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OF GENERAL INTEREST

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SOME FEATURES OF SEWER AND CULVERT DESIGN

By Herman G. Dresser, Member *

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers, March 11, 1936)

Introduction

The purpose of this paper is to discuss the principal factors which are involved in the design of sewers and culverts. There are, of course, many cases which are special in their nature, and it would be impossible to consider them in a short discussion.

In the past a great many erroneous ideas have been held by engineers and practical sewer men regarding the bearing capacity of sewers. This has probably been due, in part, to the opinion of those less experienced, who think that anything when once buried in the ground will support an extremely high load. Many of the practical sewer men have learned only by bitter experience just what can and cannot be done in the laying of sewers.

Another difficulty that distinguishes the properties of sewers from those of other structures is the fact that since sewers are of brittle material, such as tile or concrete, they show no evidence of deflection whatever before cracking. On the other hand, steel or wooden members usually indicate by their extreme deflection that failure is imminent. It is seen, therefore, that many sewers which appear to be in good condition may be simply awaiting the one additional straw which will cause failure. It is also true that many sewers are cracked, although the fact is unknown, because of the difficulty of inspection, or simply

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because inspections have not been made. In many cases these cracked sewers, although they have not collapsed, are in such a condition that a little surcharging of the pipe will cause water to pass through the cracks, saturating the surrounding material and causing a complete collapse.

Mr. Anson Marston, director of the Iowa Engineering Experiment Station, might be called the father of this subject, since he has contributed so much of value to it. In 1908 investigations were first begun at Iowa and they have been conducted almost continuously since that time. In 1913 the first bulletin on this subject was issued by the Iowa Engineering Experiment Station. This was Bulletin No. 31, the title of which was "The Theory of Loads on Pipes in Ditches, and Tests of Cement and Clay Drain Tile and Sewer Pipe," by A. Marston and A. O. Anderson. This publication was probably the first to remove the subject of loads and bearing capacity of conduits from the realms of the unknown and place it on a truly scientific basis.

There were two main reasons for the beginning of these investigations at Iowa. The first reason was that the State of Iowa was making more vitrified pipe than any other State, and the industry was vitally interested in doing everything possible to increase its knowledge of the use of pipe. Another reason was that a great deal of tile pipe had been used throughout the State, particularly for the draining of farm and other lands. There had been a tendency toward using the larger pipes in excess of 24 inches, and many unfortunate experiences had resulted. It was then that engineers began to realize how little was known about the subject, and that there was serious need of conducting investigations.

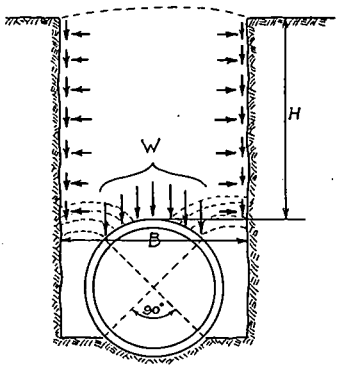
Trench Loads on Conduits

Fig. 1 shows the typical conditions in a sewer trench. The material within the trench is supported by the friction at the sides of the trench and by the conduit itself. The theory for determining the load on the pipe is simply deducting from the weight of the material the increments of friction at the sides of the trench. These increments of friction are derived from the horizontal pressure, as obtained from Rankine's formula, and the coefficient of friction between the undisturbed bank and the filling material.

Tests have shown that the load on the conduit acts almost entirely in a vertical direction. As indicated in the diagram by dotted lines, there is a tendency for the material to arch between the top of the conduit and the sides of the trench, there being practically no horizontal pressure against the sides of the pipe except for special conditions which

will be discussed later. The material at the sides of the pipe is usually placed in a comparatively loose state, and consequently a very slight settlement of this material allows the entire load, due to the backfilling, to be suspended on the pipe. It has been found that, due to the arching in the material, the earth load is usually applied only on the top quadrant of the pipe.

Below the figure is shown the formula which results from carrying through the mathematics of the theory above discussed. The simplified form of the formula contains a constant which may be readily obtained from diagrams.



$$W = \frac{1 - \frac{1}{e^{2KU'\frac{H}{B}}}}{2KU'} \; wB^2 = CwB^2$$

FIG. 1. — TYPICAL LOADING CONDITIONS IN TRENCHES

Tests have proved conclusively that the width of trench at the top of the pipe determines the extent of the load, even though the top of the trench be considerably wider. This is due to the fact that with trenches wider at the top the planes of cleavage are still vertical planes as if the trench were so excavated. The material outside these vertical planes is supported on the undisturbed material at each side of the trench.

Nomenclature

$B = B_d =$ Width of trench measured a little below top of conduit (feet).

$B_c =$ Outside width of conduit (feet).

C = Coefficient used in computing trench loads on conduits (abstract number).

D = Internal diameter of pipe (inches).

H = Depth of earth cover over conduit (feet).

W = Total vertical load on conduit (pounds per foot length).

w = Unit weight of fill material (pounds per cubic foot).

K = The ratio of active horizontal pressure at any point in the fill to the vertical pressure which causes the active horizontal pressure (abstract number).

$$K = \frac{\sqrt{u^2+1}-u}{\sqrt{u^2+1}+u} \text{ (Rankine's formula).}$$

u = The coefficient of internal friction in the fill materials (abstract number).

u' = The coefficient of sliding friction between the fill materials and the sides of the ditch (abstract number).

It is necessary to determine the probable width of trench before computing the loading. For most pipe sewers the following formula is suggested:

$$B = 1.3\ D + 16''$$

in which B and D are in inches. This formula is derived on the basis of a total clearance of 3 inches between the bell of the pipe and the rangers, 4-inch rangers and 2-inch sheeting. For small pipes, a 3-foot minimum trench should be used. For practically all other concrete or vitrified pipes the above formula will be sufficiently accurate for design purposes, unless for some reason an extra wide trench is anticipated.

Fig. 2 shows a diagram for obtaining values of C for use in the formula $W = C\,w\,B^2$. This diagram was published in Bulletin No. 31 of the Iowa Experiment Station, and has since been reprinted many times in other publications. It gives values of the coefficient for different kinds of material ranging in character from dry sand to saturated clay.

The values of w, the unit weight of filling material, should be about as follows:

Dry sand or damp top soil, 100 pounds per cubic foot
Saturated top soil, 110 pounds per cubic foot
Wet sand or damp clay, 120 pounds per cubic foot
Saturated clay, 130 pounds per cubic foot

The method of obtaining the load due to the backfilling is to determine the trench width B, obtain the value for the maximum cover H, and then from Fig. 2 read the proper value of the coefficient C.

Substituting these values, together with the weight of the material, in the formula will give the total load W, which is the total load per linear foot of pipe.

This method of obtaining the load on the pipe is applicable for all trench widths up to the point where the trench width is equal to 1¾ to 2 times the outside width of the conduit. Trenches in excess of this width approach more nearly an embankment condition, and have somewhat higher loads than this method would indicate. This is discussed further under "Embankment Loads."

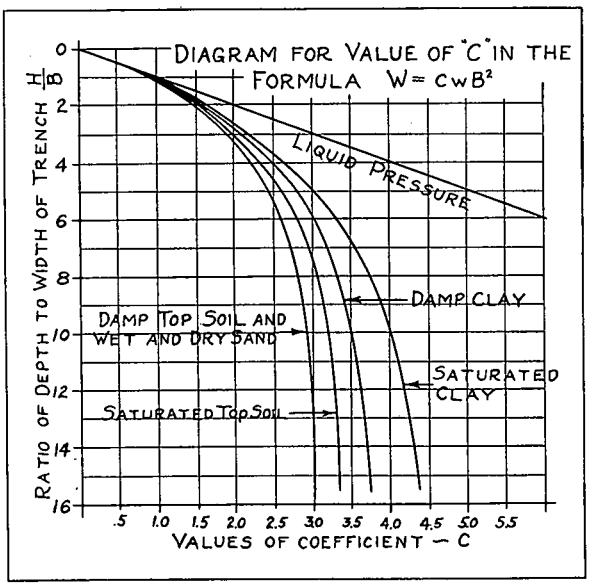


FIG. 2. — VALUES OF COEFFICIENT "C"

The use of sheeting may change the amount of the load. If sheeting is left in place the friction between the backfilling and the sheeting is less than when the sheeting is removed, with the result that the load on the pipe will be increased about 10 per cent. If the bracing is left in as well as the sheeting, the resistance of the braces will make the load on the pipe about the same as in an unsheeted trench.

If the backfilling is in a granular state, and the sheeting is removed, as the backfilling progresses the material will adjust itself so that there will be no effect on the load. On the other hand, if the backfilling is replaced without removing the sheeting, and the material is of such a

nature that it consolidates, developing cohesion within itself, then the removal of sheeting becomes a more serious matter. In this case the entire weight of earth between the sides of the trench may have to be borne by the conduit.

As previously stated, in general the load on the conduit acts only vertically and on just the top quadrant. This condition is almost invariably true immediately after placing the backfilling. Usually the most severe load on the conduit occurs at the time of the first thorough saturation of the material. This may occur within a few weeks, or it may be several years before it occurs, depending entirely on conditions.

In the case of some materials, particularly those having a tendency to flow when wet, successive saturations may result in an active horizontal load against the sides of the conduit. These horizontal loads may then be considered equal to those obtained by Rankine's formula.

In the design of large conduits the engineer is particularly interested in the horizontal load against the structure, for the absence of this load places a more severe condition on the conduit. In this connection the case should be considered of vertical sheeting being driven to the bottom of the concrete and used as a side form for the concrete, or with only a narrow clear space between the two. This sheeting may be pulled after or during the placing of the backfill, in which case a void will be left in the material at the sides of the conduit. This void will prevent any active horizontal pressure at the time the backfilling is completed, and may continue to be responsible for the absence of side pressure for several years. The attempts on the part of contractors to fill the voids left by the removal of sheeting is generally only a pretence and should not be counted on in design.

The theory of trench loads has been carefully checked at the Iowa Experiment Station by two methods.

In the first method test trenches were built in such a manner that the actual weight on the pipe could be weighed, the trench having wooden bulkheads at the ends so that the only frictional resistance was at the sides of the trench as in actual construction.

The second method of testing the theory was to investigate pipes that had been laid with known width of trench. Predictions were made by the theory as to whether or not the pipe should have cracked. Examinations were then made to determine how well the predictions were substantiated in fact, and it was found that the results agreed very consistently with the predictions.

Embankment Loads on Conduits

The typical embankment condition consists of a pipe bedded to the extent of about one fourth of its circumference in the existing ground surface and with 80 to 90 per cent of the outside diameter projecting above this surface. Earth is placed around and above the pipe to form an embankment.

Assume that the pipe is rigid and is bedded in unyielding soil. The earth at the sides of the pipe will compress and settle more than the pipe. The result is that a *wedge* of earth is left suspended on the pipe, and this explains the fact that in some cases embankment loads are greater than trench loads for the same depth of cover.

