed, i.e., $y = x^2$. Plotting Σy against y gives a similar looking curve, but one in which the (discharge rate) to (total storage) relation is not a straight line, varying most at the lowest discharges and appearing to approach a constant ratio as the values increase. The same is true for a hypothetical hydrograph recession curve assumed a straight line between point of inflection and the intersection with base flow, and for a hydrograph shape wherein $y = f(x)^n$. It is of interest to note that the recession side of a hydrograph has added two more storage relationships to the one generally known concept of the stage-storage relation used in reservoirs. The stage-storage relationship in a reservoir is not given by a straight line but to some power of elevation and does not contain the element of time. The two shown by the recession hydrograph are storage volumes vs. time and vs. outflow discharge and have the same interrelation as discharging storage volume from a reservoir.

The earlier assumption made that the base of the unit hydrograph, regardless of the peak, is of constant length in time, cannot be validated on the basis of the later studies on the recession curve of the hydrograph. The time-storage release relation shown by a true recession curve in the hydrograph definitely demands that the hydrograph base be longer for larger peak run-off, much as a reservoir with fixed outlets requires greater time to empty larger volumes. Counterbalancing this to some extent, a greater intensity of rainfall within the time of concentration must fill the basin storage more rapidly and

be conducive to shortening the time from beginning of run-off to peak flow.

In Figure 4 and Figure 5 of the paper are shown the effect of hydrographs for uniform rainfall of a duration less than, equal to and greater than the concentration period L/v. When the duration is just equal to or exceeds the time of concentration there will result a peak value the maximum possible for the given rate of rainfall. The diagrammatic flood run-off inflow hydrograph diagrams do not show this maximum value for the rainfall duration equal to concentration time, but the corresponding flood inflow hydrograph does.

It is possible that the value L/v is reasonably constant for higher discharges applied to a fairly long reach of river. The value "v" from the Chezy formula is dependent on the hydraulic slope, hydraulic radius and a friction coefficient. The average slope for an appreciable length of river will tend to be approximately constant for all above bank-full stages and to be parallel to the average stream bed slope. In this same reach the channel has within certain physical limits, an infinite number of cross-sectional shapes with corresponding hydraulic radii and frictional coefficients. For a given discharge, certain of the sections will discharge at maximum efficiency. At other discharges different sections will in turn discharge at maximum efficiency. This action may tend to average out over a river reach, and give a constant value of "v", where "v" is considered more as the velocity at which a volume is moved through the reach rather than as a mean velocity (Q/A) at the measuring point on the river.

DISCUSSION

By Bruce R. Colby*

The paper presents a very interesting method of computing flood hydrographs. This method of applying a previously computed relation between discharge at a point and the storage in the basin upstream from that point to the hypothetical discharge hydrograph which would result at the point if no water went into temporary storage in the basin will certainly cause much thought and discussion among the hydraulic engineers of the Boston Society.

The authors assume that the run-off generation during a storm can be considered to be uniform. Probably this assumption should be made only for drainage basins which have enough storage to smooth out most of the irregularities which would otherwise result from differences in the rate of run-off generation. If uniform run-off generation is assumed for a basin the net ordinates (ordinates above base flow) of each flood hydrograph are proportional to the corresponding ordinates of a unit hydrograph of one inch of run-off uniformly generated during the length of generation of the actual flood; therefore the flood hydrographs for the basin can be computed easily by using the unit hydrograph theory. The following discussion shows that unit hydrograph theory can be applied to reconstructing the same floods on the Merrimack River above the Concord River at Lowell, Mass., which the authors studied.

The hydrographs for the floods of April-May 1923, November 1927, March 1936, and September 1938 were traced from the authors hydrographs. The lengths of run-off generation were considered to be 24 hours for the storm in 1927, 54 hours for both the storm in 1936 and the storm in 1923, and 60 hours for the storm in 1938. The authors give no estimate of the length of run-off generation for the storm in 1938; their estimates for the other storms were changed slightly whenever necessary to simplify the computations by making the length in hours divisible by six. Their estimated recession curve for the flood in 1936 is reproduced on Fig. 20 without material change.

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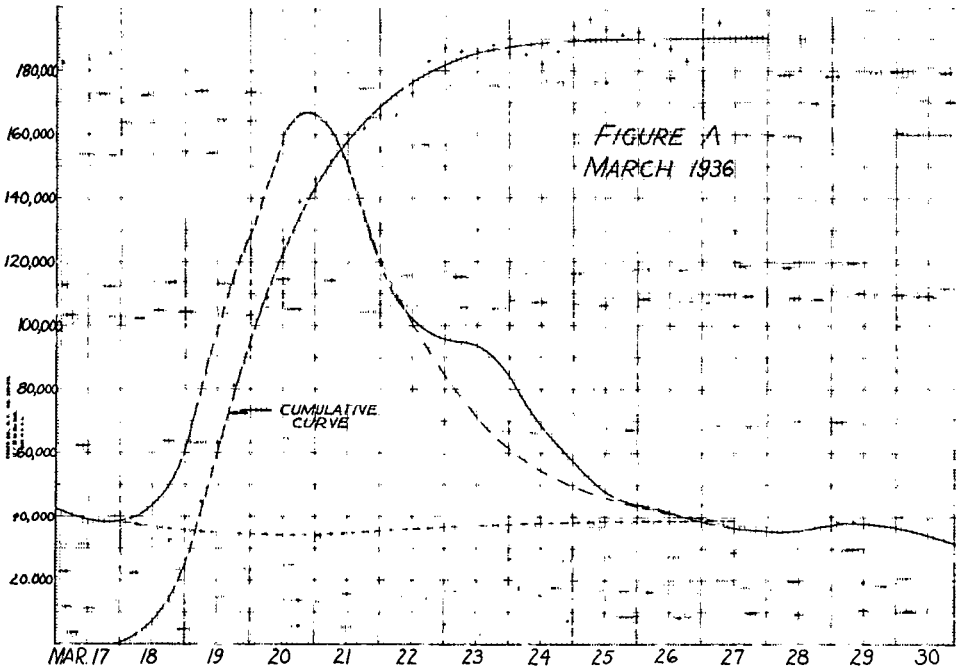

FIG. 20.—HYDROGRAPHS AND CUMULATIVE CURVE OF FLOOD IN MARCH, 1936.

At first their straight-line estimates of base flow were used, and some flood hydrographs were reconstructed. The reconstructed hydrographs agreed reasonably well with those which actually occurred. When the net ordinates of the flood in 1927 were assumed to be proportional to the ordinates of a 24-hour unit hydrograph, the reconstructed hydrographs differed more from the actual ones than they did when the ordinates of the net flood in 1936 were assumed to be proportional to the ordinates of a 54-hour unit hydrograph. Perhaps the hydrograph of a large flood usually can be reduced to reproduce the hydrograph of a small flood more accurately than the hydrograph of a small flood can be expanded to reproduce the hydrograph of a large flood. The estimates of base flow seemed to be causing some inaccuracies in the reconstructed hydrographs.

Figures 20, 21, 22, and 23 show estimates of base flow which may be better for use in unit hydrograph studies than straight-line estimates. The first small rise in the hydrograph for the flood in 1938

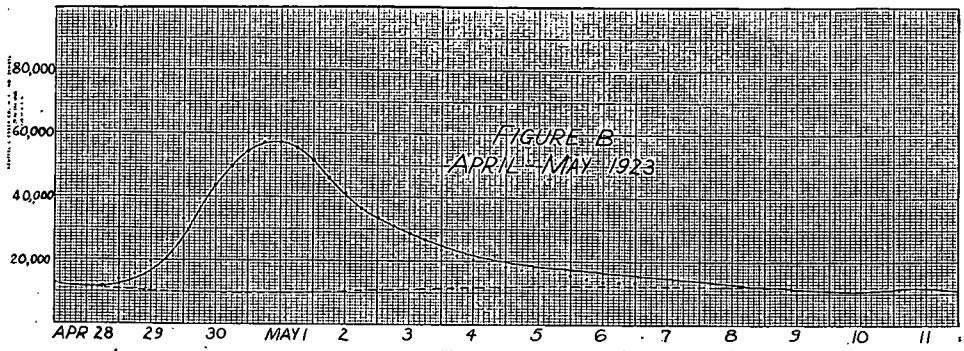

FIG. 21.—ACTUAL AND RECONSTRUCTED HYDROGRAPHS OF FLOOD IN APRIL AND MAY, 1923.

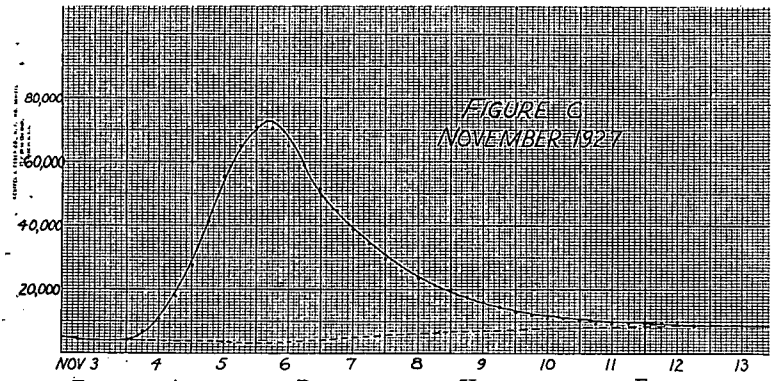

FIG. 22.—ACTUAL AND RECONSTRUCTED HYDROGRAPHS OF FLOOD IN NOVEMBER, 1927.

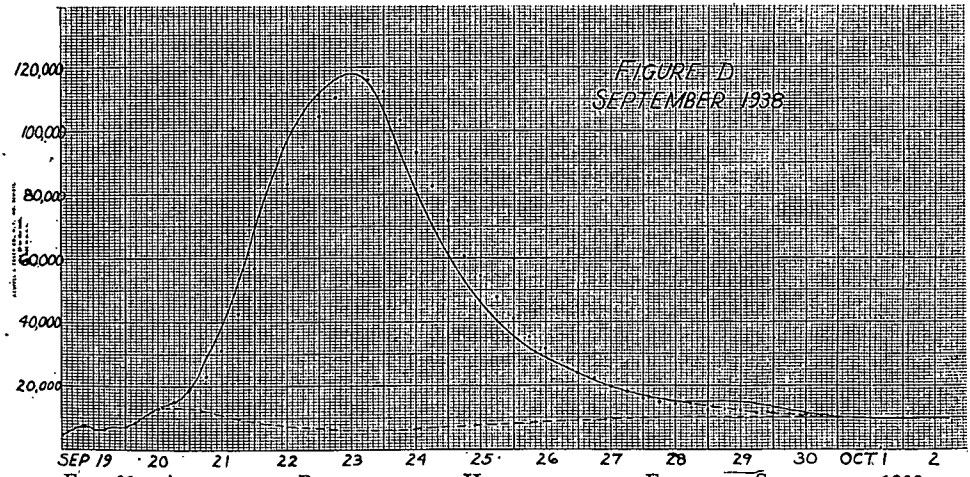

FIG. 23.—ACTUAL AND RECONSTRUCTED HYDROGRAPHS OF FLOOD IN SEPTEMBER, 1938.

was assumed to be caused by local precipitation in the lower part of the basin near Lowell. Hence both base flow and some flow from precipitation which was assumed to have preceded the main storm were deducted to obtain the net discharge. The reconstructed hydrograph indicates that a better estimate of the net discharge might have been made. The net volumes of each flood, expressed as ratios of the volume of the flood in 1936, are as follows:

April-May 1923	0.363
November 1927	.432
March 1936	1.000
September 1938	.885

The computations which were used to reconstruct the three other hydrographs from the hydrograph of the flood in 1936 are given in the table of computations.

Column 1 in the table is a list of net ordinates in thousands of second-feet taken at 6-hour intervals from the hydrograph on Fig. 20 of the flood in 1936. As the storms in 1923 and 1936 are assumed to have resulted from equal lengths of run-off generation, the net ordinates for the reconstructed hydrograph of the storm in 1923 are products of the items in column 1 and the ratio of net flood volumes. These ordinates are listed in column 2, and the points plotted from them are shown by the dots on Fig. 21.

Column 3 is a list of cumulative totals of the items in column 1 at 54-hour intervals. That is, items 1, 10, 19, 28 and 37 were accumulated as a separate series, and the cumulative totals were entered as the corresponding items in column 3. Items 2, 11, 20, 29 and 38 form a second series which was treated like the first. Thus cumulative totals were computed for nine different series of items. The data in column 3 define the smooth cumulative curve on Fig. 20. The plotted points scatter from this curve, at least in part, because the distribution and rate of run-off generation during the assumed period of 54 hours were not uniform. The ordinates of the smooth curve are listed in column 4. Morgan and Hullinghorst[1] have described the use of such a curve. Its ordinates are proportional to those of the rising portion of the net hydrograph which would result from uniform unterminated

[1]Morgan, Russell and Hullinghorst, D. W., Unit hydrographs for gaged and ungaged watersheds, unpublished manuscript, Corps of Engineers, U. S. Army, Binghamton, New York.