This is practically the worst embankment case outlined above. If a flexible metal pipe is used, or if the soil beneath the pipe is yielding, then the top of the pipe moves downward, due to the weight of earth, thus reducing the load. This may occur even to the extent of compelling the earth at the sides of the pipe to partially support the earth above it through friction in the material. In other words, the condition may become similar to that of trenches, as previously discussed.

The above cases indicate the two extremes of embankment loading. All combinations of conditions will result in loads somewhere between these limits.

The complete theory of embankment loads is not discussed in this paper, since it is rather involved. It consists of considerations similar to those discussed for trenches, and, in addition, the consolidation of the earth at the sides of the pipe, the flexibility of the pipe, and the compressibility of the ground on which the pipe and surrounding earth rest. For all practical purposes the method given below for obtaining embankment loads will be sufficiently accurate for design use.

The results of actual weighing tests at Iowa indicate that for trench conditions, if the width of trench is increased, it eventually reaches a width beyond which there is no increase in load on the conduit. This width, at which the load by both theories is the same, is known as the "transition width." The "transition-width" ratio is the ratio of the trench width (B_d) to the outside width of conduit (B_c) when the value of B_d is equal to the "transition width."

Table 1 gives values of the transition-width ratio for several types of foundation materials and for a range of values of $\frac{H}{B_c}$ (ratio of depth of cover to outside width of conduit), also for flexible pipe. The column

for rock may be either rock underlying the conduit or firm gravel of high bearing value or similar material. The firm soil may be similar to a firm clay. It will be noted that the better the foundation conditions the higher the embankment load.

The values in the last column are for flexible pipe or trench condition. In other words, if the ratio of width of a trench to the width of conduit exceeds the values in this column the trench is too wide to figure the load by the trench method and the embankment method should be used.

To compute the load due to an embankment, select the proper value of the transition-width ratio from Table 1.

TABLE 1. — VALUES OF THE TRANSITION-WIDTH RATIO

$\frac{H}{B_c}$	Foundation Material — Rigid Pipe			Flexible Pipe or Trench Condition
	Rock	Firm Soil	Yielding Soil	
2	1.70	1.70	1.60	1.30
4	2.25	2.10	1.95	1.50
6	2.60	2.40	2.20	1.70
8	2.85	2.65	2.40	1.85
10	3.05	2.85	2.55	2.05
15	3.55	3.30	3.00	2.40

Multiply this value by the outside width of conduit to obtain the transition width. Using this transition width as width of trench and the depth of cover H, compute the load by the method for trenches first discussed. The resulting load will be the actual load on the conduit for the embankment condition.

One of the principal differences in the load on the conduit due to embankment and trench conditions is that in embankments there is an active horizontal pressure on the projecting portion of the conduit. Tests have shown that the intensity of this horizontal pressure follows very closely Rankine's formula.

LOAD DUE TO CONCENTRATED SUPER-LOAD

A simple method has been developed for computing the load on pipe due to a concentrated surface load. This method first appeared in Iowa Bulletin No. 96, in 1930.

In 1885 Boussinesq developed a method for expressing mathematically the stress at any point in an elastic material, having an infinite plane surface, due to a concentrated force acting normally. It was suggested that this method might be applicable to super-loads above pipes, and investigation proved that to be the case.

Fig. 3 is a diagram based on the above theory which has been made available for easy use. By using the proper curve for outside width of the conduit and depth of cover the per cent of super-load may be read. This per cent represents the portion of the total concentrated super-load

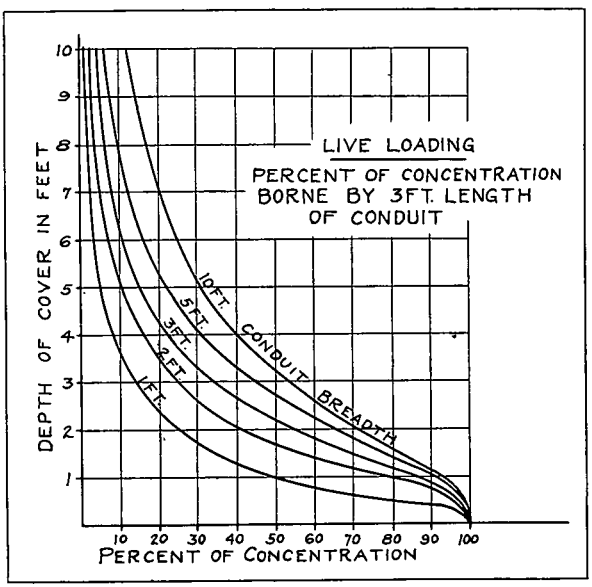


FIG. 3. — LIVE LOAD ON CONDUITS

(such as a truck wheel) which is borne by a *3-foot* length of conduit directly below the load.

Weighing tests of a pipe in trench with moving trucks passing over the trench indicate that normally a 50 per cent increase for impact due to moving loads is sufficient. Unusual cases of high speed and an obstruction immediately over the trench will give an impact of 100 per cent or over, but this is a contingency that may well be cared for by the factor of safety.

Usually for small pipe the concentration is taken as one wheel with 50 per cent impact added. It is true that for depths of cover in excess of

4 or 5 feet there is some overlapping of load from the two rear wheels. However, when the depth becomes sufficient for overlapping the effect of impact is less, so that they partially compensate each other. For large conduits having trench widths greater than the width of trucks the load of both rear wheels should be taken as a single concentration. The load due to a super-load is distributed over the top quadrant of the conduit.

Bearing Capacity of Pipes

The strength of standard sewer and culvert pipes, of vitrified tile or concrete, is measured by two loading tests. These tests are known as the "sand-bearing" test and the "three-edge bearing" test. The sand-bearing method consists of bedding the pipe evenly in sand for one quarter of the circumference, top and bottom, and applying the test load evenly on the top quadrant. For the three edge-bearing test the pipe is supported on two narrow strips an inch or two apart and the load is applied through a 6 by 6 timber running the full length of the pipe on top and directly above the two supporting strips. See A. S. T. M. Specifications for the required test loads.

In the following discussion the capacity of pipe based on the sand-bearing test will be taken as the standard, or 100 per cent, value. Tests by the sand-bearing method give bearing capacities 50 per cent in excess of those by the three-edge bearing method, and this relation between the tests may be used if needed for converting from one to the other.

Capacity of Pipes in Trenches

The method of laying pipes in trenches affects very materially their supporting strength. The three principal classifications of pipe laying in trenches follow:

"Impermissible" pipe laying is the method that has been altogether too common in the past. It consists of excavating the trench to a flat bottom, as shown in Fig. 4, laying the pipe thereon and placing the backfilling at the sides of the pipe with no attempt at consolidation. The bearing capacity of pipes laid with "impermissible" bedding is 80 per cent of that by the sand-bearing test.

"Ordinary" pipe laying consists of shaping the trench bottom to fit at least one sixth of the pipe barrel (or one half of the pipe breadth), making only local depressions for the bells. The pipe is firmly and evenly bedded on the bottom which has been prepared for it. The backfilling is placed so that it completely fills all voids and is tamped with shovels

only, not rammed, to a height of 6 inches above the pipe. Ordinary pipe laying gives bearing capacities equal to the "sand-bearing" test.

"First-class" pipe laying is obtained by shaping the bottom to fit at least 0.6 of the pipe breadth, and surrounding the remainder of the pipe to at least one foot above its top with granular material carefully placed by hand and under rigid inspection. The material around the pipe is so placed that all voids under and adjacent to the pipe are completely filled, and the material is thoroughly tamped on each side and

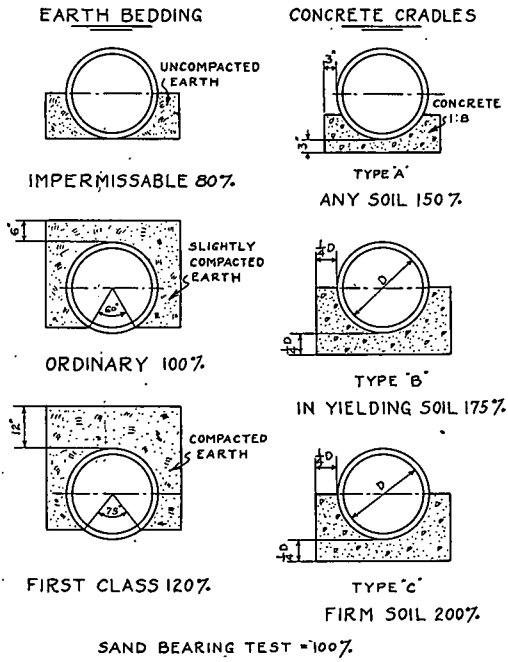


FIG. 4. — CLASSIFICATIONS OF PIPE LAYING IN TRENCHES

under the pipe so far as possible in layers not more than 6 inches thick. The result of "first-class" pipe laying is a bearing capacity 20 per cent greater than that obtained by the sand-bearing test.

It should be noted that for "ordinary" or "first-class" bedding the material placed around the pipe must be placed against the undisturbed sides of the trench, or if sheeting is driven to this level it must be left in place to insure side support. In the case of "impermissible" bedding the sheeting may be withdrawn.

There are many types of concrete cradles that may be used to increase the bearing capacity of conduits. Three general types are indicated in Fig. 4.

Pipes laid with Type A cradles in firm or yielding soils will have a bearing capacity 50 per cent greater than the sand-bearing test.

If the Type B cradle is used in yielding material and with no positive support at the sides of the concrete the capacity of the pipe will be about 75 per cent greater than that of the sand-bearing test.

The Type C cradle is similar to the previous one, except that if used in firm material, and with the concrete at the sides poured against the undisturbed earth walls of the trench, the resulting capacity will be about twice that by the sand-bearing test.

Capacity of Pipes in Embankment

The load on conduits due to embankment is greater than for the same depth of cover in a trench; fortunately, however, the capacity is also greater for pipes in embankment. It is apparent from the previous paragraphs that lateral support increases the capacity, and pipes laid in embankment have an active horizontal load against them.

"Impermissible" projection bedding is that method of bedding with no attempt to shape the foundation surface and no attempt to fill the spaces under and around the pipe with granular material. This type of bedding also includes placing pipes on rock foundation with only a very shallow cushion of earth. For this type of bedding the bearing value is about equal to the "sand-bearing" test.

"Ordinary" projection bedding is that obtained by bedding the pipe with ordinary care in an earth cushion or on earth shaped to fit the barrel of the pipe. In both cases the bedding shall be for at least 10 per cent of the outside over-all height and the remainder of the conduit surrounded by granular material, shovel placed to completely fill all voids. The bearing capacity under the above conditions is 30 per cent in excess of the sand-bearing test.

"First-class" projection bedding is similar to "ordinary," except that the pipe is thoroughly bedded or has firmly compacted material supporting it for at least 30 per cent of the over-all height. Granular material shall surround the remainder of the pipe, being carefully placed to fill all voids. The bearing capacity under these conditions is 60 per cent greater than the sand-bearing value.

Projecting conduits bedded in cradles similar to Type A (Fig. 4) have capacities equal to about twice the sand-bearing value.