COMPUTATION TABLE IN THOUSANDS OF SECOND-FEET

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8
0	0	0	0	0	0	0	0
3	1	3	3	3	3	3	2
7	3	7	7	7	7	7	6
14	5	14	14	14	14	14	11
25	9	25	25	25	24	25	20
45	16	45	42	39	38	42	33
63	23	63	60	53	52	60	48
81	29	81	78	64	62	78	62
93	34	93	95	70	68	95	76
109	40	109	111	69	67	111	88
124	45	127	123	63	61	123	98
132	48	139	134	56	55	131	104
132	48	146	144	49	48	137	109
127	46	152	152	41	40	138	110
115	42	160	158	35	34	133	106
99	36	162	164	30	29	122	97
85	31	166	169	25	24	109	87
73	26	166	173	21	20	95	76
64	23	173	176	18	17	81	65
56	20	183	179	15	15	68	54
48	17	187	182	13	13	59	47
40	15	186	184	11	11	50	40
34	12	186	185	9	9	41	33
28	10	188	186	7	7	34	27
23	8	185	187	5	5	29	23
19	7	185	188	4	4	24	19
16	6	182	188	3	3	19	15
13	5	186	189	3	3	16	13
11	4	194	189	2	2	13	10
9	3	196	189	1	1	10	8
7	3	193	189	1	1	7	6
5	2	191	190	1	1	6	5
4	1	192	190	1	1	5	4
3	1	188	190	1	1	4	3
2	1	187	190	1	1	3	2
1		183	190	0		2	2
1		187	190	0		2	2
1		195	190	0		1	1
						1	1
						1	1
						1	1

run-off generation at the rate of one inch in 54 hours. Evidently the rising portion of a hydrograph from any uniform unterminated rate of run-off generation has ordinates proportional to those of the rising portion of the hydrograph from any other uniform unterminated rate of run-off generation. The ordinates are proportional to the rates of generation. Therefore, unit hydrographs for any length of run-off generation can be computed from a curve like that on Fig. 20.

Column 5 is composed of the differences between each item in column 4 and the fourth item preceding it in column 4. Near the top of the column, the fourth preceding item is, of course, zero. The data in column 5 are proportional to the ordinates of a 24-hour unit hydrograph. Items in column 6 are products of the corresponding items in column 5 and 9/4 of 0.432, the ratio of the volume of flood discharge from the storm in 1927 to that from the storm in 1936. The fraction 9/4 is the ratio of 54 to 24; or, except for slight errors because the computations were not made more precisely, it is the ratio of the sum of the items in column 1 to the sum of the items in column 5. The data in column 6 are plotted as dots on Fig. 22.

Items in column 7 are the differences between each corresponding item in column 4 and the tenth item that precedes it. Items in column 8 are the products of the corresponding items in column 7 and 9/10 of 0.885. The dots on Fig. 23 were plotted from the data in column 8.

The reconstructed hydrographs, which are represented by the dots on Figs. 21, 22 and 23, match the actual hydrographs well, except that the reconstructed hydrograph for the flood in 1938 lags about 5 hours behind the actual hydrograph. A better estimate of the combined base flow and flow from antecedent precipitation might have decreased or eliminated this lag. Each reconstructed hydrograph could be made to agree better with the actual hydrograph if the right changes were made in the somewhat uncertain estimates of base flow and the even more uncertain estimates of length of run-off generation, but juggling these estimates is not necessary to demonstrate the applicability of the unit hydrograph theory.

REVIEW

By Messrs. Turner and Burdoin

The authors are grateful for the interesting discussions of this paper. There has not been sufficient time for them to review all these thoroughly so that these comments are briefer than is deserved by many of the questions raised.

Mr. Bailey raises some interesting points. It appears to the authors that one of the advantages of the analysis proposed is the simplicity of treating the storage all at the point of measurement once the fact is established that the storage curve is the same for all floods at that point. This renders any separation between storage on the ground, in the tributary channels, and in the main river channel unnecessary. Such a separation is difficult to make and would probably be different for different floods, whereas the use of one storage curve for the whole river appears to work for all floods.

Mr. Bailey's suggestion that the storage curve may be different for the rising stage from that of the falling stage, as has been described in the case of channel storage in a reach of a river, is interesting. It was noted in the paper that the figured hydrograph was usually slightly outside the actual hydrograph on the rising side, particularly at the very beginning. This might be accounted for by such a difference in the storage curve.

Prof. Barrows, using the 1927 flood of the Connecticut River as an example, takes the flood hydrograph and using the storage curve figures the rising limb backwards from the flood hydrograph to obtain an inflow hydrograph. This is the reverse of the authors' method of using an assumed inflow hydrograph and figuring from that to the flood hydrograph. He gets a different inflow hydrograph than the one assumed, in that it is triangular in shape with a consequently much higher peak. The authors doubt that, if the inflow hydrograph analysis derived by them (Fig. 4) is correct, inflow hydrographs figured back from the flood hydrograph, as Prof. Barrows has done, will give consistent values for other floods, for the length of the storm,

t_o, and the concentration time, $\frac{L}{v}$. It is this difficulty, as the authors point out in the paper, which makes it preferable to develop an inflow hydrograph and figure the flood or outflow hydrograph from it rather than figuring back the other way. The principle is of course the same, that the shape of the actual flood hydrograph on a stream is the effect of the inflow from the storm runoff flowing into the storage on the surface of the ground and in the channels of the river system and then draining off.

Mr. Jetter takes exception to the use of the term "inflow hydrograph" to describe the runoff without storage. This term has also confused some others. He suggests the use of the term "outflow hydrograph without storage" so as to distinguish it from the rainfall which he considers the inflow. This hydrograph representing a flow at the point of measurement is considered by him as outflow. The term "inflow hydrograph" was used on account of the general terminology used in routing computations. Perhaps it would be clearer if it was called "inflow hydrograph without storage" or even "runoff hydrograph without storage" to distinguish it from the rainfall. He carries the flood hydrograph one step further back to the pluviograph and shows how this inflow hydrograph without storage figured by the storage curve is the intermediate stage between rainfall and final flood runoff. He also discusses the velocity of travel "v", which appears to be fairly constant for varying floods for a given area, pointing out that it may well be subject to the same kind of integration that would appear to be the case with the total storage. However, as was noted by the authors, changes in "v" do not have as much an effect on the flood hydrograph as the other elements involved, chiefly the storage.

Mr. Drisko mentions the paper by C. H. MacDougall, presented before the Boston Society in 1930, but not published until 1933, as being essentially the unit hydrograph method. The authors consider that the foundation of the unit hydrograph method, as H. K. Barrows[1] pointed out in his discussion of Mr. Sherman's original article, was the report of the Society's Committee on Floods, September, 1930, which derived a unit characteristic curve for a given area for 1″ runoff based on actual past hydrographs. MacDougall's curves were based

[1]New Formulas Based on Same Principles as Unit Graph, H. K. Barrows, Engineering News-Record, August 25, 1932.

on "topography and stream slope." Certainly the essence of the unit hydrograph method is the principle originated in the 1930 Flood Report, i.e., that the stream itself furnishes in its hydrograph the measure of its own flood producing characteristics.

The statement quoted by Mr. Drisko that "flood hydrographs . . . had substantially the same length regardless of the amount of runoff so that the flood flow varied with the peak" was the authors' summary of the conclusion of the 1930 Flood Committee Report and not a part of their conclusion. By the authors' analysis this statement is not strictly true, though for storms of about the same length and less than the concentration time it is not far from correct. In Mr. Drisko's table the ratios of peak flow to total runoff for the floods of 1927 and 1938 differ by only 7%. The runoff of the flood of 1936 cannot correctly be compared to the other two without correcting for the additional runoff from rainfall which occurred a few days after the peak of the flood.

Mr. Drisko also compares the method of analysis proposed with the use of unit hydrographs on sub-areas routed through the main river channel storage to the station. He suggests that the unit hydrograph method might apply better to a type of stream with "negligible" storage and the authors' analysis to streams with larger storage. The authors have not made any comparison with the unit hydrograph method in any actual case. It seems doubtful, however, that one method will be found to apply better to one area and the other to another; also that there are any streams where the storage may be regarded as "negligible".

The simplified hydrograph used to illustrate the unit hydrograph method for storms of more than "unit" length was a triangle, the summation of successive triangles resulting in different recession curves. As actually applied, however, the unit hydrograph is obtained from an actual hydrograph so that actual recession curves are used. With a straight line storage curve and no base flow, the authors' analysis should show similar results to those given by the unit hydrograph method, that is, hydrographs of floods for storms of different intensities and different lengths, figured by the authors' method, should give the same results as would be obtained by summing up unit hydrographs according to the standard method. For areas where the storage curve is not a straight line and in cases where the base

flows are different the authors' analysis may be expected to show different results than those from the unit hydrograph.

Mr. Colby presents a series of hydrographs for the various floods on the Merrimack River considered by the authors, which he has worked out by a unit hydrograph method using the 1936 flood to obtain the unit hydrograph. His results as would be expected from the experience in the use of the unit hydrograph, show very close agreement with the actual hydrographs. The whole theory of the flood hydrograph is that a given drainage area acts in the same way in producing different floods, and the unit hydrograph working back from a given flood is based on this.

The authors carry the analysis of the flood hydrograph a step further and endeavor to evaluate the different elements that enter into its shape. This they hope may provide additional means for an extension of prediction of flood flow under varying conditions of rainfall and base flow, for better estimating of floods at points where there are no gagings available, and for classifying areas as to their flood characteristics.

A FEW MOOT QUESTIONS IN STRUCTURAL DESIGN AND ANALYSIS

By O. G. Julian*

(Presented at a meeting of the Northeastern University Section of the Boston Society of Civil Engineers held on December 13, 1940.)

When pressed for an immediate decision as to the subject of this talk; the speaker chose "A Few Moot Questions in Structural Design and Analysis", with the thought that such a subject would allow him plenty of room within which to rattle around without getting cornered. I was later advised that the term "moot" is no longer in general use and that its use dated me as a contemporary of Queen Victoria. With that thought in mind, I should like to go back even a little further than the reign of the dear queen and reiterate a quotation dating from 1820:

> "An intelligent being who knew for a given instant all the forces by which nature is animated and possessed complete information on the state of matter of which nature consists—provided his mind were powerful enough to analyze these data—could express in the same equation the motion of the largest bodies of the universe and the motion of the smallest atoms. Nothing would be uncertain for him, and he would see the future as well as the past at one glance."

As you all undoubtedly know, that quotation is from LaPlace's classic work on the analytical theory of probabilities, and has been the subject of voluminous discussion by logicians and mathematicians, who fundamentally are the same, differing only in that the first expresses his ideas in words while the latter deals largely with symbols. They are both concerned with form rather than with objective realities. In fact, another great mathematician, Jules Henri Poincaré, has stated in "Science and Hypothesis" that:

> "Mathematicians do not study objects, but the relations be-

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tween objects. Matter does not engage their attention, they are interested in form alone."

Engineers, on the other hand, in addition to being interested in form and the relations between objects are primarily interested in how to direct "the great sources of power in Nature for the use and convenience of man."

It is believed that all of structural mechanics may be derived from three fundamental laws:

1. The so-called basic law of statics; i.e., in any isolated system the vectorial sums of the forces and moments must each equal zero. By virtue of d'Alembert's principle this law may be extended to cover dynamics as well as statics.

2. The relationship between stress and strain for homogenous and isotropic materials which, provided the proportional limit is not exceeded, may be expressed by Hooke's law:

$$e_x = \frac{1}{E}\left[\sigma_x - \nu\left(\sigma_y + \sigma_z\right)\right] \qquad \text{(a)}$$

in which σ = stress, e = strain, ν = Poisson's Ratio and the subscripts indicate the axis—to which the component is parallel.

For unilateral stress and strain Hooke's law reduces to the more elementary form:

$$e_x = \frac{\sigma_x}{E}.$$

It is important to note that the elastic "constants" in Hooke's Law vary with the rate of loading and with the temperature of the material. Also, the law applies strictly only to perfectly elastic materials and requires considerable modification when applied to such plastics as steel at high temperatures or concrete. Weird results are often obtained by the application of this law beyond its proper limits.

In the domain of pure plasticity, the corresponding law is:

$$e_x = \frac{1}{C}\left[\sigma_x - \frac{1}{2}\left(\sigma_y + \sigma_z\right)\right] \qquad \text{(b)}$$

in which C is a "constant" the value of which is the strain derivative of stress for the stress rate and duration of loading being considered. Similar expressions to (a) and (b), of course, apply to the orthogonal components of strain.[1]

[1]For a discussion of this most important law, see "Theory of Elasticity" by Timoshenko, p. 7 *et seq.* and "Plasticity" by Nádai, Chapter 14.