The above values for projection bedding are for at least 70 per cent of the height of the pipe projecting above the adjacent ground surface. In cases where the pipe is laid in a shallow trench, such that one half of the pipe or less is projecting, and the width of trench is less than the transition width (discussed under embankment loading), it is oftentimes advantageous to widen the trench beyond the transition width to insure an active lateral pressure against the pipe.

Factor of Safety

It is generally considered good design to use a factor of safety of 1½ for vitrified or unreinforced concrete pipe sewers. There are several important reasons for use of this safety factor. The A. S. T. M. Specifications for vitrified or unreinforced concrete pipes allow some of the test specimens to go as low as 75 per cent of the strength required. On this basis it is obvious that some pipes in the line may have a bearing capacity of only 75 per cent of the expected strength. Another important reason for the use of this factor of safety is that, not infrequently, trenches become wider than anticipated, due to a slip in the bank or large boulders at the edge of trench, or for many other unforeseen reasons. These increases in width make the load on the conduit considerably greater, since the load varies as the square of the trench width. There are other uncertainties, such as the quality of the pipe bedding. It is not always possible to inspect the laying of every pipe, and also the material of the trench bottom may vary, so that instead of "first-class" or "ordinary" bedding the pipe may be laid under conditions that are no more favorable than those for "impermissible" bedding. In view of the foregoing reasons it should be apparent that a factor of safety of 1½ is justified for vitrified or unreinforced pipe.

In the case of reinforced concrete pipe the A. S. T. M. Specifications require that all test pipes show strengths equal to those required. Also the test load on which the design is based is the load to produce a 0.01 inch crack and not the ultimate load. For the above reasons, and since there is less likelihood of these pipes collapsing under an overload, a factor of safety of 1¼ is recommended for reinforced concrete pipe.

Combined Load

Fig. 5 is a diagram which shows the result of combining the live and dead loads on conduits. In this particular case the conditions are as follows: 24-inch tile pipe, trench 47 inches wide, saturated top soil weighing 110 pounds per cubic foot, live load consisting of one rear wheel

of a 20-ton truck (16,000 pounds) plus an allowance of 50 per cent for impact.

The curve for dead load has been obtained by the method previously described for trench loading. The live load curve has been obtained by use of the diagram (Fig. 3). The curve for combined load is the addition of the live and dead loads.

Several things are to be noted in this diagram. The live load becomes almost negligible when the depth of cover exceeds 10 feet, and beyond depths of 6 or 7 feet the dead load is the primary consideration. The combined load has a minimum value which is in the neighborhood

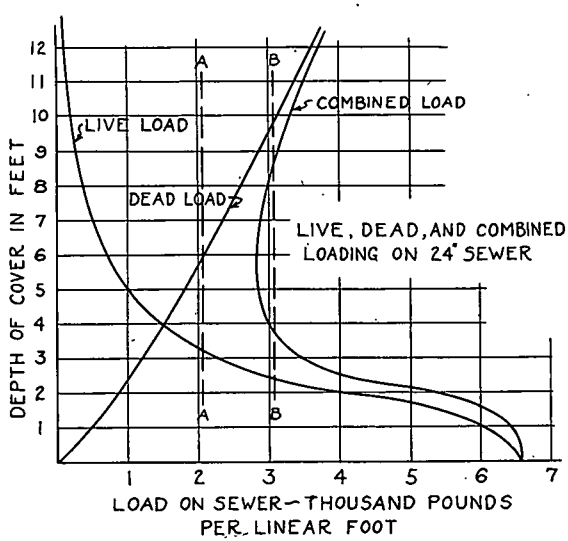


FIG. 5. — LOADING DIAGRAM FOR 24-INCH SEWER

of 2,850 pounds per linear foot, this minimum occurring at a depth of 5 to 6 feet.

If standard vitrified sewer pipe is used, the A. S. T. M. Specifications require a bearing capacity, by the sand-bearing test, equal to 3,070 pounds per linear foot. Assuming that the pipe will be laid with "ordinary" bedding and a factor of safety of 1½, the bearing capacity in place is about 2,000 pounds per linear foot. This value is indicated by the dash lines A—A on the diagram. It is apparent that for the assumptions made there is no depth of cover at which it would be safe to use the pipe under these conditions, since in all cases its capacity falls below

the curve for combined load, and even though "first-class" bedding were used the capacity would still be insufficient.

The conditions indicated in this diagram are nearly identical for all sizes of vitrified pipe down to and including 12-inch. It is for this reason that many engineers have made it a practice in recent years to cradle all vitrified tile pipe 12 inches and larger, particularly when the pipe is to be laid under roadways.

If a cradle of Type A was used, and the factor of safety maintained at 1½, the allowable load would be slightly over 3,000 pounds (dash line B—B) per linear foot, which would be satisfactory for depths of cover between 3½ and 8½ feet. For depths of cover in excess of 8½ feet cradles of Type B or C should be used.

There are two types of reinforced concrete culvert pipe, the standard-strength and the extra-strength. The A. S. T. M. sand-bearing test values for these pipes are respectively 4,500 and 6,000 pounds per linear foot. The standard-strength pipe laid with ordinary bedding and a factor of safety of 1¼ has a capacity of 3,600 pounds. The extra heavy reinforced pipe for the same conditions has a capacity of 4,800 pounds. It is interesting to note that this latter pipe is obviously an attempt on the part of the concrete pipe manufacturers to produce a pipe of this size which will be suitable for a wide range of depths of cover without the need of cradling.

Analysis of Sewer Sections

Concrete sewer sections are usually analyzed by one of two principal methods known as the "elastic theory" and the "method of indeterminate structures," both of which are based on the elastic theory of the arch having fixed ends. These two methods are very fully explained and illustrated in Volume I of "American Sewerage Practice," by Metcalf & Eddy, and for that reason only a few points will be mentioned here. The "elastic method" is somewhat unwieldy, as it requires dividing the arch ring into sections such that the value of $\frac{ds}{I}$ shall be constant. This operation does not lend itself particularly well to sewer sections. It is also a disadvantage to have nearly all of the sections of different lengths.

The "method for indeterminate structures" places no restriction on the length of the divisions, and requires no graphical work preliminary to the analysis. It adapts itself readily to the analysis of sewer sections, and is the method that is used most generally at present.

There are two cases for which analyses are made, depending on foundation conditions. In one case the sewer section is considered as a complete ring and the analysis is made as though it were an arch with the fixed ends meeting at the invert of the section. This type of analysis, for the complete ring, is made when sections are constructed on foundations that are at all yielding.

When the foundation material is rock or some extremely firm material it is considered that the vertical thrust in the side walls goes directly into the foundation and is not distributed across the invert. In this case the analysis is made as though the section were an arch having fixed ends at the bottom of the side walls, the invert being entirely disregarded.

The first consideration in making the analysis is that of the design loads. It was the practice of designers in the past to distribute the vertical load over the entire horizontal projection of the outside of the conduit, and to assume active horizontal pressure on the entire vertical projection. It was customary to assume the intensity of the vertical load to be equal to the actual weight of earth above, plus an allowance for superimposed load. The intensity of the horizontal load was considered to be one third of the vertical intensity.

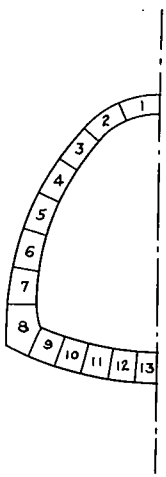
In cases where the sheeting is used as a form for the concrete the above assumption for vertical load is too high, while in cases of relatively wide trenches it may be much too low; also, the distribution of this load over the entire horizontal projection would rarely, if ever, occur. The assumption of active horizontal pressure equal to one third the vertical is also too high in many cases, and will show the conditions of design to be more favorable than they actually are.

As previously discussed, the backfill in a granular state tends to arch between the top quadrant of the conduit and the sides of the trench. For this reason it is suggested that the vertical load be computed by the method for trench loads previously discussed, and that this vertical load be distributed over the equivalent of the top quadrant of the conduit. To this load there should be added any additional loads due to superimposed surface loads.

If the construction conditions are such that thoroughly compacted backfill at the sides of the conduit is assured, or if the concrete is poured directly against undisturbed material forming the sides of the trench, then lateral support may be assumed equal to no more than one third the vertical intensity. Failure to compact the refill, removal of sheeting, or rotting of sheeting may be causes of reduced lateral support.

In some cases designs are made capable of withstanding the vertical loading with or without lateral support.

Some designers assume a distribution of upward pressure on the bottom of conduits such that the pressure is of maximum intensity under the side walls varying uniformly to a lesser amount (sometimes to zero) at the center of the invert. It is the opinion of the author that this should be done only when the foundation material is very firm, such as a hard-packed gravel or equal material which has not been disturbed before the concrete is placed thereon. For all other cases of more yielding founda-



	TOP QUADRANT LOADING		HOR. AND VERT. LOADING	
VOUSSOIR	MOMENT	THRUST	MOMENT	THRUST
CROWN	10,000	2,340	2,300	5,400
1	-10,200	3,300	2,800	5,700
2	4,800	7,100	1,650	7,100
3	-1,600	10,400	700	8,400
4	-8,100	11,000	-1,200	9,500
5	-12,300	11,400	-3,800	10,700
6	-14,000	11,500	-6,600	11,700
7	-13,450	11,400	-9,300	12,300
8	-9,300	9,700	-11,300	10,800
9	2,250	500	-3,400	5,600
10	10,500	700*	4,000	4,500
11	16,900	1,500*	9,800	3,600
12	21,200	2,100*	13,700	3,100
13	23,400	2,300*	15,700	2,800

*TENSION

FIG. 6. — COMPARISON OF ANALYSES FOR DIFFERENT DISTRIBUTIONS OF LOADING

tion material the upward load should be distributed uniformly across the base.

To illustrate the effect of different types of loading Fig. 6 has been prepared. In this figure the results of two different analyses are shown. In both cases the analyses were made of a 10-foot semi-elliptical sewer having 19 feet of earth cover. In the case of the "top quadrant" loading the vertical load has been assumed to be distributed over six voussoirs at the crown and no lateral support assumed. For the analysis using "horizontal and vertical" loading the same total vertical load was assumed but distributed over the entire horizontal projection. A hori-

zontal loading on the entire vertical projection was assumed, the intensity of which was equal to one third the intensity of the vertical load. In both cases the upward load was uniformly distributed on the base. The moment is in foot pounds, and if positive causes tension on the inside of the section; the thrust is in pounds.

There are several important things to notice. In general it is apparent that the "top quadrant" loading gives more severe conditions, particularly at the crown and at the invert. It is also apparent that the reversal of stress occurs at somewhat different points for the two analyses.

The critical points to be watched in the design of most sections are the crown, the point at which the reversal of stress occurs, the lower end of the side wall just above the invert, the invert close to the side wall and also at the center. These points are usually the most important to consider. The invert at the side wall will have high shear and consequently high bond stresses. The other points, with the exception of the point of reversal of stress, will have bending and direct stresses which will require investigation to determine the unit stresses.