3. The principle of Conservation of Energy; i.e., in a conservative field of force the sum of the changes in potential and kinetic energies is constant; or perhaps more simply, energy may be changed in form from potential to kinetic or vice versa but can neither be created nor destroyed.

Modern "rarefied" physics indicates that the only real entity in the universe is energy; matter is a form of energy. Accepting this hypothesis, all of physics and its branches such as mechanics, and hence structural analysis, can be reduced to fundamental laws regarding quanta of energy. Although this line of thought is most interesting, it may be compared to a trip to the stratosphere and it is important, figuratively speaking, that practicing structural engineers keep their feet on the ground. The four entities—mass, space, time and temperature, it is believed, may be considered as fundamental and the other concepts dealt with in structural mechanics, such as velocity, acceleration, force, stress, strain, creep, energy, etc., can all be expressed in terms of these four. I should like to emphasize the importance of energy in structural mechanics. By the use of this fundamental concept, many otherwise involved problems can be solved easily. Among such, are problems in elastic stability, the buckling of columns, beams, etc., deflection problems, statically indeterminate frames, arches, etc., vibration problems and the determination of initial yielding of materials subjected to combined stress.

Force may be defined as the negative of the rate of change of energy with respect to distance, $P = -\frac{dV}{ds}$. This definition is just as sound as that implied by Newton's second law of motion; force is the rate of change of momentum, i.e., the product of mass and acceleration, $P = \frac{d\,(mv)}{dt} = m\frac{d^2s}{dt^2}$. From this energy definition of force, it follows that moment may be defined as the negative of the rate of change of energy with respect to angle of rotation, $M = \frac{-dV}{d\phi}$. It is interesting to note that the term "energy" was born no earlier than the first part of the Nineteenth Century and was created by Thomas Young to whom we are also indebted for the term "modulus of elas-

ticity" or Young's modulus.[2] It is my opinion that the use of this important concept should be given far more emphasis early in our educations. We should all habitually think in terms of energy; there is no reason that electrical or mechanical engineers should do so to a greater extent than structural engineers. This may be a moot question in pedagogy but it is noted that leaders in the field such as Swain, Timoshenko, Von Kármán, Van den Broek, Cross and others have used energy methods to great advantage.[3]

In addition to understanding thoroughly the three fundamental laws previously mentioned, it is necessary to have a working knowledge of the properties of the materials dealt with. The compilation of such data is the province of the testing engineer, rather than the designer and analyst. The latter should not hesitate to confer with and consult the former. When doing so, it is important that he state the given problem fully. This point may best be illustrated by a specific example. Suppose the effective modulus of elasticity of a given lot of concrete is desired, it is not sufficient to state the mix. In addition, at least the water-cement ratio, a full description of materials used, conditions of mixing, placing, curing, age of the concrete, its moisture content, its full history and the rate of loading should be stated, also whether the tangent or secant modulus is desired and for what stress. The effective E for long-time loading may be of the order of but $\frac{1}{10}$ of E for short-time loading. Also, the E obtained in running a test at the standard rate of strain may be materially smaller than that pertaining to high-frequency vibration such as is encountered in many machine supports, or in bridges subjected to high-speed traffic.

Having an understanding of the three fundamental laws and knowing the properties of the materials dealt with, the balance of structural design and analysis consists of horse sense and logic chopping. Since it is far more convenient to deal with concise symbols having precise meanings than with lengthy and indefinitely defined words, this logic chopping is usually put in the form of mathematics.

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[2]Reference: "Source Book in Physics" by Wm. F. Magie, p. 59.

[3]References: "Strength of Materials" and "Stresses, Graphic Statics and Masonry" by Swain. "Statics" and "Dynamics" by Timoshenko and Young. "Strength of Materials," "Theory of Elasticity," "Theory of Elastic Stability," "Theory of Plates and Shells," and "Vibration Problems in Engineering" by Timoshenko. "Mathematical Methods in Engineering" by Von Kármán and Biot. "Elastic Energy Theory" by Van den Broek. "Virtual Work, A Restatement" by Hardy Cross—Trans. Am. Soc. C.E. Vol. 90.

In many cases, it is surprisingly simple while in other cases it leads to arguments beyond the skill of even the best analyst. In such cases it is probably best to resort to simplified premises which are surely on the safe side and to keep on simplifying the problem until it is brought within the power of the mind of the analyst. It is often said regarding such problems, that "theory differs from physical fact" meaning that the result arrived at by considering the simplified problem differs from the true answer observed on a model or the prototype. What is really meant by the phrase "theory differs from physical fact" is that the theory has not been carried out far enough to obtain a close approximation to the true answer.

To illustrate this point, suppose it is required to obtain a close approximation of the deflection of a uniformly loaded, short, stubby beam of rectangular cross section. If the ends of the beam are simply supported, it may be shown by mathematical analysis which considers the effect of compressive stresses acting normal to the neutral surface as well as longitudinal fibre and shearing stresses, that a close approximation for the deflection at mid-span is given by

$$\delta = \frac{5\,wl^4}{384\,EI}\left[1 + \frac{48}{25}\frac{h^2}{l^2}\left(1 + \frac{5}{8}\nu\right)\right] \qquad \text{(a)}^4$$

However, if one has not the opportunity to make such an analysis he may reason as follows:

1. The deflection due to straining of the longitudinal fibres is $\frac{5\,wl^4}{384\,EI}$.
2. Since the beam is short and stubby, the deflection due to longitudinal laminae sliding with respect to each other may not be neglected as compared to that due to flexure; this sliding effect is due to shearing stresses.
3. The intensity of shearing stress at the neutral surface is $\frac{3}{2}$ the average shearing stress over the cross section and it is obvious that the beam must follow its neutral surface.
4. The shearing strain is the shearing stress divided by the modulus of rigidity which equals $\frac{E}{2(1+\nu)}$.

[4] "Theory of Elasticity" by S. Timoshenko, p. 38.

5. Hence, the deflection at mid-span due to shear only, equals $\frac{3}{2} \times$ the area of the shear diagram between the end of the beam and mid-span divided by the product of the cross sectional area and the modulus of rigidity, or

$$\delta_s = \frac{3}{2} \cdot \frac{2(1+\nu)}{E} \frac{w}{bh} \int_o^{\frac{l}{2}} x dx$$

$$= \frac{1}{32} \frac{wh^2l^2}{EI} (1+\nu)$$

6. By adding (1) and (5) the total computed deflection is found to be

$$\delta = \frac{5}{384} \frac{wl^4}{EI} \left[1 + \frac{12}{5} \frac{h^2}{l^2} (1+\nu) \right] \qquad \text{(b)}$$

For Poisson's ratio $= \frac{1}{6}$ the results obtained from (a) and (b) are seen to be practically identical when $\frac{l}{h} = 4$, to differ by 11 per cent when $\frac{l}{h} = 2$ and by 22 per cent when $\frac{l}{h} = 1$. For the latter cases the theory on which the formula (b) is based has not been carried out far enough; cognizance has not been taken of the effect of the compressive stresses acting normal to the neutral surface. These stresses have a braking effect on the sliding of longitudinal laminae and thereby reduce the deflection due to shear. If one gives the problem no thought he may over-simplify it and say that the total deflection is $\frac{5}{384} \frac{wl^4}{EI}$ (because that is the formula ordinarily tabulated in handbooks). For the case cited above, this result is seen to be 13 per cent in error when $\frac{l}{h} = 4$; 53 per cent in error when $\frac{l}{h} = 2$; and 212 per cent in error when $\frac{l}{h} = 1$. For a beam with restrained ends, the relative effect of shearing detrusion is, of course, much larger. Some may think that this example has little practical impor-

tance. However, concrete beams with $\frac{l}{h}$ of the order of 2 are not uncommon in machine supports and for such members it is important to make a fairly accurate estimate of the statical deflection in order to compute their natural frequency of vibration and be sure it is not in resonance or near resonance with the forced vibration set up by the supported machine.

A second example may not be out of place. According to Lamé's well-known elastic analysis, the tangential stress in a thick hollow cylinder subjected to internal pressure only is

$$\sigma_t = \frac{a^2 p}{b^2 - a^2}\left(1 + \frac{b^2}{r^2}\right), \tag{a}$$

in which a = internal radius of the cylinder
b = external radius of the cylinder
r = radius to the circumferential lamina being considered
p = the internal pressure.

This equation states that the stress is maximum where $r = a$, i.e., at the inside of the cylinder. Does this mean that if tested to destruction the first part to rupture would be the inside? Except for brittle material this appears contrary to "horse sense". The metal on the inside of the cylinder, if it stretches appreciably, cannot rupture because it is held by the surrounding metal. If the limit of plasticity ($e_x + e_y + e_z = 0$) is reached throughout, the thickness of the cylinder analysis by Nádai (Chapter 28 of "Plasticity") indicates

$$\sigma_t = \frac{2\sigma_{y.p.}}{\sqrt{3}}\left(1 - ln\frac{b}{r}\right), \tag{b}$$

in which $\sigma_{y.p.}$ is the yield point of the material in simple tension. It should be noted that for this equation to be valid and have a meaning the pressure must be such that yielding in the cylinder is maintained; this condition according to Nádai's analysis is complied with provided

$$p = \frac{2\sigma_{y.p.}}{3}\, ln\frac{b}{a}, \tag{c}$$

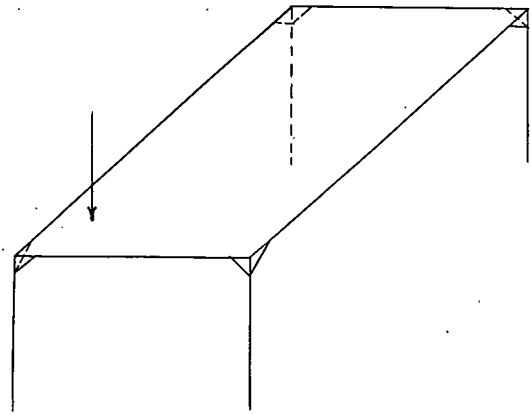
Equation (c) states that the value of the critical pressure increases as a logarithmic function of the thickness of the cylinder while equation (b) states that in case of such pressure the stress will be maximum where r = b, i.e., at the outside of the cylinder and will then equal approximately $1.15\sigma_{y.p.}$ There is really no contradiction between equations (a) and (b); the former applies within the domain of pure elasticity while the latter applies within the domain of pure plasticity. More complicated cases in which some parts of the cylinder, measured radially, are elastic while other parts are plastic on account of the imposition of stress are discussed in the aforementioned reference.

The simple examples just given, it is believed, illustrate the magnitude of the errors that may be introduced by over-simplifying a problem, or of applying well-known formulae beyond their proper range. Such errors are often charged up to "differences between theory and physical fact" and the problem termed a moot question. One should not use a formula without understanding its derivation, seeing clearly the assumptions upon which the argument is based, realizing the proper field of application of the result, and being able to interpret it consciously. Elaborate mathematical discussions are too often based on false and perhaps impossible premises; at other times the premises are not stated clearly. One must take pains to keep "horse sense" in constant command and not to get lost in a maze of abstract and purely mechanical logic chopping. As Swain has said, "You do not understand a conclusion unless you see the steps in its logical demonstration."[5]

Practically speaking, the simplest stable, integrated structure I can think of is that indicated in outline by the adjacent figure which might be a bridge table, or the steel frame of a building. For the sake of simplicity but a single load is indicated, and it is assumed that the distribution of this load among the four bents is known. The design and analysis of this simple structure is full of moot questions.

Assuming, for the moment, that the bottoms of all columns are unrestrained as regards rotation; in order that the structure be stable and capable of resisting either horizontal or vertical loading there must be moment connections between the columns and the beams.

[5]For a masterly discussion of the point of this paragraph see the last chapter of Swain's "Strength of Materials", also "How to Study" by the same author.



Each bent in the frame is obviously statically indeterminate, as are practically all structures. The beams are subjected to flexure in, at least, vertical planes. The magnitude of the moments at the intersection of the beams and columns depends on the degree of restraint between these members. If it is assumed that these joints remain right angles each bent may be analyzed by well-known methods such as the Moment and Shear Distribution (Hardy Cross) Method. However, except in the case of full continuity, there will always be some slip and distortion of the parts at the joints which will cause the angles to become other than right angles. Although it is possible to make corrections for these deformations of the joints their magnitudes are difficult to evaluate. In fact, in the time usually allotted for the design of structures, in practice, it is impossible to figure even approximately the magnitude of these deformations at all joints, and all that can be said with certainty is that the moments at the joints and in the columns are somewhat less than those figured on the assumption of full continuity while the so-called positive moments in the beams are somewhat greater than those figured on that assumption. A moot question is introduced in the design and one must rely on judgment rather than on precise mathematics. It is believed ordinarily advisable to design the structure on the basis of full continuity at the joints, using conservative stresses in the connections and making them as

stiff as practicable, also to have their stiffness roughly proportional to the moments.

Some authorities, however, advocate proportioning the beams on the assumption that they are simply supported when resisting vertical loads and have rigid connections to the columns when resisting lateral loads such as wind; the columns being proportioned to resist the resultant axial forces figured from these assumptions and the flexure due to lateral loads only. Just how the joint connections can be rigid when resisting the lateral loads and act as frictionless hinges when subjected to contemporaneous vertical loads is a mystery. It is noted that the American Society of Civil Engineers Committee on Steel Structures condemn the above-mentioned practice and state in Proceedings for December 1933 (page 1609)

> "Attention has properly been drawn . . . to the need for considering the combined effect of gravity and wind loads on connections. This is an essential of careful designing and is presumed to be observed when designs are made according to the Sub-Committee's recommendations."

In the foregoing it was assumed that the bottoms of all columns are unrestrained as regards rotation. This assumption is valid provided no special precautions are taken for preventing base rotation.[6] However, in many cases, a more economical structure, considering both superstructure and foundations, will be obtained by restraining column base rotation. It is sometimes assumed, without due warrant, that the limiting case of full fixity at these points may be obtained by the use of comparative limber or light foundations. It is axiomatical that, strictly speaking, full fixity can never be obtained; its assumption is paramount to saying that stress may be imposed without causing strain. The degree of restraint actually obtained is a moot question and ordinarily indeterminate on account of lack of data as to the effective modulus of elasticity of the soil under the foundations, the restraining effect of the soil bearing against the sides of the piers, the deformation of the foundations *per se* and the deformation of joints between foundations and the columns. In the case of foundations supported on piles the question of the strains in the piles and

[6]See "An Investigation of Steel Rigid Frames" by Inge Lyse and W. E. Black, Proceedings American Society of Civil Engineers, November 1940.

their supporting medium is also pertinent. Again the question is one that must be answered by judgment rather than by precise mathematics. In certain cases, it is believed justifiable to figure the structure for two limiting cases, i.e., with no restraint at the column bases and with full fixity at those points.

Another moot question in the design of this simple structure is how much to allow for differential settlement of the columns. With some of the column foundations bearing on, say, rock while others bear on clay or other plastic material in which settlement takes place over a number of years, this question may be of prime importance and lead either to radical changes in the superstructure or building it on an integral foundation which may tip as the clay settles with time but which will not flex sufficiently to impose destructive stresses in the superstructure. A question of somewhat the same nature is of common occurrence in the design of power houses where some of the members may grow on account of the proximity of boilers while others do not grow. The moments resulting from such growth may be of considerable magnitude and cause unsightly cracking of supported concrete floors and other masonry if not the imposition of dangerous stresses in the steel frame. It should be noted that the moments resulting from such differential temperature changes may be minimized by the use of long slender members. However, the natural frequency of vibration of such members may be near resonance with the forced frequency of operating machinery and thereby at least place considerable nervous strain on the operating personnel who are subjected to the effect of the resultant vibration which may be of appreciable amplitude. The designing engineer in such cases may find himself between the devil and the deep blue sea and called upon to live up to his name "ingenious one". Too often he may rely on the mercy of providence and the ductility of structural steel.

It is apparent that the load indicated in the figure causes some torsion in the beams which, in turn, affects the magnitude of the moments in the columns. Also, with beams having high torsional rigidities, the moments in parallel bents are interdependent to an appreciable extent. The analysis for this effect is somewhat involved and is ordinarily neglected. In case of a highly restrained floor system, i.e, rigid connections between the floor beams and the beams between columns, its neglect may introduce errors of considerable magnitude,

the figured moments in one bent being too high, while those in the parallel bent are too low. The moot question in this case is for what cases should transfer of moment through beams in torsion be considered and for what cases may it be neglected; one cannot afford to figure it for all cases.

It is customary to neglect the stiffness of the floor system between main beams in evaluating the stiffness factors $\frac{I}{L} = K$ of the latter. As has been pointed out by Hardy Cross and others[7] this may lead to large errors, but the evaluation of this stiffness factor is complicated by numerous moot questions. What is the effective E of the floor which is usually made up of concrete and other masonry? The E of the concrete increases with time but its effective E as applicable to loads imposed over long periods (dead loads) may be but a small fraction of its E for short-time loading (live loads). For what age and what proportion of live load to dead load should E_e be evaluated? How much restraint do the steel floor beam connections afford? Do these floor beams act as an integral part of the masonry floor? What is the effect of openings for stair wells, ducts, etc. through the floor? All of these questions can only be answered by judgment; about the best that can be done is to make a he-man guess at the answer.

If the structure is of reinforced concrete, additional moot questions are introduced, and the designing engineer cannot then fall back on reliance in the ductility of structural steel to cover up his poor guessing, and insure the safety of his work. A number of these additional questions are due to the current infamous practice of allowing important concrete structures to be built without the employment of strict field supervision and competent testing engineers working together. A variation in f'_c of 40 percent or more is not uncommon. There is, of course, a corresponding variation in the stress-strain relationships of the material, its permeability and durability. Steel mills keep chemists and testing engineers on the job constantly; the employment of such talent is reflected in the uniformity of their product. But when it comes to concrete, it is not uncommon to have it manufactured, placed and *not* cured under the supervision of a foreman, who has not read the specifications, and is totally out of

[7]Third Progress Report on Wind Bracing by American Society of Civil Engineers Committee, Proceedings, December 1933 (page 1606).

sympathy with their provisions. Also many specification writers apparently are afraid of being called impractical by those who will manufacture and place the concrete; they write specifications which will cause them the minimum amount of criticism by the so-called practical man rather than specifications which will insure the owner the greatest value for his expenditure.

Concrete design specifications in current use leave much to be desired. Although reinforced concrete has been used for over 87 years and numerous tests have shown that concrete is far from being a perfectly elastic material, most current design specifications are written as if E_c, within the range of working stresses, did not vary with stress or the duration of the loading.[8] They also contain other defects (especially in the treatment of shear and diagonal tension) too numerous to even mention here. It is hoped that the discussion on the Joint Committee Report which is current in Proceedings of the American Society of Civil Engineers will help place such specifications on a more rational basis. At present they give a set of rules of thumb, the judicious use of which in most cases, results in fairly safe and economical structures.

It is interesting and refreshing to note that the A.C.I. Committee on "Plain and Reinforced Concrete Arches" states:

> "Because of the large number of indeterminate factors involved, it has become apparent that it is impossible to predict the unit stresses in an arch rib with any degree of accuracy. Furthermore, the nominal stresses calculated by the best methods available do not bear any consistent relation to the actual ultimate or useful strengths of the rib. Consequently, a new method of calculating the strength of an arch rib has been derived which is based on a recognition of the plastic qualities of the concrete."[9]

There appears no reason why this remark should be confined to arches. (See:—"A New Method of Design Omitting *m*" by K. Hajnal-Konyi in *Concrete & Construction Engineering* (England) January to October, 1937 and "Plastic Theory of Reinforced Concrete Design" by Charles S. Whitney in Proceedings Am. Soc. C.E. for Dec. 1940.)

In this report a set of simple design formulae based on a plastic

[8]For further discussion on this point see discussion on "Unsymmetrical Concrete Arches" and references there given by the writer; Journal Boston Society of Civil Engineers, May 1933.

[9]"Plain and Reinforced Concrete Arches" Report of Committee 312 by Charles S. Whitney, Journal A.C.I., September 1940.

distribution of stress in the concrete without considering the tension is presented. This point of view, of recognizing things as they are rather than as found convenient or as laid down by precedent, is believed to be a long step in the right direction.

It should be noted that the recognition of concrete as a plastic rather than an elastic material is not new; it was pointed out by C. Bach 45 years ago,[10] and by numerous others since then. The articles "Neglected Factors in the Analysis of Stresses in Concrete Arches" American Society of Civil Engineers Proceedings, January 1930, and "Plastic Flow in Concrete Arches" Transactions 1931, by Lorenz G. Straub, in which a theory of plasticity and analysis of plastic members subjected to flexure are given in detail, although highly mathematical, are believed to be especially valuable.

Although even the roughest stress analysis of any reinforced concrete structure requires knowledge of the rate of change of stress with respect to strain and the rate of change of strain with respect to time, such data as are published on these important relationships are marked by their lack of definiteness and clear description of the materials and conditions. (See:—Discussion on Concrete and Reinforced Concrete by the writer in Proceedings Am. Soc. C.E. Feb. 1941, Fig. 1, p. 252.) This lack of clear data introduces moot questions into the design and analysis of all such structures.

Returning to the simple frame outlined by the figure, it is obvious that the columns are subjected to direct forces and bilateral flexure. I know of no text in English which gives a direct analytical general solution for this most common problem. An article published in *Annales Techniques,* Athens, May 1933, by A. Roussopolis, and abstracted in *Beton und Eisen* for March 5, 1939, gives a general analysis in algebraic form and charts which greatly facilitate the figuring of stresses in homologous rectangular sections. This analysis is based on the commonly used hypothesis that concrete is an elastic material. The graphical analysis of this problem, based on the same hypothesis, has been given by B. A. Rich and W. W. Bigelow in the *Journal* of the Boston Society of Civil Engineers for February 1926. This graphical analysis has been extended by A. B. Rich in an unpublished paper. Articles on this subject by Paul Anderson published in *Civil Engineering* for August, 1938, January, and October, 1939 and January, 1940, and by Wm. Saville in the same publication for

[10]Zeitschrift des Vereines deutscher Ingenieure, 1895.

March, 1940, are worthy of note. (See also:—"Design of Reinforced Concrete Columns Subject to Flexure" by Hardy Cross, Proceedings A.C.I. Vol. XXVI (1929) p. 157 *et seq.* and Discussion on p. 775 *et seq.*) However, the writers of textbooks on reinforced concrete in general dodge this most important problem.

If I have given you the impression that the profession knows very little about structural design and analysis, I am sorry. We have come a great distance since Galileo, about 1600, assumed the neutral axis of a horizontal beam to be at its lowest extremity and all fibres to be in tension. His great mind apparently missed entirely the simple law of statics. The introduction of the simple approximate formula, $\sigma = \frac{My}{I}$ about 1800, by Navier was a pure stroke of genius. Even he assumed that the neutral axis was always perpendicular to the forces, and it was not until the early part of the present century that we had an adequate treatment of the common theory of flexure for elastic members subjected to forces in planes other than those parallel to a principal axis of inertia. Reference is made to "An Analysis of General Flexure in a Straight Bar of Uniform Cross-Section" by Prof. L. J. Johnson, Transactions, American Society of Civil Engineers, 1906. We, however, still have a long distance to travel before we can hope to analyze even the simplest structure with any degree of precision, to say nothing of being comparable to Laplace's intelligent being, referred to in the first part of this talk. It is only by the closest sort of cooperation between the testing engineer and the analyst, i.e. mathematician, with common sense always in command that we can hope to progress. Also, we cannot afford to lay so much stress on the application of statics, that the laws of elasticity, plasticity, and above all, dynamics are neglected. The importance of giving careful consideration to elastic and dynamic stability and to the laws of motion are emphatically brought to our attention by the recent failure of the Tacoma Narrows Bridge.[11] It is possible for any structure to be thrown into destructive resonant vibration by small force impulses reoccurring at critical intervals.

In 1686 Newton defined mechanics as the science of motion. It is still just that; we cannot afford to leave the important entities, extension and time, out of our equations. I doubt if there is any such thing, strictly speaking, as a statically determinate structure.

[11]Engineering News-Record, November 21, 1940.

NEW SEWAGE PLANT AT THE MONSON STATE HOSPITAL

By Bayard F. Snow*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers held on January 15, 1941.)

Four score and seven years ago our forefathers brought forth in this Commonwealth a new institution, dedicated as the State Almshouse. Thus, a decade before Abraham Lincoln used the introduction paraphrased above, the institution now known as the Monson State Hospital began its existence. An institution more than half as old as our nation, it has survived many changes, changes in name and purpose, changes in control and administrative boards, and changes in sanitary treatment of its wastes. It is with the more recent phases of its sewage treatment that this paper will treat.

In May, 1854, the State Almshouse was opened in Monson for the housing of paupers. Administration was by a superintendent and the Board of Inspectors of the State Almshouse. In 1863, control was taken over by the newly established State Board of Charity. It soon became evident that most paupers in the care of the state at this time were children and orphans, which brought about, in 1866, the change from an almshouse to the State Primary School. Recurrence of infectious diseases among children resulted in the building of a hospital for the isolation and care of such cases, apparently some time before 1883. From 1879 to 1886, the State Board of Health, Lunacy and Charity exercised general supervision, but from 1886 on, Lunacy and Charity, by legislative edict, had to get along without Health.