Formulas for Analysis of Semi-elliptical Sections

The standard semi-elliptical section developed by Metcalf & Eddy, and appearing in "American Sewerage Practice," Volume I, is so dimensioned that all of the dimensions can be expressed in terms of the diameter. For this reason it was possible to develop formulas for analysis, the procedure being simply to generalize an analysis, all dimensions being left in terms of the diameter. Two sets of formulas have been developed by the author for such analyses. One set of these formulas appearing in Metcalf & Eddy's book mentioned above is for horizontal and vertical loading. The other set of formulas, appearing in the February 13, 1930, copy of "Engineering News," is for vertical loading only, the vertical load being distributed on the equivalent of the top quadrant. In some cases a designer may choose to investigate for both types of loading and then design for the most severe, or, using his judgment, interpolate for some condition between the two. After determining the extent of the load it is only a matter of slide rule calculation to obtain the moment thrust and shear at any point, using these formulas.

Box Culverts

In Fig. 7 there is a diagram which the author has found convenient for designing or investigating the design of box culverts. From this diagram it is possible to obtain the moment factors for moment at the

center of the top and bottom slabs, and at the corners. These factors appearing on the vertical scale are the values which are used in the denominator of such expressions as $\frac{wL^2}{8}$ for the center moment of a simple beam, or $\frac{wL^2}{12}$ for the center or end moment of a single span restrained beam. These factors are on the basis of absolute continuity in the culvert; the three sets of curves are for three relative values of moment of inertia for the horizontal and vertical portions of the culvert. A little study will make clear the basis of the diagram. It will be noted that the

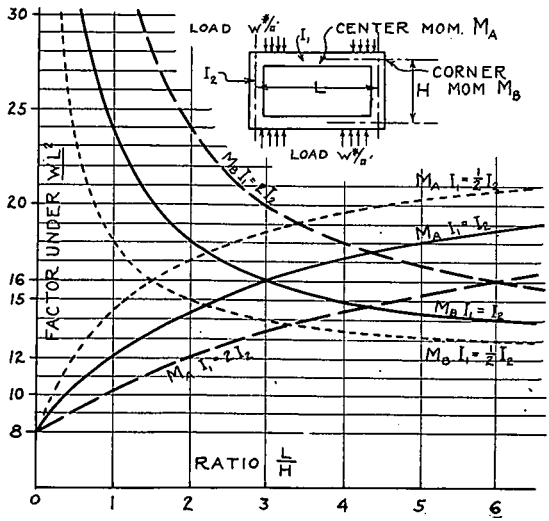


FIG. 7. — MOMENT FACTORS FOR BOX CULVERT DESIGN

three curves for the center moment are approaching the value of $\frac{wL^2}{24}$, while those for the corner moment approach $\frac{wL^2}{12}$ as $\frac{L}{H}$ increases.

The values given by this diagram are for the *theoretical* moments based on *absolute continuity*, and should not be taken too literally. Just as the Joint Committee requires the use of $\frac{wL^2}{12}$ at center and ends of a single restrained span, so also these moments should be increased somewhat for design use to take care of unsymmetrical loading or imperfect continuity. In no case should moments any less than $\frac{wL^2}{24}$ be used.

This diagram gives information for obtaining the moments in the top and bottom slabs due to vertical loading only. If horizontal loading is expected the moments may be calculated by rotating the culvert on the diagram 90 degrees and proceeding as though for vertical loading. When combining the results it should be remembered that the corner moments are negative and are added, while the center moments are of opposite sign and are added algebraically.

Sewer Sections

For sewers less than 5 feet in diameter the common practice is to use circular sections. These sections have the best hydraulic properties, the forms are simple, and they are as economical to use as any shape.

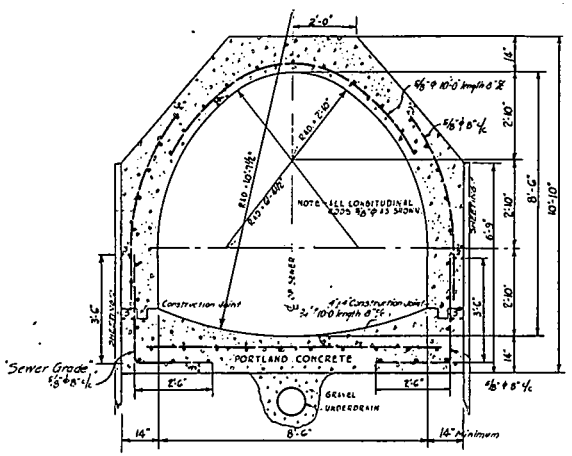


Fig. 8. — North Metropolitan District Relief Sewer

When the size exceeds 5 feet it is generally advantageous to use a section similar to the so-called semi-elliptical. The city of Chicago developed a sewer section of the semi-elliptical type in which the width was 0.9 of the height. This section has been used for much of the work built in tunnel. Making the width somewhat less than the height gives less severe stresses, particularly in the invert.

Fig. 8 shows the construction drawing of an 8′ 6″ semi-elliptical sewer now under construction in Medford as a part of the North Metropolitan District Relief Sewer. It is patterned after the Chicago sections and illustrates a recent trend toward the use of steel in only one face of

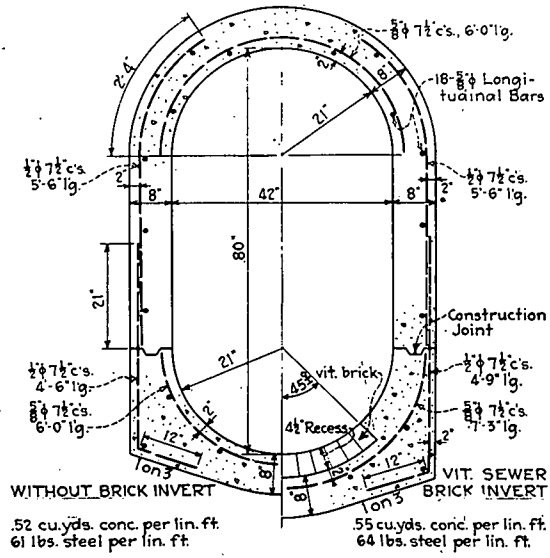


FIG. 9. — LINK SECTION

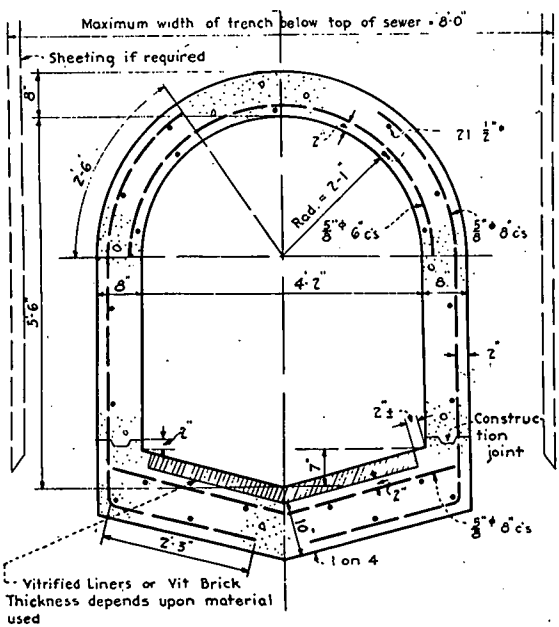


FIG. 10. — SEMI-LINK SECTION

concrete. This practice simplifies construction but requires thicker concrete to offset the absence of steel in the compression face. Sections having steel in only one face are not as capable of resisting unusual stresses due to unsymmetrical loading or unequal settlement.

Figs. 9 and 10 are two sections that have been used recently by Metcalf & Eddy at Springfield, Massachusetts. The former is called the Link Section, because of its shape, and the latter the Semi-Link, resembling one half of the previous section.

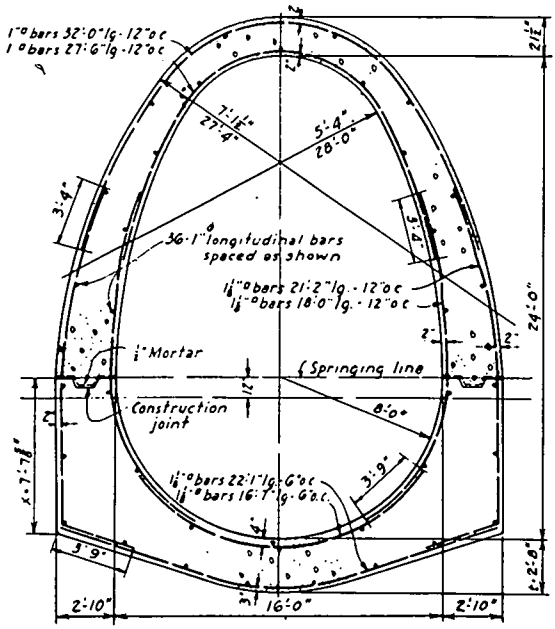


FIG. 11. — INVERTED EGG

It has become increasingly difficult to find room in streets for the construction of relief sewers. The above sections have the advantage of requiring narrow trenches, and are also economical of material. In some cases the reduction in total excavation may more than offset any increase in the material required.

The forms for both of these sections are simple to construct. For very low flows the hydraulics are somewhat better for the Link Section than for the Semi-Link, also the invert of this section is very strong. On the other hand, the invert of the Semi-Link Section has the advantage of ease in placing, for it requires no top form.

The box culvert type of section is often used in shallow cuts, particularly where large streams are to be conducted underground. It is simple to build and lends itself readily to multiple use. There are many other special types of sections which are advantageous for unusual conditions, but probably those derived from the semi-elliptical section are most popular for the larger sizes.

The city of Louisville, Kentucky, has recently built a 16 foot by 24 foot relief sewer in the shape of an inverted egg. This section is of interest because it is so unusual for such a large sewer. It is an extremely strong section capable of withstanding an unusually heavy load, and well adapted, both hydraulically and structurally, to the conditions encountered.

Conclusion

This paper is not intended to be a complete treatise on sewer design, but it is hoped that it will prove valuable as a ready source of reference for designers. Much of the information contained herein can now be found only by referring to a group of publications. The author has attempted to digest the literature on the subject, combine it with his own experience, and condense the whole into the present publication.

The sewer sections included are less familiar types and illustrate a few of the recent designs.

CHEMICAL PRECIPITATION OF SEWAGE AND INDUSTRIAL WASTES

By Almon L. Fales, Member *

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers held on April 1, 1936)

The American Public Health Association's Committee on Sewage Disposal presented a very comprehensive report on the "Chemical Treatment of Sewage" at the meeting of the Public Health Engineering Section at Milwaukee on October 8, 1935, and that report was published in the November issue of the "Sewage Works Journal." However, it may be of interest to summarize the history, recent developments and present status of the chemical precipitation of sewage and industrial wastes.

Scope of Paper

Chemical treatment of sewage in a broad sense includes the application of chlorine or chloride of lime, but chlorination is for the purpose of disinfection, odor control or other improvement, but not for chemical precipitation except as the chlorine is used with some other chemical for the production of the precipitant, as, for example, with copperas, ferrous iron sulphate, for the production of chlorinated copperas, which is a mixture of ferric iron chloride and ferric iron sulphate.

In a broad sense, the chemical treatment of sewage may also be taken to include the addition of chemicals to the sludge from any process of sewage treatment, for the purpose of conditioning the sludge for dewatering and disposal. This treatment is not a part of the process of chemical precipitation of sewage except as it applies to the disposal of the sludge from chemical precipitation.

This paper will deal only with the process of chemical precipitation of sewage and industrial wastes, with some modifications and supplementary treatments that have been proposed. According to the "Definition of Terms Used in Sewerage and Sewage Disposal Practice" adopted by the American Society of Civil Engineers and the American Public Health Association, Chemical Precipitation is "sedimentation accelerated

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by the coagulation of suspended or colloidal matter through the addition of chemicals." In this paper it will be feasible to deal with this subject only in a general way. The paper will utilize the report of the American Public Health Association's Committee on Sewage Disposal and other sources of information on the subject, as well as the personal experience of the writer.

History of Chemical Precipitation

During the latter half of the nineteenth century, numerous European patents involving the chemical precipitation of sewage were issued, particularly in Great Britain.

Chemicals Used. — Many different kinds of chemicals and combinations of chemicals were proposed. These included —

Lime in the form of milk-of-lime.
Lime and ferrous iron sulphate, commonly called copperas or green vitriol.
Lime and chloride of iron.
Lime and clay (marl was also proposed for use with the precipitant to facilitate sedimentation).
Ferric iron chloride.
Ferric iron sulphate.
Sulphate of aluminum.
Calcium phosphates.
Alumino-ferric, a mixture of sulphate of aluminum and ferric sulphate.
Alumino-ferric, blood, charcoal and clay, the so-called "ABC Process."
Alum, clay and sulphuric acid, commonly called vitriol.
A chemical precipitant with fibrous matter, such as wood pulp or paper pulp.
And many others.

Allied Processes. — There were numerous processes allied with chemical precipitation. The Webster Process called for electrolysis with iron plates from which the iron was gradually dissolved and precipitated. Certain patents involved air mixing and re-use of sediment. Various patented processes called for the re-use of the precipitant. Spence's Magnetic System called for the use of carbide and magnetic oxide of iron filters.

Claims for Chemical Precipitation. — It was claimed that by chemical precipitation, the putrefactive substances in sewage would be removed, and that the precipitated matter could be sold as a fertilizer which would pay the cost of treatment and give a substantial profit. Chemical precipitation was hailed as the process which would solve the sewage disposal problem, and between 1880 and 1890 nearly all the sewage treatment plants constructed in Great Britain were chemical precipitation plants.

The hope of financial gain was not realized because the precipitate, although containing fertilizing substances, also contained so much water, averaging about 94 per cent, that it was found impossible to prepare a marketable product at a cost as low as the price of other chemical fertilizers. It was also gradually recognized that although the process produced a clear effluent, it still contained a large amount of soluble organic matter and was putrescible. The treatment proved to be very expensive and the disposal of the large quantities of sludge that were produced constituted a serious problem, even after sludge presses were introduced.

Chemical Precipitation Superseded by Biological Treatment. — Chemical precipitation became very largely superseded by biological treatment, although chemical precipitation is still in use at several sewage treatment plants in Great Britain, principally as a preliminary treatment for biological filters.

History of Chemical Precipitation in the United States. — Classic experiments on the chemical precipitation of sewage were made by Hazen and his successors at the Lawrence Experiment Station of the Massachusetts State Board of Health prior to the year 1890, and the results were published in the Reports of the Board. About the year 1890, when chemical precipitation was at its height in Great Britain, several chemical precipitation plants were built in the United States. Nearly all of these had been abandoned before 1910. The plants at Worcester and Providence were notable exceptions. The Worcester chemical precipitation plant was superseded by an Imhoff tank-trickling filter plant in 1925, and that at Providence by an activated sludge plant in 1934.

Experience at Worcester. — It may be of interest to discuss briefly the Worcester chemical precipitation experience, with which the writer was familiar.

The selection of chemical precipitation at Worcester was due primarily to the presence in the sewage of large quantities of sulphate of iron from wire mills and foundries resulting from the dipping of the metal in sulphuric acid to remove the scale. Milk-of-lime formed by slaking of quicklime was added to the sewage in sufficient quantity to precipitate the iron as hydroxide, which was the coagulant largely depended upon for the clarification of the sewage. When little or no sulphate of iron was present, the calcium carbonate formed acted as the coagulant. The application of milk-of-lime was controlled by maintaining the proper alkalinity, or *pH*, as shown by a pink color on throwing into the treated sewage at frequent intervals a few drops of phenolphthalein from a suitable bottle.

The milk-of-lime added was thoroughly and quickly mixed with the sewage by causing the treated sewage to flow rapidly down the mixing channel, which was provided with side baffles in the form of a fish ladder. The treated sewage was then passed through "roughing," or coagulating, tanks, and thence through a channel to the "finishing," or final settling, tanks, which were operated in parallel. The effluent, even though clear, was putrescible, as shown by blackening and foul odors on standing in tightly stoppered bottles in a warm room.

The sludge from chemical precipitation was conditioned with milk-of-lime, averaging about 50 pounds of lime per thousand gallons of sludge, and filter pressed, the pressed cake being dumped into cars and hauled by motor car to a low area of waste land, where it was dumped from a trestle, a small proportion being taken away by farmers in the vicinity.

The quantity of lime required at Worcester varied widely during the day and from day to day and year to year. In 1893 the quantity of lime averaged 1,233 pounds per million gallons of sewage treated. In 1908 it averaged only 871 pounds per million gallons, due to a widespread depression in business and a corresponding reduction in the quantities of acid-iron wastes discharged into the sewers.

Chemical precipitation at Worcester during the period of ten years from 1903 to 1912 effected an average removal of 42.4 per cent of the total organic matter, and 82.0 per cent of the suspended organic matter, as measured by albuminoid ammonia. For these years the greatest annual average removal of organic suspended matter was 93.1 per cent and the lowest, 74.7 per cent. However, after 1900 all the sewage was not treated chemically, the strongest portions requiring the most complete treatment being subjected to plain sedimentation and intermittent sand filtration up to the capacity of the filter bed area available.

The quantity of sludge produced averaged between 5,000 and 10,000 gallons per million gallons of sewage, and contained 5 to 10 per cent solids. The sludge cake contained between 25 and 30 per cent solids on the average, and the quantity of sludge solids averaged between 1 and 2 tons per million gallons of sewage treated.

The cost of chemical precipitation averaged about $5 per million gallons, and the cost of sludge disposal about the same amount, making the total approximately $10 per million gallons.

Chemical precipitation alone became inadequate treatment to meet the requirements for an effluent to be discharged into the Blackstone River. Consequently, a program of construction of intermittent sand filter beds was begun in 1898 and continued until 1910, when

about 75 acres of filter beds were available. This area was less than one third of the area that would be required to filter all the sewage. Chemical precipitation was tried preliminary to filtration, but its advantage was not at all commensurate with the cost. There were disadvantages, including the penetration of the fine suspended solids in the effluent into the sand, and the formation of a hard crust on the surface of the beds due to the lime.

At about this time plant scale tests with the Imhoff tank trickling filter method of sewage treatment clearly demonstrated that this treatment would be much more economical. Consequently, an Imhoff tank-trickling filter plant was constructed and put in operation in 1925, as previously stated.

Chemical Precipitation of Industrial Wastes. — From 1910 to 1928 there was little interest in the chemical precipitation of sewage, although the use of chemicals for treating industrial wastes continued to receive consideration and chemical precipitation was adopted in cases where it was found advantageous. Chemical precipitation of some industrial wastes, including those from certain paper mills, tanneries, woolen mills, laundries, dairies, canneries, etc., has been continued, at least during periods of warm weather and low stream flow. Wool scouring wastes have been treated with sulphuric acid by the so-called acid-cracking process for many years. In the case of some industrial wastes, such as distillery wastes, high in dissolved organic matter, chemical precipitation has been found to accomplish very little and to be prohibitively expensive. Many organic wastes can be treated much more economically by biological processes. Chemical precipitation is advantageous as preliminary treatment in some cases.

Landreth or "Direct Oxidation" Process. — The Landreth electrolytic process, or so-called "direct oxidation" process, which was first tried out experimentally between 1914 and 1920, involves the addition of sufficient lime to produce a caustic or hydroxide alkalinity for maintaining passivity of the electrodes. Several small plants for this process were built, but have since been abandoned. Tests of the process proved that the results accomplished were due, not to the electrolytic treatment, but to chemical precipitation and disinfection by the lime treatment. The process was found to be much more expensive and no more effective than chemical precipitation. The only plant of this type now in operation so far as could be ascertained is that at Winston-Salem, North Carolina, and this is no longer called a "direct oxidation" plant.

Recent Developments

Since 1929 or 1930 various processes involving chemical precipitation have been brought out. Most of these are still in the experimental stage with insufficient plant-scale operating, analytical and cost data to appraise them properly. Chemical precipitation experiments have been made at several existing sewage treatment plants, looking to improved efficiency. In this paper it will be feasible only to discuss some of the processes for which sewage treatment plants have been constructed.

Developments in Chemical Coagulation. — There have been developments in chemical coagulation tending to improve the efficiency and reduce the cost of chemical precipitation, and some of the old ideas have been revived. The more important developments along these lines, as listed in the American Public Health Association's Committee report, are as follows:

1. The adjustment of the hydrogen ion concentration for optimum coagulation.
2. The development of devices for controlling the *pH*.
3. The introduction of stirring by air or mechanical means to promote floc formation prior to sedimentation.
4. Improved equipment of various types, including:
   (*a*) Tank equipment.
   (*b*) Chemical feed:
      (1) Dry.
      (2) Mechanical control.
5. The decrease in the cost of the ferric salts which are the more efficient, such as ferric chloride or ferric sulphate.
6. The availability of liquid chlorine at greatly decreased cost.
7. The general reduction in the cost of chemicals.
8. Improved facilities for shipping and handling corrosive iron salts.
9. The availability of improved vacuum filters for de watering sludge.

Among other and older ideas which have been revived of late are:

1. The introduction of inert material to weight the sludge.
2. The return of the precipitated sludge to the raw sewage.
3. The use of air for mixing.
4. The splitting of the procedure into two stages, utilizing a preliminary settling followed by coagulation and precipitation.

The Laughlin Process. — The Laughlin process of sewage treatment, adopted at Dearborn, Michigan, derived its name from W. C. Laughlin, whose magnetic filter has been associated with the process throughout its development.

During the first stage of development of the Laughlin process,

emphasis was placed on the coagulation using ferric chloride, spent wet lime from calcium carbide used in the manufacture of acetylene, and paper pulp. Ferric chloride was at first manufactured from scrap iron and chlorine in an absorption tower, but difficulties in operation of this process have led to its abandonment and to the use of purchased ferric chloride solution. Ferric sulphate has been used at times instead of ferric chloride. Paper pulp, first considered a waste product free of cost except for the handling, was later found to be an item of considerable expense when it had to be purchased, and consequently the quantity of paper pulp used has been reduced, and, according to recent information, has been discontinued. Spent lime from acetylene manufacture has not always been available, and at such times lime has been purchased.