In 1895, legislation was passed, abolishing the State Primary School and establishing the Massachusetts Hospital for Epileptics, one of the very early institutions specializing in their care, and the result of agitation by Drs. Bullard and Stedman, who had tried for many years to interest the medical profession in such an institution. Accordingly, all the lands, buildings and personal property belonging

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to the primary school were taken over for this purpose and administered under a board of seven trustees. Much of the sewage of the Primary School had been discharged into the Quaboag River. For the new institution, plans for sewage treatment consisting of eight filter beds covering less than 0.6 A. and built without underdrains were approved by the State Board of Health in 1897. There was a screen and a dosing siphon about 800 feet from the filters. The hospital was opened in May 1898 and had about 200 patients in 1899.

During the next ten years the sewage of the hospital was used mainly for the irrigation of crops on the hillside below the main sewer and was disposed of by filtration during part of the year only. In 1909 plans were presented and approved for three additional beds, somewhat irregular in shape so as to take advantage of the topography and located in the area now occupied in part by the Imhoff tank and trickling filter. These beds were not built, but in 1910 substitute plans by True and Arnolt of Albany were submitted. Three additional beds were proposed, located easterly and south-easterly from the eight existing filters. It was also proposed to do away with the screen and siphon, which had given out, and to build a new screen house, settling tank and dosing chamber with multiple siphons. Changing the northwesterly unit of the old filter beds into two sludge beds was also part of the plan. The records indicate that while this plan was being considered, two of the proposed filter beds were under construction. Approval of the proposed works by the Board of Health was qualified by recommendations for the elimination of the settling tank and sludge beds. The southerly one of the three proposed beds was never built, but was for a number of years considered the next step in enlargement of the plant. The two beds built at that time were made larger and deeper than had been proposed, discharging into an open channel in the meadow.

In the Spring of 1932, when the late Mr. Goodnough and the speaker were first asked to look into the sewage treatment situation, all of the filters were in bad shape and would not pass the flow, which escaped unfiltered to the meadow through breaks in the embankments. At that time there was a small sum of money available for additional sewage filters, not enough to warrant starting a new plant, if this had been possible under the appropriation, or even to make additions of any magnitude to the then existing plant, if such additions were proper.

To clarify the situation we submitted a plan showing sufficient extension of the filter area by a series of beds to take care of the sewage of the hospital. This extension, which was necessarily to the east and approached the well-field and land available for its further development, was not approved by the Department of Public Health. We were, however, later allowed to spend the small appropriation for underdrains in the old 1897 filters and for an intercepting drain and outfall sewer to carry the effluent sewage to the river. This was in the summer of 1933, when P.W.A. was first organizing. We had also made studies and estimates of cost of an Imhoff tank and trickling filter plant near the existing beds and of a complete new sand filter plant in the area between the railroad siding and the river. With the Federal grant offered and emphasis placed on labor, particularly labor at the site of the project, we were instructed to prepare plans and specifications for a new plant, with sand filters, settling and dosing tanks and sludge beds. After several months of the usual alternate periods of feverish activity and more or less patient waiting for action by one or another of the State and Federal agencies which had to pass upon some part of the program, bids were received in January 1934 and the new sand filter treatment plant was constructed. Even those who survived the experiences with new agencies operating under new laws, rules and regulations, will have difficulty in understanding the delays in a relatively simple project. Drawings and specifications were submitted in August 1933, bids received in January 1934, work started in April 1934 and the project completed in November 1934.

The works constructed in 1934 as a W.P.A. project included a ten-inch main sewer, a bar screen, three settling tanks 10 feet by 17 feet in area and with depth variable between six and nine feet. These tanks could be operated in parallel or in series. Four open sludge beds, forty feet by fifty feet in area, received the sludge from the settling tanks. An area for four acres of sand filters was laid out but only six beds of 1/3 acre each were built. These beds were 150 feet by 97 feet at the sand surface. Sewage was discharged on to the beds at two points midway of the long sides, the discharge being accomplished by a dosing siphon discharging from a tank 32 feet by 26 feet in plan and with a six-foot drawing depth. All but three inches of this drawing depth was above floor level. The three inches took in the slope of the floor down to the shallow sump in which the siphon

was located. The volume was about 35,000 gallons, equivalent to four inches in depth on each filter bed. A float operated water level recorder was provided at the dosing tank.

From the Farm Group, which lies across a small brook westerly from the plant, a small amount of sewage is brought to the works through an eight-inch sewer and a six-inch inverted siphon about five hundred feet long. This siphon is provided with a gate valve at its upstream end and a hose connection for flushing out stoppages or accumulations of sediment.

Thus, when in the Summer of 1939, we were authorized to prepare drawings and contracts for new sewage treatment works, the hospital had sewers, settling tanks, dosing tank, sludge beds and two acres of sand filters. The latter had been badly damaged by the hurricane flood of September 1938, and the influent line had been broken; permitting sewage to flow from a break in the bank directly to the river. It is of interest to note that the filters were damaged most severely next to the embankments which acted as barriers to the river flow till overtopped, with a water level in the filters presumably corresponding to the river level at the outlet of the main drain, some eight hundred feet downstream. The water flowing over the embankments gouged huge pits in the sand and undercut the banks, mixing sand, loam, and earth to such an extent that a large part would have had to be washed if the filters were to be restored.

In order to obtain data upon which to base a design, records of the level in the dosing tank over a period of three months early in 1938 were carefully analysed by converting to rate of flow the rise in water level for each hourly period. The short and exceedingly steep main sewer from the hospital buildings, notably the laundry, resulted in very sudden changes in flow at the tanks. Since these changes might occur at any period in the cycle, we were unable to determine the total of each discharge by adding to the tank capacity the indicated rate of inflow during the period of emptying the tank. On the rectangular charts, however, the intersections of the graph of rising water level with the engraved hourly lines were sufficiently clear to permit determination of rise in level for each full hour the tank was filling. No attempt was made to get a complete record for any one day, but during the period of nearly 100 days there were a sufficient number of determinations for each hour of the twenty-four to

permit finding the average for each such hour, and from these to compute the average daily flow as 165,000 gallons. We also found the maximum two hour flow to be at the rate of 335,000 g.p.d. The highest hourly flow noted was at a rate of 481,000 g.p.d.

As a basis for design, therefore, we had the average and maximum flow rates for the present tributary population of 1950, including patients and attendants. Design was based on these figures and a plant built, in which but slight changes will be necessary to care for a population of 3000. The table below gives rates and capacities of the principal features of the plant.

	Present	*Future*
Tributary population	1950	3000
Average flow g.p.d.	165,000	250,000
Max. 2 hr. rate	335,000	500,000
Imhoff tank, flow chambers	27,700 gal.	
Detention, average flow	4 hrs.	2 hrs. 40 min.
Detention, max.	2 hrs.	1 hr. 20 min.
Sludge space, 3810 cu. ft., per capita	1.95 cu. ft.	1.27 cu. ft.
Dosing tank, 5′ x 6′ x 30′		
Trickling filter, 75′ diam.	1.65 m.g.d.	2.5 m.g.d.
With central well 10′ diam.	3.35	5.0
Stone 7′ deep, 0.1 A. in area		
*Secondary tank. Two units,	2 hrs. 45 min.	1 hr. 50 min.
10′ x 17′ x 7.5′ — 19,000 gal.	1 hr. 20 min.	55 min.

**Two of the units of the 1934 tanks utilized.*

There is a fall of forty-three feet in five hundred seventy feet of sewer built in 1934 between the 1910 point of collection and the entrance to the tanks built in 1934 as primary settling tanks and now proposed as secondary tanks for the new tank. With this head available the question of how much loss of head should be allowed in the new works was entirely different from that usually met in design. In fact, the situation was almost unique. No existing or immediately proposed buildings were so located as to prevent our using as much of the available head as might be desirable. On the other hand, increase in number of patients, persuasiveness of department heads, reasonableness of budget director, and generosity of the legislature, may at any time result in added buildings, one or more of which may be so located as to require a sewer considerably lower than the 1910 collecting point.

The new works were accordingly designed for vertical location so as to provide no more than a reasonable allowance for expected losses through the plant, taking the settling tanks as fixing the elevation of the effluent and working back to an intersection with the existing main sewer. At certain critical points, allowance was made for greater losses than normal increase in flow would require, in order that any extreme growth of the hospital can be accommodated by adding whatever works may then become necessary without loss of the investment now made. The elevations at certain critical points are given below:

Invert 12″ main drain, at outlet	El. 34.5
Invert 12″ main drain, at upper end	El. 36.0
Invert 12″ effluent, old dosing tank	El. 43.95
Water surface, secondary settling tanks	El. 52.1
Invert trickling filter effluent at settling tanks	El. 52.5
Invert trickling filter effluent at filter	El. 53.55
Invert radial drains at central well	El. 55.75
Top of filter stone	El. 63.75
Center line of distributor arms	El. 64.63
High water, dosing tank	El. 68.75
Water level, Imhoff tank	El. 69.00
Invert, below Comminutor and screen	El. 71.00
Invert, Kennison Flow Nozzle	El. 72.83
Invert, influent sewer at junction with 1934 main	El. 73.1

The exact elevations of the various units could not, of course, be determined without consideration of their horizontal relationship to each other and to the points of connection to existing works. It was not difficult, however, to find, within the area which was most appropriate for the new works, ground at any desired elevation. Within a distance of five hundred feet, from the ground surface near the upper end of the main sewer of the 1934 plan to the base of the embankment for the railroad siding, the ground dropped from El. 100 to El. 35. There was also considerable slope from west to east. The 1897 and the 1910 filters had been in terraces and were still substantially as built except that the two most westerly beds had been filled in to the embankment level and a salvage shed constructed along their easterly embankment line. Northerly from this shed, on ground sloping northeasterly to a soft, peaty area very little above river level, was a dump of miscellaneous debris and cinders with a covering of earth, this dump being extended to the north and east with a face of ten feet or more, but with no appreciable depth within the area

proposed for the new structures. There appeared to be no serious foundation problem although it was obvious that the trickling filter should be located well to the south and west rather than to invite foundation difficulties further out on the filled area.

The trickling filter was therefore fixed for location by the hydraulic and foundation conditions. The Imhoff tank and Control House were then located on an axis parallel to the 1910 filters and to the end of the salvage shed, taking advantage of an excavation in the hillside from which a small amount of sand and gravel had been removed and leaving room between filter and tank for a roadway to the salvage yard. The influent sewer connects to the 1934 construction at a point determined by the elevations of the works at the Control House.

The new plant includes the following items:

1. 10″ Main influent sewer
2. 10″ Kennison Flow Nozzle; Builders Iron Foundry
3. 10″ Comminutor, quarter-inch slots, hand operation; Chicago Pump Co.
4. Bar screen in by-pass channel.
5. Imhoff tank, 26′ square, 24′-6″ deep.
6. Dosing tank, 5′ x 6′ x 25″ drawing depth. Siphon by Yeoman Brothers Co.
7. Trickling filter, 75′ diameter, 7′ depth. Distributor, with 4-3″ arms by Yeoman Brothers Co.
8. Secondary settling tank, two units 10′ x 17′ x 7.5′.
9. Effluent dosing tank, 35,000 gallon capacity.
10. 12″ effluent sewer.
11. Sludge beds, 4 at 40′ x 50′.
12. Alternating sewage pumps, 250 g.p.m. Chicago Pump Co.

Items 1 to 10 are listed in the normal order in which they function. The sludge beds, Item 11, take sludge from the Imhoff tank or from the secondary tanks. The sewage pumps, Item 12, discharge to a manhole in the basement of the Control House, at the lower end of the main influent sewer. By-passing of the trickling filter, or of the Imhoff tank and trickling filter, is possible.

With the exception of those used by Mr. Kennison himself, on the Rutland-Holden sewer, the metering device is the first Kennison Flow Nozzle installation in New England. The 10-inch diameter nozzle resembles a piece of flange and spigot pipe with which someone took a few liberties, warping it from a circular section at the flange or upstream end, to a section of invert arc topped by converging side

walls. As long as nozzle discharge is unimpeded, and within wide limits, the water level varies directly with the rate of flow. By appropriate piping, with grit catcher and clean-outs, the water level is transmitted to a float well in the basement of the Control House, directly above which is the indicating, integrating, recording instrument. Nozzle and instrument are products of the Builders Iron Foundry. Clear water is piped to a point just above the float well, with a small flow discharged there to help keep the piping clear. An air pump is also provided to build up a pressure in the piping in case of stoppages.

Flumes, channels and nozzles of various kinds and shapes have been used for sewage measurement. With free discharge, careful calibration and a dependable instrument, the flow can be measured with reasonable accuracy by many such devices, whether built in the field or in a factory. This meter should serve its purpose admirably, having no obstructions to the free flow of sewage through the nozzle, and having also the backing of a long and favorably known firm as to its calibration and instrumentation. The point about obstructions is even more important in such an institution than is ordinarily the case, because of the uncanny cunning evidenced in getting the most unexpected articles into the sewer, although those with experience in domestic sewage may well question whether anything could cause surprise.