Recently attention has been diverted from the coagulation process and the emphasis placed on the Laughlin Magnetite Filter, which is offered by the Filtration Equipment Corporation for use in all kinds of sewage treatment plants for improvement of the clarification accomplished by sedimentation tanks.

The Dearborn plant has two 60-foot diameter tanks, one of which is being used for the chemical treatment of sewage from the West Side, designed for a one-hour sedimentation period at a flow of 4,000,000 gallons per day. A Laughlin Magnetite Filter 8 feet wide and about 1 foot deep is constructed around the wall of the tank on the inside at the top. The bottom of the filter is a slotted screen of phosphor bronze with openings 1 millimeter wide. A 3-inch layer of magnetite sand graded between screens of 10 and 16 mesh, rests on the screen and the tank effluent flows upward through this filter. The magnetite sand is cleaned by a traveling peripheral electromagnet which is intermittently energized, thus lifting the magnetite sand beneath it and permitting a rush of water through the sand. The wash water is removed by a pump mounted on the traveling device and is returned to the incoming treated sewage. Provision is made for chlorinating the effluent. The sludge produced by the chemical precipitation of the sewage, together with the sludge from sedimentation at the East Side plant, is conditioned with lime and ferric chloride and de-watered by vacuum filters. The filter cake was dumped on land near by until 1935, since which time it has been incinerated.

The report of operation for the period from November, 1933, to April, 1934, showed an average of 1.74 million gallons of sewage treated per day, with an average of 306 pounds of ferric chloride, 147 pounds of lime, and 73 pounds of paper per million gallons of sewage. By this treatment, including the Magnetite Filter, the suspended solids were

reduced from 230 parts per million in the sewage to 19 parts per million in the effluent, a reduction of 92 per cent, and the five-day biochemical oxygen demand was reduced from 128 parts per million in the sewage to 28 parts per million in the effluent, a reduction of 78 per cent.

For sludge conditioning prior to de-watering, 82 pounds of lime and 19 pounds of ferric chloride were used per 1,000 gallons of sludge. Sludge cake of an average moisture content of 67.2 per cent was produced at the rate of 7.5 pounds of dry solids per hour per square foot of filter area.

The costs of this sewage treatment cannot be satisfactorily determined on account of the inclusion of the sludge from the East Side plant.

A Laughlin plant has been built for Perth Amboy, New Jersey, and this has been in operation for several months. At this plant it was proposed to use chemical precipitation only during the summer season.

The Guggenheim Process. — Experimental work conducted by the Research Department of Guggenheim Brothers led to the development of a process of sewage treatment involving —

Chemical precipitation with ferric sulphate and lime.

Disposal of the sludge by de-watering with vacuum filters and incineration with recovery of the iron in the ash by leaching with sulphuric acid.

Removal of soluble nitrogenous compounds in the chemical effluent by filtration through a bed of zeolite, regeneration of the zeolite from time to time by means of salt solution and distillation, and recovery of ammonia from the spent salt solution.

This process was tried out at an experimental plant at the North Side sewage treatment works of the Sanitary District of Chicago in 1932 to 1933.

This process, including the zeolite filter, gave complete sewage treatment comparable with the activated sludge process, but was quite com plicated. The process was subsequently modified by eliminating the zeolite filter and recovery of ammonia and abandoning the idea of recovering the iron from the incinerated sludge. The latest process includes the continuous return to the sewage of a part of the sludge from chemical precipitation, adding ferrous or ferric sulphate solution, mixing with air agitation, and aeration for approximately one hour, followed by sedimentation with a detention period of about one and one half hours. It is proposed to condition and de water the sludge by vacuum filters and incinerate the sludge cake, or otherwise dispose of it.

This process was developed from experiments at Raleigh, North Carolina, Dyckman Street, New York City, and New Britain, Connecticut. A plant designed for this process is now under construction at New Britain.

Process in Use at Shades Valley Plant in Birmingham, Alabama. — The Shades Valley sewage treatment plant in Birmingham, Alabama, put in operation something over a year ago, provides for chemical precipitation during the summer and plain sedimentation during the remainder of the year, the sludge being digested in two-stage digestion tanks and de-watered on open sludge drying beds. Chlorinated copperas is used as the coagulant, the copperas or ferrous sulphate being obtained from the local steel mills, and the chlorine purchased in 1-ton containers. An excess of chlorine is used for disinfection. The sewage is very weak.

This plant now serves a population of about 3,500, with a sewage flow of about 1,000,000 gallons per day, but is designed for a population of 15,000 and a sewage flow of 2,000,000 gallons per day. The plant includes a 5 foot x 5 foot flash mixer with variable speed impeller; a 22 foot x 21 foot x 17½ foot flocculator divided into two units equipped with three rows of paddles and providing a detention period of thirty minutes at a flow of 2,000,000 gallons per day; and two clarifiers, each 45 feet in diameter by 7 foot side wall water depth, and each providing a one-hour detention period at a flow of 2,000,000 gallons per day.

The plant as designed is very flexible, permitting any one of several different methods of operation. The method adopted for normal operation with chemical treatment involves series operation of the clarifiers with chemical precipitation of the settled sewage. The sewage flows through the primary clarifier, flash mixer, flocculation tank and secondary clarifier with the return of sludge to the sewage. Sedimentation prior to chemical precipitation and digestion of the chemically treated sewage sludge are covered by the Ornstein patent, owned by the Dorr Company, which furnished most of the equipment.

Results published* for the period of chemical treatment from June 15 to November 19, 1935, show a reduction in average five-day biochemical oxygen demand from 85.5 parts per million in the raw sewage to 12 parts per million in the effluent, and a reduction in average suspended solids from 124 parts per million in the sewage to 10 parts per million in the effluent. These results were accomplished with an average of 480 pounds of copperas and 70 pounds of chlorine per million gallons of sewage treated. The cost of chemicals for precipitation averaged $5.70 per million gallons of sewage treated, based on copperas delivered at $15 per ton and chlorine at $0.03 per pound. The total cost of operation of the treatment plant during the summer months when chemical treatment was employed was reported to be $17.10 per million gallons of sewage treated.

* "Sewage Works Journal," March, 1936, Vol. 8, p. 233.

It has been found that the period of detention of one hour on which the design of the settling tanks was based is insufficient, making it necessary to operate the tanks in parallel at high flows.

Contrary to a conclusion published earlier, it has been found that sedimentation prior to the addition of chemicals does not reduce the quantity of chemicals required. However, it is claimed that pre-sedimentation has operating advantages, including the return of the sludge from the chemical treatment of the settled sewage to the raw sewage. It is claimed that the return of this sludge is effective for odor control, but it is pointed out that care must be exercised to add sufficient chlorine to provide free chlorine throughout the plant. It is also claimed that the return sludge increases the efficiency of sedimentation and produces a more uniform and denser sludge to be discharged to the sludge digestion tanks. No data are presented on the digestion of the sludge from chemical precipitation.

Process in Use at Coney Island Plant, New York City. — In July of last year a sewage treatment plant was put in operation at Coney Island, New York City, to be operated as a chemical precipitation plant during the summer, using lime and ferric sulphate for precipitation and chlorine for disinfection during the bathing season, and as a plain sedimentation plant during the winter.

This plant has a nominal capacity of 35,000,000 gallons per day, including four mechanically cleaned circular sedimentation tanks 90 feet in diameter, providing a two-hour detention period. Chemicals are to be handled by pneumatic conveyors to storage tanks and thence to the charging hoppers with five dry feed machines, each having a capacity of 500 pounds per hour of lime and 1,500 pounds per hour of ferric sulphate or ferrous sulphate. Four chlorine machines are provided, each with a maximum capacity of 2,000 pounds per 24 hours. Rapid mixing devices and mechanical flocculators are provided, giving a flocculation period of 11.5 minutes. There are eight sludge digestion tanks, having a total capacity of 478,000 cubic feet. Gas holder-type of covers gives a total gas storage capacity of about 200,000 cubic feet. One vacuum filter is to be installed at the outset. The estimated cost of the present installation was $1,823,000, including the effluent outfall, wharf, bulkheads, fill and force mains, which totaled $666,000.

Treatment Adopted for Minneapolis–St. Paul. — At Minneapolis–St. Paul, sedimentation of the sewage is to be provided with provision for chemical precipitation if and when necessary. The flow of the Mississippi River is indicated as the controlling factor, inasmuch as in many years sedimentation alone might suffice, and chemical flocculation prior

to sedimentation with chlorination of the effluent would be practiced at such times as necessary to bring the treatment up to 70 per cent reduction in the five-day biochemical oxygen demand.

Present Status of Chemical Precipitation in the United States

Number of Plants Using Chemical Precipitation. — According to the American Public Health Association's Committee report, of 1,228 sewage treatment plants in the United States in cities of over 2,500 population in operation in 1934, there were 5 plants using chemical precipitation; of 42 plants under construction, 1 was a chemical precipitation plant; and of 123 plants being designed, 5 were for chemical precipitation. These chemical precipitation plants did not include those for seasonal use of chemical precipitation, which is coming to be the plan considered. Including all sewage treatment plants in the United States in 1935, large and small, using chemical precipitation for seasonal treatment, preliminary treatment, or for any other purpose, there were 35 plants in operation, 9 plants under construction, and 18 plants proposed.

Efficiency of Chemical Precipitation. — The consensus of opinion of unbiased sanitary engineers is that very little, if any, of the dissolved organic matter in sewage can be removed by the addition of chemicals, and that it is practically impossible to remove all of the colloidal matter. Because of this limitation, chemical precipitation is inferior to biological oxidation in the removal of biochemical oxygen demand, organic nitrogen, organic carbon and ammonia compounds. The degree of efficiency of chemical precipitation will depend upon the kind and extent of chemical precipitation employed.

Disposal of Sludge from Chemical Precipitation. — Chemical precipitation results in an increased weight of sludge solids over that produced by other processes in which no chemicals are added to the sewage. The use of aids to precipitation, such as lignite, marl, cement dust and clay, correspondingly increases the quantity of sludge solids to be disposed of. The volume of sludge and quantity of sludge solids depend upon the kind and extent of chemical precipitation employed.

There is no question that sludge from chemical precipitation of sewage can be satisfactorily de-watered by vacuum filters if sufficient lime or other chemicals is present. Such sludge is more or less offensive in odor if not previously digested.

With moderate quantities of chemicals used for precipitation, the indications are that the sludge can be digested satisfactorily if sufficient digestion capacity is provided for the volume of sludge produced by

chemical precipitation. If large quantities of lime are used, sludge digestion will be retarded.

Cost of Chemical Precipitation. — The cost of chemical precipitation will naturally vary according to the kind and extent of chemical precipitation employed. If chemical precipitation is employed for a high degree of efficiency throughout the year, the annual cost will be much greater than that of biological oxidation processes, which give higher efficiencies. If such chemical precipitation is operated only for two or three months each year, if and when such treatment will suffice, the annual cost of chemical precipitation may be less than that of biological treatment for six months or more.

Chemical precipitation is characterized by low construction cost and high operating cost as compared with biological oxidation processes. The cost of chemicals is such a large proportion of the cost of chemical precipitation that changes in the market price of chemicals used may greatly affect the cost of the treatment. The high cost of chemicals for chemical precipitation in the past has frequently caused a curtailment in the quantity of chemicals used, resulting in a lower efficiency of the treatment.

Conclusions. — Among the conclusions reached by the American Public Health Association's Committee are the following:

Chemical treatment usually accomplishes a degree of treatment between plain sedimentation and biological processes.

The seasonal or occasional use of chemicals is attracting favorable notice in situations where sedimentation alone may suffice for the greater part of the year or for many years out of a cycle.

The use of chemicals as a preparatory treatment in addition to sedimentation ahead of biological filters is practiced in many situations, and prolongs the useful life of existing plants, otherwise overloaded.

Of the various processes for chemical treatment of sewage considered in the United States, the use of iron salts. particularly ferric chloride or ferric sulphate, seems most favored.

The manufacture of iron salts from scrap iron or tin cans with chlorine has been favored in many situations for reasons of economy.

The comparison of chemical treatment of sewage with other procedures should be studied carefully from the standpoint of adequacy, performance and cost.

The use of chemical treatment in handling industrial wastes appears to have a definite place, where performance and cost justify the procedure as compared with other methods.

For greater detail in regard to chemical precipitation history and recent developments, reference is made to the American Public Health Association's Committee report published in the "Sewage Works Journal" for November, 1935.

THE MARCH, 1936, FLOODS IN NEW ENGLAND

A Symposium, presenting briefly an Outline of the Flood Conditions on Certain of the New England Rivers

FLOOD FLOWS IN NEW ENGLAND DURING THE MARCH, 1936, FLOOD

By H. B. Kinnison, Member*

The great flood of March, 1936, presented an unusually favorable opportunity for the study of a severe flood while it was in progress. Many organizations, both permanent and temporary, public and private, were prepared to collect data and were able to accomplish a great quantity of field work before the flood receded, the most favorable time to obtain much of the flood data, especially that relating to flood discharge.

In co-operation with the various New England States, the Water Resources Branch of the United States Geological Survey was operating about 150 gauging stations on the New England rivers at the time of the flood. In some instances where the water was extraordinarily high, the gauging station structures were submerged, but in general the automatically recording instruments provided an excellent record of the flood at each gauging station.

The river investigations of the Branch are concerned principally with determination of stage and discharge. In this connection, the development of a discharge rating curve for each gauging station throughout the full range of stage is of primary concern. Due to the timely warning of the storm's approach, our full force was in the field before the flood arrived. Each engineer was equipped with complete current meter equipment of the most modern type, which included 50-pound, 75-pound, or 100-pound lead weights with specially constructed cable to support the weight and current meter, and a reel to assist in lowering and raising the equipment in obtaining velocity observations in the water.

* District Engineer, Water Resources Branch, United States Geological Survey, Post Office Building, Boston, Mass.

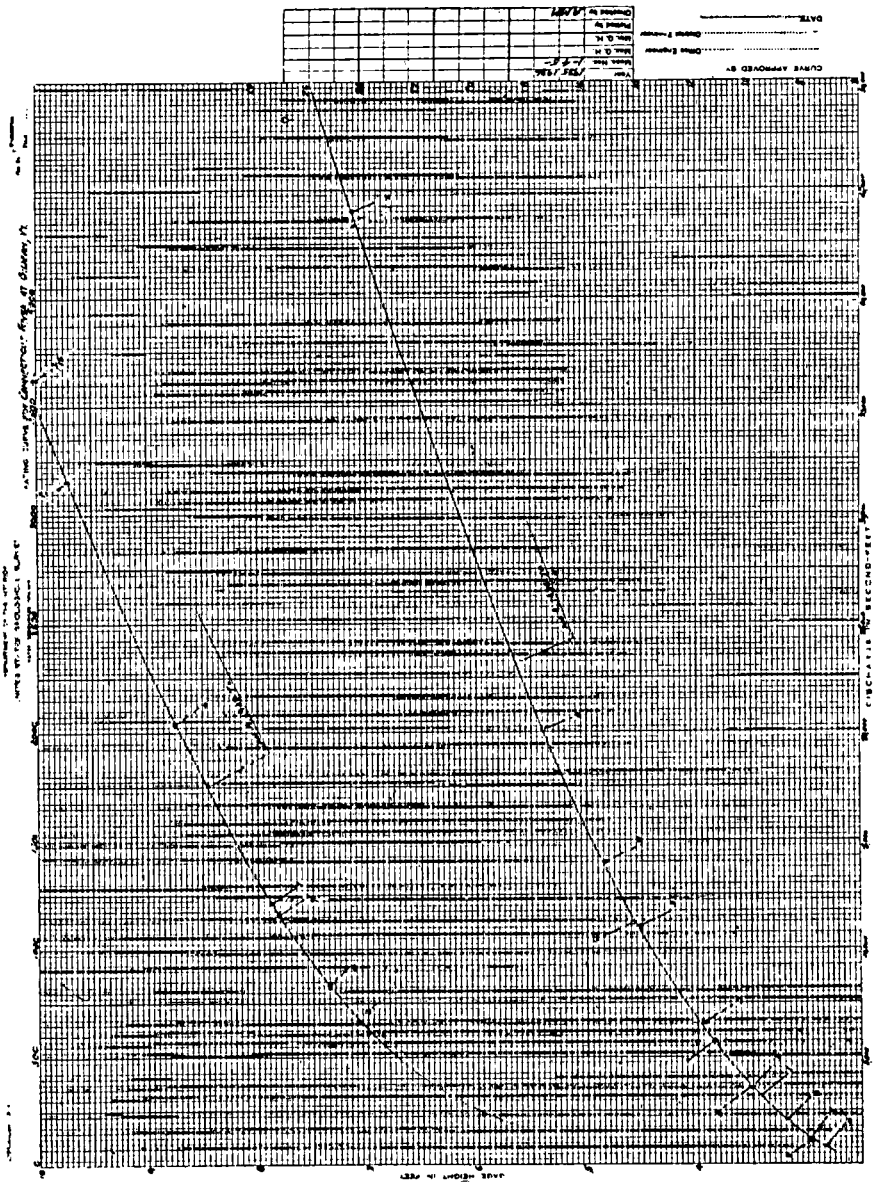

FIG. 1.—RATING CURVE OF CONNECTICUT RIVER AT GILMAN, VT.

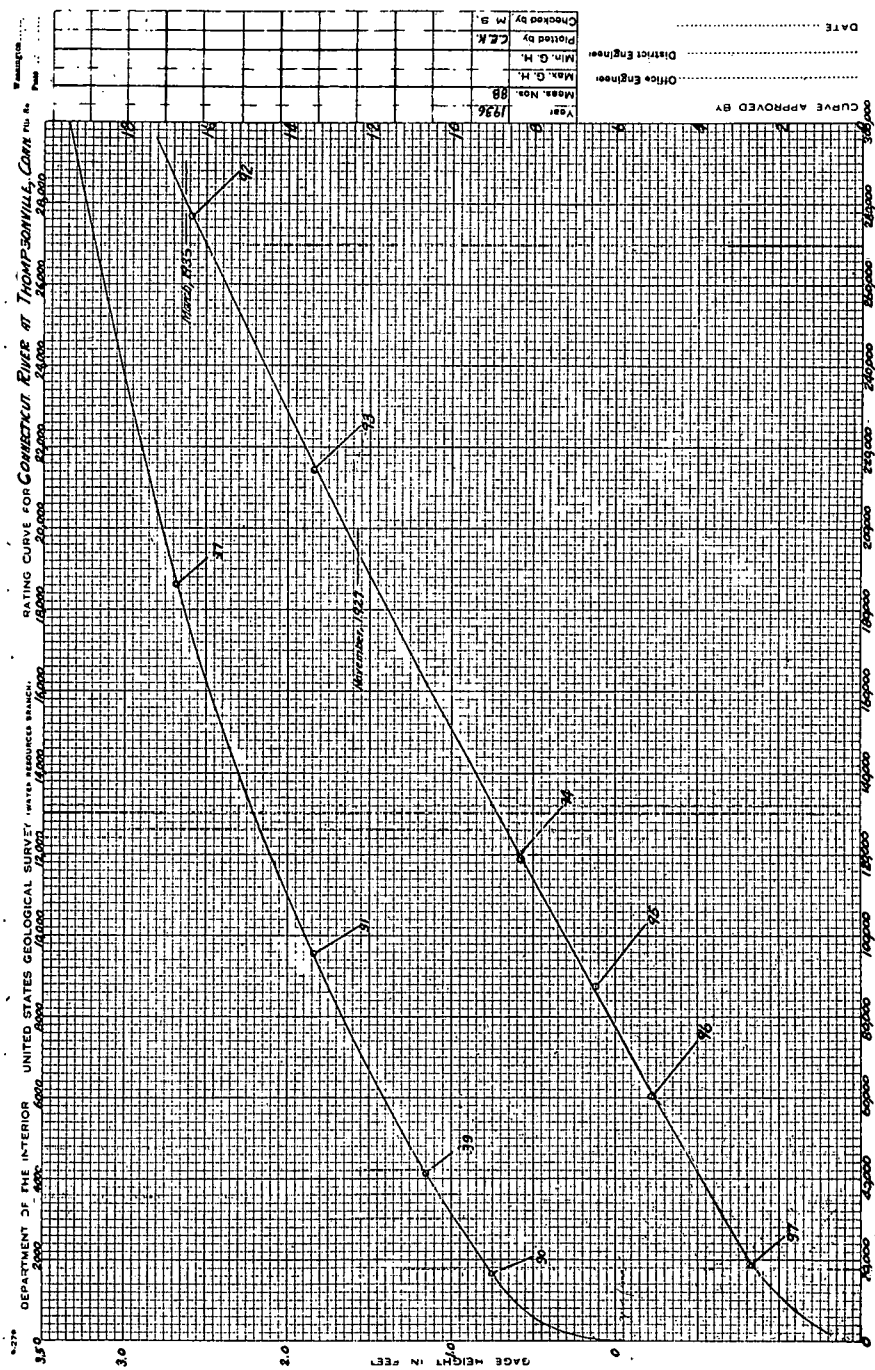

FIG. 2. — RATING CURVE OF CONNECTICUT RIVER AT THOMPSONVILLE, CONN.