The meter is good for flows up to 800,000 g.p.d. and although the week-day flows have been found to be under 175,000 g.p.d., momentary peaks due to emptying of laundry wastes and pumping of collected farm group sewage have been more than four times the average. No harm results from such short periods of high flow, but if they exceed the meter capacity too frequently, some way to correct such conditions should be provided in order that flow records may be complete.

From the flow nozzle the normal course is through the 10-inch Comminutor. This device, with ½ Hp motor driving a series of stellite cutters in the quarter-inch slots in the drum, cuts the solids instead of screening them from the sewage. Not only is the screening nuisance largely eliminated but the solids are in better condition for effective treatment in the Imhoff tank. An inclined bar screen in an adjoining channel provides for by-passing the comminutor during repairs or shut-down of electric current.

The Imhoff tank is square in plan, 26 feet on the side in the upper part of the settling space and 19′-6″ in the upper part of the sludge compartment. The bottom is a single hopper, 3′-6″ square at the base and 19′-6″ at the top five feet higher. To facilitate construction and prevent mixing of earth and concrete during the rather difficult building of the hopper, a 2-inch concrete mat was first placed. This permitted steel to be placed and concrete to be spaded without danger of crowding earth into the space where sound concrete was necessary.

The upper part of the Imhoff tank is divided into four flow compartments and three gas and scum spaces. The dividing walls were designed to be cast in place, 6-inches thick and 8′-6″ high in the upper or vertical section and with flaring wings to form the V-shaped bottom of the flow compartments. There would normally be no force, other than their own weight, acting on these walls. The contractor, however, requested permission to pre-cast the dividing walls and set them in slots or rabbets in the main wall. Immediately a complicated series of problems arose. First was a basic question of policy. Within what limits should an engineer prescribe route and destination, methods and results? Engineers are naturally conservative and protect themselves and their clients by following accepted practices. This is generally right and proper, although the cost of conservatism is not always warranted. Contractors, on the other hand, are bold and venturesome, else they would not be contractors. On the credit side, their boldness has resulted in the development of construction methods and economies not dreamed of by earlier generations. On the debit side, construction boldness has sometimes resulted in added costs, delays, bankrupt contractors and occasional acceptance of borderline work.

Other problems, details of stresses during the casting, moving, hoisting and setting of the walls, provisions for bedding, methods of handling, stresses in main walls, details of many kinds were part of the problem, but were all subordinate to the basic question of policy. The contractor submitted sketches and description of the additional reinforcement, sling supports, bearing supports, handling methods, grouting and other details. These seemed reasonable, so he was permitted to proceed.

Two of the walls weighed fourteen tons each, the other four about

twelve tons each. They were cast horizontally, that is, with the main wall horizontal and the flaring wing rising at a slope of six vertical to ten horizontal. Extreme care was necessary in forming these walls to prevent unintentional contacts in their final position. The ends of the upper or vertical sections were made to project four inches into five by eight inch grooves, giving an inch clearance on sides and ends, but the wings were made slightly short to avoid bearing on the supporting wall, which at that level had a flare corresponding to that of the wings. Pipe sleeves through the upper part of the wall, with extra reinforcement passing over the sleeves and down into the middle of the slab, provided opportunity for wire rope slings to be placed. Extra reinforcement was placed to strengthen the wall seats and the ends which would come into bearing. Additional steel was also placed in the main body of the slab to take care of stresses while being moved in a horizontal position and while being turned to the vertical.

Not the least of the problems was that of lowering the heavy and awkward walls into place, using two cranes. More than once the remark was made that one crane could work together but two would be almost certain not to. One crane was out of the question, however, not only because of the weight, but also because a lowered boom to get over the center of the tank would not let the weight swing clear. Accordingly two cranes of M. J. Walsh & Sons of Holyoke were placed, one at each end of the tank, with their outriggers set and counterweighted. Using a Caterpillar tractor, a "dead man" and such help from the crane falls as was possible, the slabs were skidded to a position where the cranes could take the load.

It was in the skidding that the principal difficulty occurred. The walls were prepared on ground about level with the top of the Imhoff tank and some seventy-five feet away. The only practicable route was level for a short distance and then sloped down a grade of about 10%. One of the walls, in tipping at the change in grade, was badly cracked and had to be discarded. Experience gained as the work proceeded, however, resulted in a relatively smooth performance in handling the last section. This was jacked up, before moving from its initial position, and two 12 x 12 skids placed under it. From there to the point where the cranes could pick it up, well greased planks permitted skidding without damage and without great difficulty. Lowering the load into the slots turned out to be simpler than was feared.

The capable operators, one at a time, lowered away for a few inches, first one, then the other, then the first again, until the wall was set. It was tedious. There was nervous tension in all who watched or took part. But it was smoothly done.

The completed tank has five pre-cast walls and one which was cast in place. The contractor says he saved enough by pre-casting so that his total cost for seven dividing walls, one of which was broken, was no more than he figured for casting six in place. Grouting of the grooves was a simple matter and it is doubtful if an observer would notice anything to indicate that the tank was in part assembled from pre-fabricated units. It may be, however, that when some of the effect of surface rubbing has worn away, a future observer will wonder what manner of men left their footprints on the vertical face of concrete walls.

As an experiment to determine whether the dividing walls could safely be pre-cast and hung in place, the job was a complete success, and there is no doubt that, given a similar tank to construct, the contractor will profit by this experience and handle the pre-fabrication and placing without serious difficulty, if and when he has a similar tank to build.

Other features of the tank are of interest primarily to designers of similar structures. Attention is directed to the trough surrounding the tank, permitting reversal of flow. The sewage level in the tank may be carried at El. 69.1 or with just enough depth in the bottom of the trough to carry the flow, or it may be raised about two feet. At the low level it has been found necessary to set weighted deflecting baffles in the influent trough to prevent short circuiting. In fact, the velocity of the sewage entering this channel carried the bulk of the flow to the fourth compartment and acted as an eductor at the entrance to number one, resulting in a reversal of flow. The baffles, however, provide a ready means of controlling the flow.

The trough is 18 inches wide, paved with vitrified liner plates, half of which are flat, the others being curved to a radius of 21 inches and laid tangent to the flat plate so as to raise the outer edge and confine low flows to the side of the trough against the tank. The vitrified plates protect the concrete and provide a surface which is much easier to keep clean.

The flow compartments, four in number, are 4′-6″ wide, with

vertical walls extending six feet below the influent and effluent troughs. Below this point the flare of the walls produces a V-shaped bottom 45 inches deep. A six-inch slot allows solids to settle to the sludge compartment, 19′-6″ square in plan, with vertical walls eight feet deep and a square hopper five feet deep. An 8-inch cast-iron sludge pipe with outlet twelve feet below water level permits periodic removal of sludge to the open sludge beds which were built in 1934. Sludge capacity is figured to the point where the walls start to flare, 21 inches below the slots.

The dosing chamber is designed for frequent small doses, is 5 feet by 6 feet in plan, and draws about 25 inches in depth at each discharge. Some difficulty was experienced at first in that the chamber did not empty sufficiently to start a new cycle, but this was remedied by a baffle to prevent the mixture of excessive air with the incoming sewage. The effect of the entrained air, which carried into the dosing siphon, was to make it appear that the discharge capacity of the siphon was no more than the rate of inflow, and a continuous flow resulted. After the baffles were placed, the cycles of operation were clean cut, beginning and stopping at the designated levels. A drain from the dosing chamber permits by-passing the trickling filter, if required. From the dosing siphon the sewage flows normally to the trickling filter, over which it is discharged from the nozzles in the four arms of the reaction driven rotary distributor. Distributor and dosing siphon are Yeomans-Simplex products. The filter is of interest in that it is the first circular trickling filter in the state. Its area is 0.1 Acre, being 75 feet in diameter, with a 10 foot central well. Seven feet of 2 to 3 inch stone, over an underdrain system of Universal Sewer Pipe Corporation's "aerodrane" blocks provide the filter media. Channels in the Aerodrane blocks lead to eight radial drains which terminate in the central well. The filter stone is Monson granite which has been noted for its hardness and quality as a building material, but which was not known to have any previous use for this purpose. Samples tested by the Sodium Sulphate Soundness methods, A. S. C. E. Manual No. 13, passed by a wide margin. Each piece tested is considered to have failed if, in the 20 day period of alternate immersion and drying, it has broken into 3 or more parts, each of which is 10% or more of the original sample. If 10% of the samples fail in this way or if loss by chips, spalls and flakes exceeds 10% of

the original weight of the sample, the material is regarded as unsound.

Ten pieces of Monson granite were tested; none failing by breaking up, and the loss of fine material being 0.2% of the total sample. This was only 2% of the allowed limit for loss of fine material. It should be noted that neither the samples tested nor the filter media used included pieces from the occasional veins of granite in which there was an excessive amount of mica. At the quarry many garnets were to be seen embedded in the rock. No attempt was made to eliminate them from the stone placed in the filter.

The central well not only receives all filter effluent and permits it to pass through a ten-inch plug valve to the effluent line, but also contains the central column of the rotating distributor, the drain valve for the influent piping and the overflow pipe, which connects to the effluent below the plug valve. Flooding of the filter is possible by closing the plug valve and letting the water rise to the overflow level, set flush with the top of the stone. There were no unusual problems of construction in connection with the filter, although care had to be taken to ensure sound foundation. The eight radial drains divide the bottom into eight sectors. Under the drains and elsewhere as required to bring the slabs to reasonable dimensions, footings were constructed to support the floor slab.

The remainder of the new plant was designed to take full advantage of existing works. One unit of the three settling tanks was emptied and converted to a pump well in which were installed a pair of Chicago Pump Company's alternating sewage pumps to force the sewage from a small part of the institution up to the Control House. With these pumps the incoming sewage flows backward through a screen in the discharge line, through the pump and into a wet well, walled off at the end of the chamber. At the predetermined level, one of the pumps starts, flushing accumulated screenings with the screened sewage, which, by the operation of check valves, the incoming sewage is still passing into the wet well. An overflow into the adjoining secondary tank provides a safeguard in case of pump or power failure.

The two remaining units of the original settling tanks provide an opportunity for secondary treatment, providing space for two hours and forty-five minutes detention at present average rates. Sludge from these tanks can be drawn to the four sludge beds provided in 1934, as can also the sludge from the new Imhoff tank.

The writer wishes to express his appreciation of the cordial co-operation of the various State officials and employees, the contractor, Warnard Constructors, Inc., and of the representatives of the manufacturers of materials and equipment. One point on which the writer takes pride is that the total construction cost of the new plant, $32,757.11, was less than forty percent of the amount which was available. It has been an interesting project to carry out. We believe it will be entirely satisfactory in operation.

OF GENERAL INTEREST

DEDICATION OF WINSOR MEMORIAL AT QUABBIN RESERVOIR

Joint Excursion of Boston Society of Civil Engineers and Northeastern Section, A. S. C. E., June 17, 1941

The Winsor Memorial, a bronze plaque on a monument located at Winsor Memorial Park, an attractive setting overlooking the Quabbin Reservoir and Winsor Dam, was erected in memory of the late Frank E. Winsor, Chief Engineer of the Metropolitan District Water Supply Commission from 1926 until his death on January 30, 1939.

The exercises for unveiling this memorial and its dedication were held on June 17, 1941, at 11 A. M., under the direction of a Joint Committee of the Boston Society of Civil Engineers and of the Northeastern Section of the American Society of Civil Engineers, which had sponsored this undertaking and which had arranged for the erection of the Memorial, a gift from friends and admirers of Mr. Winsor.

The Program was as follows:

WINSOR MEMORIAL UNVEILED BY GRANDSON OF THE LATE FRANK E. WINSOR

Presentation address by George T. Seabury, life-long friend of Mr. Winsor and Secretary, American Society of Civil Engineers.

Unveiling of Memorial by Edward Winsor, Grandson of the late Frank E. Winsor.

Acceptance by His Excellency Leverett Saltonstall, Governor of the Commonwealth of Massachusetts.

Address by Eugene C. Hultman, Chairman, Metropolitan District Water Supply Commission.

Following these exercises a luncheon was provided at the Hangar in the Administration Building of the Metropolitan District Water Supply Commission, at the Winsor Dam.

The excursion in the afternoon included the following points of interest in a 37-mile drive.

Winsor Dam, 2640 feet long, 170 feet high above the bed of Swift River and 295 feet above the sound ledge foundation of the caisson core wall. Constructed by hydraulic fill, 4,000,000 cubic yards above the original surface.

Quabbin Hill Tower—Unsurpassed view of Quabbin Reservoir. 415 billion gallons with a water surface of 38.6 square miles at maximum depth of 150 feet; at present, 100 billion gallons and 17.4 square miles at 96.3 feet depth. Shore line 118 miles, not including 110 islands. Watershed, 186 square miles, not including 96.8 square miles Ware Watershed tributary via Quabbin Aqueduct.

4 miles along scenic highway overlooking flooded site of Enfield.

Quabbin Dike—2140 feet long, 135 feet high above the bed of Beaver Brook and 264 feet above the sound ledge foundation of the caisson core wall. Constructed by hydraulic fill, 2,500,000 cubic yards above the original surface.

Quabbin Park Cemetery. 7500 bodies reinterred.

Reservoir Intake at Shaft 12 of Quabbin Aqueduct. Splendid view of Main Portion of Middle Branch of Reservoir overlooking flooded site of Greenwich and Greenwich Village.

East Branch Shallow Flowage Regulating Dam.

10-mile drive through site of North Dana and upper unflooded portion of Reservoir, finally coming to Route 122 through Petersham and Barre, at which point the excursion was concluded.

The arrangements for this dedication and excursion were made under the direction of the Joint Committee consisting of Arthur D. Weston, Chairman, Francis H. Kingsbury, Treasurer, Frank A. Barbour, Harrison P. Eddy, Jr., Samuel M. Ellsworth, Gordon M. Fair, Frederic N. Fay, Frank M. Gunby, Karl R. Kennison, Robert S. Weston.

This Committee was assisted by John H. Harding, Chairman of the BSCE Social Activities Committee, Ralph M. Soule, and by the Secretary, Everett N. Hutchins.

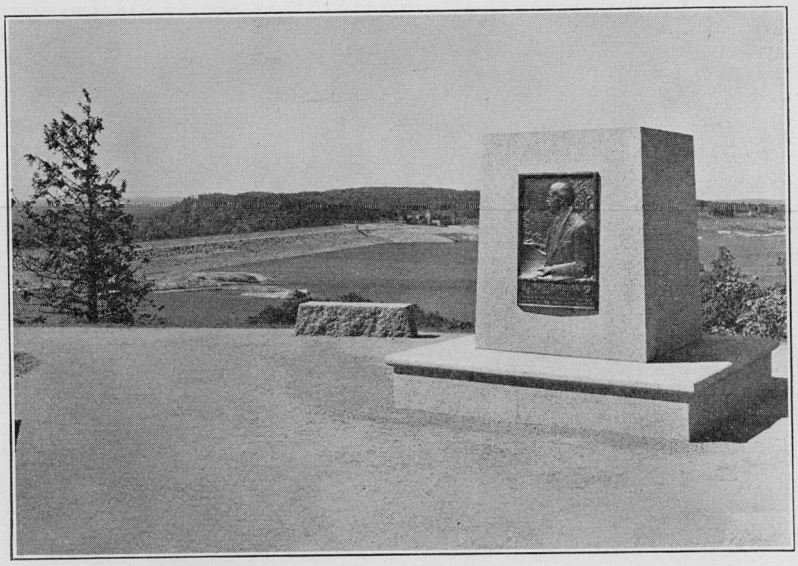
About 175 members and guests attended the exercises and excursion.

of work necessitated beyond expectation. Frank Winsor was the personification of justice as well as a conscientious administrator.

In 1915, he was selected as Chief Engineer of the new water supply for the City of Providence, Rhode Island, which he finished in 1926.

It was in October of that year that he became the Chief Engineer of your Metropolitan District Water Supply Commission and the designer and constructor of these new structures we see about us and others of almost infinite variety, hidden from our sight, but a part of the integrated whole of a 73 million dollar project. He died before they could be finished.

Those who had worked with him and those who knew him well, accounted him famous for his quality of integrity. Mr. Winsor was kind, he was patient, but as has been said, "his faithfulness to what he conceived to be right, constitutes his true imperishable memorial."



Winsor Dam As Seen from Winsor Memorial

He was master of the technical features of design of those elements incident to his major field of work—water supply. He knew tunnels, he knew dams, he knew foundations, he knew aqueducts, he knew the characteristics of rainfall, runoff, percolation, evaporation, floods. He knew the principles of hydraulics, he knew pressures, stresses, strains. These he knew and, knowing, saw to it that they were appraised fully in their respective value, and applied soundly and surely in his works.

Mr. Winsor had the rare capacity of finding capable men and of securing from them loyal service to their very utmost. Nor was it by force or fear that he ruled them. It was by example. He was a leader who inspired loyalty and admiration. No place would he send a man where he himself had not gone or would not accompany him.

ABBREVIATION OF REMARKS BY GEORGE T. SEABURY, SECRETARY, AMERICAN SOCIETY OF CIVIL ENGINEERS, AT THE UNVEILING OF THE WINSOR MEMORIAL OVER-LOOKING WINSOR DAM

We stand here today to do such honor as we may to a man who has done great things, and with flawless integrity—Frank Edward Winsor, recent Chief Engineer of your Metropolitan District Water Supply Commission.

For this brief moment, when the world is seething with hate, when destruction is unrestrained and murder rampant, we pause—to recall a gentle man, a man whose vital physical energy and alert brain were directed solely to the good of mankind; a man whose character was faultless and who died in the very act of successfully defending that character from assault.

Mr. Winsor was born, and received his education as a civil engineer at Brown University, in your neighboring state; but it was in the service of the people of your commonwealth that he began his engineering career, and here it was ended.

It is the primary function of a civil engineer to design and supervise the building of those varied structures that contribute to the well-being of the people as a whole, while others pursue their individual tasks and pleasures.

Frank Edward Winsor began his life work as an engineer in 1891, and for four years continued with the Metropolitan Sewerage Commission of Massachusetts. From 1895 to 1900, on the staff of the Metropolitan Water Board of Massachusetts, he helped to design and build the Wachusett Dam, Reservoir and Aqueduct. For two and a half years more he served your commonwealth in charge of the Weston Aqueduct in Saxonville, in Framingham, and in Wayland.

In 1903, for about a year, New York City engaged him in preliminary study of a suitable supply of water from the Catskill Mountains. Then back to Massachusetts he came for three and a half years to design the dam, locks, and flood control structures incident to the development of your beautiful Charles River Basin.

In 1906, Mr. Winsor became one of that famous group of engineers gathered to build New York City's Catskill Water Supply. There, for nine years, he was a valued member of that unusual group, as Engineer in Charge of the Southern Aqueduct Department.

It was on that work, one day, which I shall long remember, if I may be permitted a personal reference, that under his direction some half dozen engineers, conversant with the problem, discussed a matter of justice to the contractor. The "payline" had been established in the contract but the nature of the rock was such that even by careful handling its excavation could not be held within that line. In consequence it had been necessary to remove considerably more rock than anticipated and to fill the enlarged cavity with more concrete than anticipated. It was decided, under Mr. Winsor's leadership, that the payline should, in fairness, be changed and extra compensation approximating $25,000 given to the contractor.

It was an experience new to me. Mr. Winsor's subordinates had all been instructed to hold the contractors to their obligations in the quality of their work. Here was an instance of being fair to a contractor on the basis of the quantity

It is not to be said that he did not worry. He did, intensely. To him the slightest defect in design or in accomplishment was a matter for concentrated effort towards rectification. Perfection, or its nearest possible attainment, was a goal to which he constantly aspired. A cheat he despised, a subterfuge he fought with all his might. For him nothing should stand in the way of honest work and honest dealing. It was while in defense of these that he died. Frank Winsor has gone but his influence will continue for years in the persons of those for whom he constituted their ideal.

BOSTON SOCIETY OF CIVIL ENGINEERS SCHOLARSHIP IN MEMORY OF DESMOND FITZGERALD AWARDED TO WALTER B. KELLEY, STUDENT AT NORTHEASTERN UNIVERSITY.

Walter B. Kelley of Dorchester, Mass., a senior student, class of 1941, in the Civil Engineering course at the School of Engineering, Northeastern University, was awarded the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald on April 24, 1941, at a convocation of Students held in Jordan Hall. The presentation of the Scholarship of $75 was made by Prof. Albert Haertlein, President of the Boston Society of Civil Engineers.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

April 16, 1941.—A Joint Meeting with the American Society of Civil Engineers, Northeastern Section, and with the Society of American Military Engineers, Boston Post, was held this evening at the Engineers Club. One hundred twenty-five members and guests were present and 100 persons attended the dinner. Mr. C. A. Farwell, President, ASCE N. E. Section, presided. During a brief business session of the BSCE President Albert Haertlein announced the plans for an excursion on June 17, to Quabbin Reservoir and to the unveiling of the Winsor Memorial; also the Secretary announced the election of the following Student Members:

Grade of Student: Jack Cechtel, Warren T. Boutelle, Alamont J. Cornwell, Norman S. Crossley, John J. Cusack, Abraham Danberg, Ernest De Veuve, Paul A. Di Pietro, John B. Dutting, George I. Engle, Frederick B. Jackson, Henry A. Kingsbury, Donald B. Kirby, Edward F. Lobacz, Ralph L. Metcalf, Newton L. Miner, Albert J. Moriaty, Nikola C. G. Patev, Richard J. Spencer, Jr., Herbert W. Standke, Ellsworth H. Tidd, Francis R. Tinsler, Louis P. Vuona, John W. Wiita, Matthew Wolozin.

President Farwell then introduced the first speaker of the evening, Col. Frank A. Gunby, member of the firm, Chas. T. Main, Inc., who later introduced the other speakers on the Symposium "Some Engineering Features at Camp Edwards". Col. Gunby in describing the project as a whole emphasized the rapidity with which every phase of this great project was necessarily carried out and the responsibility for the creation and direction of a huge organization of personnel and the procurement of materials. Mr. Lawrence J. Phillips, Chief Inspector, Chas. T. Main, Inc., outlined the rapid construction procedure of the Camp. Mr. Truman H. Safford, of the same firm, described the facilities for transportation, highways and railroad and the artillery and rifle ranges. Mr. Samuel M. Ellsworth, Consulting Engineer with Chas. T. Main, Inc., described the construction of the Ground Water Supply System and the Sewage disposal plant.

The meeting adjourned at 9:45 P. M.

Everett N. Hutchins, *Secretary*

May 21, 1941.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the New Lecture Hall, Harvard University, Cambridge, Mass., and was called to order at 7:15 P. M., by the President, Albert Haertlein. One hundred and seventy-five members and guests, including ladies attended this meeting. Twenty-four persons attended the dinner at the Faculty Club.

The Secretary reported on the election of the following new members:

Grade of Student: Harold R. Bolivar, Carleton E. Bolivar, Paul W. Cameron, D. K. Clapp, Emory T. Haire, John G. Jarnis, Carl E. Johnson, Harvey Kaufman, Joseph W. Lavin, Wm. McQueen, Richard F. Newton, Lelio Serafini, Ernest J. Scarpa, Francis W. Taylor, Chatson Wong.

The Secretary also reported the resig-

nation from the Society of the following members: Waldo F. Pike, Herbert S. Cleverdon, Edward A. Varney.

The President announced the plans for the excursion to the unveiling of the Winsor Memorial at Quabbin Reservoir to be held on June 17, 1941.

The President then introduced the speakers of the evening, Mr. E. H. Peplow, Jr., who gave a general history of the John A. Roebling's Sons Company and Mr. Blair Birdsall, Assistant Chief Engineer of the Bridge Department of the John A. Roebling's Sons Company, who gave a talk on "Recent Improvements in the Design of Suspension Bridge Cables".

The meeting adjourned at 9:30 P. M.

EVERETT N. HUTCHINS, *Secretary*

DESIGNERS' SECTION

MAY 14, 1941.—A regular meeting of the Designers' Section was held this evening in the Society's rooms. Chairman John B. Wilbur called the meeting to order at 7:00 P. M.

The clerk's report of the last meeting was accepted as read, after which an announcement was made of a proposed inspection trip to Rhode Island sponsored by the Sanitary Section.

Professor Wilbur then introduced the speaker of the evening, Mr. William P. MacDonald, District Manager of the New York office of the Shepard-Niles Crane & Hoist Corporation, who spoke on "The Design of Supporting Structures for Heavy Building Cranes". Mr. MacDonald discussed the types of loads and forces brought on to supporting structures by cranes and presented some "coefficients of common sense" to serve as guides for the approximate design of crane structures. The description of some of the external factors which affect crane stresses and cause failure was particularly valuable. The talk was illustrated with slides and moving pictures showing both typical and highly specialized examples of crane installations.

The meeting adjourned at 9:45 P. M. Attendance 45.

HERMAN G. PROTZE, *Clerk.*

APRIL 9, 1941.—A regular meeting of the Designers' Section was held this evening in the Society's rooms. Chairman John B. Wilbur called the meeting to order at 6:50 P. M.

The Clerk's report of the last meeting was accepted as read.

The Chairman then introduced the speaker of the evening, Joseph S. Newell, Professor of Aeronautical Structural Engineering, Massachusetts Institute of Technology, who gave an illustrated talk on "Structural Problems in Airplane Design". This timely paper dealt with the selection of magnitudes of forces which come on an airplane during flight conditions and with methods of economical design and test of the structure. The specific, quantitative information which was important to both the novice and expert was excellently presented.

An interesting discussion followed the talk. The meeting adjourned at 8:55 P. M. Attendance, 36.

HERMAN G. PROTZE, *Clerk.*

HYDRAULICS SECTION

MAY 7, 1941.—The spring meeting of the Hydraulics Section was held this evening in the Society Rooms. Following a supper at the Ambassador Restaurant, at which 36 were present, 49 members and guests took part in the meeting.

Mr. Stanley M. Dore, Chairman, conducted the meeting.

The minutes of the Feb. 5, 1941 meeting were read and accepted.

Mr. Dore introduced the speakers, Messrs. Howard M. Turner and Allen J. Burdoin, whose topic was "Flood Hydrographs". The paper was interesting and stimulated considerable discussion. The paper was illustrated.

M. T. THOMSON, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[June 20, 1941]

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

RICHARD G. BERGSTROM, Lynn, Mass. (b. February 26, 1912, Lynn, Mass.). Graduated from Northeastern University in 1937. Experience, 1936-1937, Whitman & Howard, Civil Engineers, Boston, Mass.; 1937-1938, General Electric, River Works, Lynn, Mass., and Brooks Engineering Company, Newtonville, Mass. At present, General Electric Company River Works, Lynn, Mass., field engineer and design. Refers to *C. O. Baird, E. A. Gramstorff, H. A. Mohr, J. C. Moses.*

FRANK RAYMOND BERMAN, Boston, Mass. (b. November 29, 1914, Boston, Mass.). June, 1936, received B.S. degree in Civil Engineering at Massachusetts Institute of Technology. Experience, June, 1936, to Sept., 1937, general practice of Civil Enginering (surveying, drafting, etc.); Sept., 1937, to June, 1938, graduate student in Civil Engineering at Massachusetts Institute of Technology, receiving M.S. degree in June, 1938; July-December, 1938, with Fay, Spofford & Thorndike, Boston, Mass., on stress analysis and structural design; December, 1938, to June, 1940, U. S. Navy Dept. at Boston Navy Yard, Hull design section as Junior Naval Architect; June, 1940, to December, 1940, Curtiss Aeroplane Division of Curtiss Wright Corporation at Buffalo, N. Y., as stress analyst on military and naval airplanes; December, 1940, to date, with Fay, Spofford & Thorndike, as a designer (steel and reinforced concrete). Military ROTC at M.I.T. Now 1st Lt. 319th Engrs. (comb.) 94th Div. U.S.A. Refers to *J. B. Babcock, F. H. Fay, K. R. Garland, C. M. Spofford, J. B. Wilbur.*

WARREN M. CAMPBELL, Revere, Mass. (b. February 7, 1911, Revere, Mass.). Graduated from Northeastern University, 1934. Harvard Graduate School, 1934-35. Experience, Massachusetts Department of Public Health, 1935-1938. Harvard Graduate School 1938—½ year. Building Inspector, 1938-40, for Fay, Spofford & Thorndike. At present Civil Engineer with Fay, Spofford & Thorndike. Refers to *G. W. Coffin, A. Haertlein, F. H. Kingsbury, K. C. Reynolds, R. Soule.*

FOZI MITRI CAHALY, Allston, Mass. (b. March 16, 1910, Damascus, Syria). Graduated from the Massachusetts Institute of Technology in 1933 with B.S. degree in Civil Engineering. Experience, U. S. Coast and Geodetic Survey in New York as computer from 1934 to 1935. With Mass. Geodetic Survey from 1935 to 1936. With the National Park Service Department of the Interior, as a Junior Engineer, 1936. Work consisting of designing and drawing plans for roads, dams, water supply systems, sewage disposal systems and other general

engineering work. At present with Fay, Spofford & Thorndike, Boston, Mass. Refers to *J. B. Babcock, G. W. Coffin, W. L. Hyland, M. H. Mellish.*

ARTHUR L. DOW, Rumford, Maine (b. August 25, 1893, Waits River, Vermont). Education, Hebron Academy, graduating in Scientific course in 1912; entered the University of Maine. Experience: 1912-1925, with L. B. Dow, Inc., Boston, Mass., general engineering and construction experience, designing and building coal pockets, grain elevators, flour mills, starch factories, saw mills, hydro-electric plants, industrial plants, and houses. Surveys and investigations for dam sites, pipe lines, and storage reservoirs, and general heavy construction, Supt. of foundry and machine shop two years of this time, manufacturing waterwheels, steam engines, power transmission equipment, general heavy foundry and machine work. General Supt. of all construction the last few years; March, 1925, to November, 1925, with Stone and Webster, hydraulic division, Boston; November, 1925, to May, 1929, in business for myself in Florida and Texas, electrical contracting and house contracting. Pres. of separate company distributing electric refrigeration. Electric contracting covered every phase of light, heat and power, refrigeration and merchandising; August, 1929, to April, 1930, Mechanical Engineer with N. E. Public Utilities Corp., Bingham, Me., dam. In charge of earth fill dam at night, three months balance of time in charge of installation of equipment in powerhouse. General heavy construction equipment and deep air caissons used; April, 1930, to August, 1935, in business for myself, plumbing, heating, electrical work, and general construction; August, 1935, to June, 1936, construction Supt. with U. S. Engineer Corps, Quoddy, Maine, in charge of deep water core drilling and sampling in Bay of Fundy for Foundation Investigation on Passamaquoddy Dams. Transferred to shore Feb. 1, 1936, in charge of two rock and earth fill dams, quarry, railroad, and all equipment; June, 1936, to July, 1937, Field Engineer with U. S. WPA, of Maine, on relief construction work. In charge of all engineering and construction of bridges, roads and buildings, in four counties; August, 1937, to December, 1937, night Supt. with U. S. Engineer Corps, Fitchburg, Mass., on $1,750,000.00 flood control projects. In charge of all operations in all reaches. Responsible for the efficient performance of all work in progress, including excavation, removal of structures, placing of riprap, concrete, etc. Levee construction and all other improvements. Maintenance and efficient operation of all equipment; December, 1937, to April, 1938, Supt. in charge of construction of large storm sewer project and substructure for suspension bridge for Maine WPA, in Town of Rumford, Maine; April, 1939, to May, 1941, Town Manager of Rumford, Maine, population 10,340. Duties included that of Street Commissioner, Purchasing Agent, Overseer of the Poor, Building Inspector, General Finance Officer, Executive Head of the Town Government. At present, Major in Quartermaster Corps, U. S. Army, stationed at Army Base, South Boston, Mass. Present assignment, Plant Protection Officer in charge of Plant Protection of all plants in Corps Area 1, furnishing supplies to the Quartermaster Dept. Refers to *A. R. Curtis, C. A. Mixer.*
them Wolozin.

LESLIE B. ELLIS, Melrose, Mass. (b. August 9, 1886, Melrose, Mass.). Graduated from Massachusetts Institute of Technology in 1908. Experience, 1908-1912 with Metropolitan Water and Sewerage Board on pipe lines, tunnels, pumping stations, etc. 1912-1916 Resident Engineer Inspector with Lockwood Green Engineers, Boston. 1916-1917, Gen. Superintendent of Construction

with Simpson Bros. Corporation, Boston. 1918-1932 with Lockwood Greene Engineers Inc., as Assistant Treasurer and Assistant Manager, Boston Office. 1933-1937 with Federal Emergency Administration of Public Works as engineer on review of plans and specifications for Public Works Projects. 1938-1940, Resident Engineer Inspector for Federal Works Agency on Construction of Sewer Tunnels. Since March, 1941, with Fay, Spofford and Thorndike, Boston. Refers to *G. W. Coffin, L. M. Gentleman, W. L. Hyland, K. R. Kennison.*

CHESTER R. SPIELVOGEL, Southbridge, Mass. (b. November 28, 1911, Clinton, Mass.). Graduated from Worcester Polytechnic Institute, Worcester, Mass., with B.S. degree in Civil Engineering in the class of 1934. Experience, June, 1934 through April, 1941, employed as engineer by the Southbridge Water Supply Company, duties being general engineering nature involved in the maintenance and new construction work connected with the water supply system for a town of about 17,000 inhabitants. One year of this period was spent as Resident Engineer for the construction of a dam, reservoir and pipe line for that company. From April, 1941, to present time employed by Fay, Spofford and Thorndike, Consulting Engineers, Boston, Mass. Refers to *G. W. Coffin, K. R. Garland, W. L. Hyland, M. H. Mellish.*

For Transfer from Grade of Student

PAUL M. LEVENSON, Mattapan, Mass. (b. February 25, 1917, Boston, Mass.). Graduated from Northeastern University, June, 1940. Experience, employed by Platt Contracting Company, Inc., as engineer, since August, 1940, and still working for the same concern at Newport, Rhode Island, on Naval Construction. Refers to *C. O. Baird, C. S. Ell, E. A. Gramstorff, A. E. Everett.*

DANIEL W. MILES, Norwood, Mass. (b. July 21, 1917, Norwood, Mass.). Graduated from Northeastern University, Civil Engineering School, June, 1940. B.S. degree in Civil Engineering. Experience, Construction Safety Engineer with Liberty Mutual Insurance Company from 1937 to present date. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

ADDITIONS

Junior

DANIEL J. CONLIN, 19 Wycliff Street, West Roxbury, Mass.

Students

JACK BECHTEL, 35 Bradford Road, Newton Highlands, Mass.

WARREN T. BOUTELLE, 130 Court Road, Winthrop, Mass.

ALTAMONT J. CORNWELL, 85 Gainsborough Street, Boston, Mass.

NORMAN S. CROSSLEY, 13 Lyman Street Laconia, New Hampshire.

JOHN J. CUSACK, 26 Corona Street Dorchester, Mass.

ABRAHAM DANBERG, 9 Duke Street, Mattapan, Mass.

ERNEST DE VEUVE, 77 Augustus Avenue, Roslindale, Mass.

PAUL A. DI PIETRO, 59 Broadway, Somerville, Mass.

JOHN B. DUTTING, 402 Main Street, Portland, Conn.

GEORGE I. ENGLE, JR., 133 Arnold Road, Newton Center, Mass.

FREDERICK B. JACKSON, 129 Rowe Street, Auburndale, Mass.

HENRY A. KINGSBURY, 1 Curve Street, Medfield, Mass.

DONALD B. KIRBY, 102 Highland Avenue, Dedham, Mass.

EDWARD F. LOBACZ, 161 Salem Street, Wakefield, Mass.

RALPH L. METCALF, 450 Plymouth Street, Bridgewater, Mass.

NEWTON L. MINER, 11 Washington Street, Mexico, N. Y.

ALBERT J. MORIATY, 911 Shirley Street, Winthrop, Mass.

NIKOLA C. G. PATEV, 152 Nilsson Street, Brockton, Mass.

RICHARD J. SPENCER, JR., 492 Central Street, Saugus, Mass.

HERBERT W. STANDKE, 9 Granite Street, Cambridge, Mass.
ELLSWORTH H. TIDD, 85 Central Street, Georgetown, Mass.
FRANCIS R. TINSLER, 71 Toxteth Street, Brookline, Mass.
LOUIS P. VUONA, 13 Groton Place, Worcester, Mass.
JOHN W. WIITA, Cross Street, S. Ashburnham, Mass.
MATTHEW WOLOZIN, 111 Mt. Vernon Street, Malden, Mass.
CARLETON E. BOLIVAR, 11 Upland Road, Winchester, Mass.
PAUL W. CAMERON, 59 Tamworth Hill Avenue, Greenwood, Mass.
D. K. CLAPP, 48 Cohasset Street, Roslindale, Mass.
EMORY T. HAIRE, Rush, Pa.
JOHN C. JARNIS, 29 Warren Avenue, Waltham, Mass.
CARL G. JOHNSON, JR., 152 Mary Street, Arlington, Mass.
HARVEY KAUFMAN, 121 Hutchings Street, Roxbury, Mass.
JOSEPH W. LAVIN, 21 Dunlap Street, Dorchester, Mass.
RICHARD F. NEWTON, 45 Kenwood Street, Dorchester, Mass.
LELIO SERAFINI, 112 Lancaster Street, Quincy, Mass.
ERNEST J. SCARPA, Woodward Avenue, Berlin, Mass.
CHATSON WONG, 65A Beach Street, Boston, Mass.

DEATHS

CHARLES A. FRENCH....April 6, 1941
HENRY A. VARNEY.....April 10, 1941
HENRY C. ROBBINS.....May 6, 1941
RAYMOND C. ALLEN....June 20, 1941

ADVERTISEMENTS

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

Vol. XXVIII JULY, 1941 No. 3

BOSTON SOCIETY OF CIVIL ENGINEERS

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INDEX TO ADVERTISERS

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