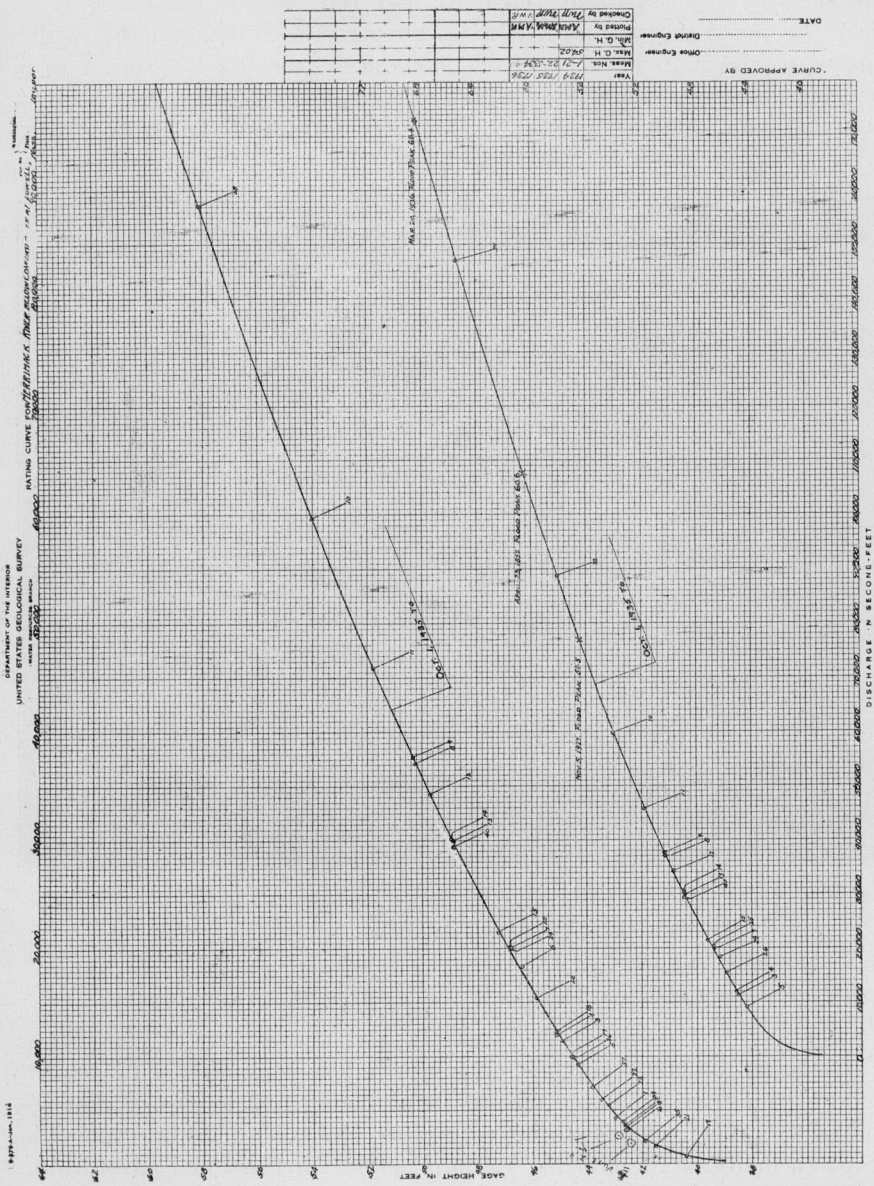

FIG. 3. — RATING CURVE OF MERRIMACK RIVER AT LOWELL, MASS.

By means of the great number of high water current meter discharge measurements, all made at unusually high river stages and many at or near the peak of the flood, it has been possible to develop the rating curves to the maximum stage reached during the flood. Figs. 1 to 3 present thus representative rating curves developed by high water measurements made during and following the peak of the flood.

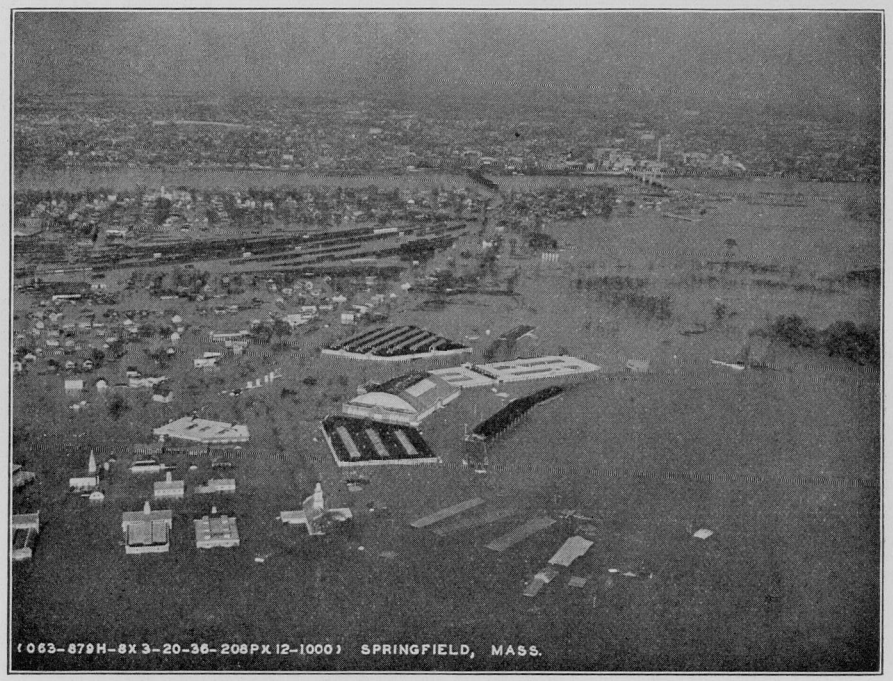

FIG. 4. — FLOODED AREA ON THE CONNECTICUT RIVER AT SPRINGFIELD, MASS., MARCH 20, 1936

In foreground, Eastern States Exposition grounds; in the distance, the Springfield Memorial Bridge, Municipal Auditorium and the city of Springfield

Due to the great number of gauging stations situated on rivers in flood at the same time, and also to the difficulty and length of time required to travel from one station to another, it was found to be impossible to reach many of the gauging stations before the flood had passed. At other stations the cableways or bridges from which discharge measure-

TABLE I

TABLE OF MAXIMUM DISCHARGES

River and point of measurement	Drainage area (square miles)	Period of record	Previous maximum			Flood of March, 1936				Runoff depth in inches for period Mar. 12-25
						Maximum				
			Date	Stage	Discharge c.f.s.	Time	Stage	Discharge		
								c.f.s.	c.f.s. per sq. mi.	
Pemigewasset River at Plymouth, N.H.	622	1903-1936	Nov. 4, 1927	27.4	60,000	Mar. 19, 8:00 a.m.	29.0	65,400	105	15.05
Merrimack River at Franklin Junction, N.H.	1,507	1903-1936	Nov. 5, 1927	30.85	(a) 83,000	Mar. 19, 3:00 p.m. to 4:00 p.m.	36.4	83,000	55.1	9.42
Merrimack River at Lowell, Mass.	4,635	1923-1936	Apr. 23, 1852	60.6	(a) 100,000	Mar. 20, 7:00 p.m. to 11:00 p.m.	68.4	173,000	37.3	9.06
Smith River near Bristol, N.H.	85.8	1918-1936	Nov. 4, 1927	(b) 7.02	5,800	Mar. 19, 8:00 a.m.	16.09	8,100	94.4	14.68
North Branch of Contoocook River near Antrim, N.H.	54.8	1924-1936	Apr. 19, 1933	6.55	2,370	Mar. 19, 2:00 p.m. to 4:00 p.m.	9.30	6,160	112	15.20
Suncook River at North Chichester, N.H.	157	1918-1936	Apr. 7, 1923	13.0	(a) 6,580	Mar. 19, noon	15.27	12,900	82.2	13.49
Soubegan River at Merrimack, N.H.	171	1909-1936	Apr. 8, 1924	11.82	(a) 9,260	Mar. 19, 8:00 a.m.	16.2	16,900	98.8	12.70
Ipswich River near Ipswich, Mass.	124	1930-1936	Mar. 7, 1934	6.06	1,580	Mar. 15, 2:00 p.m.	7.70	2,610	21.0	7.29
Taunton River at Slate Farm, Mass.	260	1929-1936	Apr. 15, 1933	10.4	3,100	Mar. 14, 10:00 a.m. to 6:00 p.m.	10.64	3,020	11.6	4.23
Blackstone River at Worcester, Mass.	31.3	1933-1936	Jan. 10, 1935	5.52	1,020	Mar. 18, 12:00 p.m.	8.58	2,520	80.5	9.27
Connecticut River at White River Junction, Vt.	4,068	1911-1936	Nov. 4, 1927	35.0	(a) 136,000	Mar. 19, 3:00 a.m. to 4:00 a.m.	32.6	120,000	29.5	8.17
Connecticut River at Montague City, Mass.	7,840	1904-1936	Nov. 5, 1927	42.7	(a) 177,000	Mar. 19, 10:00 a.m. to 8:00 p.m.	49.0	228,000	29.1	8.38
White River at West Hartford, Vt.	690	1915-1936	Nov. 4, 1927	29.3	(a) 120,000	Mar. 18, 10:00 p.m.	18.89	45,400	65.8	9.74
Ashuelot River at Hinsdale, N.H.	420	1907-1909 1914-1936	Mar. 29, 1920	9.98	18,000		10.1	16,400	39.0	10.05
Millers River at Erving, Mass.	370	1914-1936	Apr. 19, 1933	5.94	6,000	Mar. 19, 1:00 p.m. to 5:00 p.m.	10.66	19,700	53.2	10.31
Swift River at West Ware, Mass.	186	1910-1936	Apr. 7, 1923	9.08	2,390	Mar. 19, 6:00 p.m.	15.0	7,590	40.8	8.28
Westfield River at Knightville, Mass.	162	1909-1936	Nov. 3, 1927	(b) 5.2	16,000	Mar. 18, noon	24.07	25,700	159	12.03
Westfield River near Westfield, Mass.	497	1914-1936	Nov. 4, 1927	35.41	42,500	Mar. 18, 6:00 p.m.	27.20	48,200	97.0	9.74
Farmington River near New Boston, Mass.	92.0	1913-1936	Nov. 3, 1927	9.48	(a) 8,610	Mar. 18, 9:30 a.m.	10.65	9,080	98.7	10.15
Housatonic River near Great Barrington, Mass.	280	1913-1936	Nov. 5, 1927	10.0	5,690	Mar. 19, 8:00 p.m.	10.60	8,990	32.1	8.33
Winooski River at Montpelier, Vt.	433	1909-1923 1928-1936	Nov. 3, 1927	27.1	57,000	Mar. 18, 6:00 p.m.	16.57	20,000	46.2	7.65

(a) Revised
(b) Former site and datum

ments are made were damaged, submerged or destroyed, so that it was impossible to obtain measurements. At all such sites where current meter discharge measurements at high stages were not obtained, it was necessary to determine the maximum discharge by other means. One or a combination of the following three methods were used:

1. Computation of the maximum flow over dams by the application of the Francis weir formula.
2. Computation of the maximum flow through contracted sections.
3. Computation of maximum flow through short reaches of a river by the application of the Chezy and Kutter's formulæ.

The complete data resulting from these special studies, gauging stations and much data obtained from other sources will be published by the Geological Survey in an early Water Supply Paper.

A few results selected so as to give an indication of the magnitude of the flood and a comparison with earlier floods are shown in Table I. A study of all results obtained point to the following conclusions:

1. Maximum rates of discharge per square mile on small drainage areas were fairly uniformly great, but in no case so great as many of the values obtained during the flood of November, 1927.
2. Maximum rates of discharge per square mile on large drainage areas lying within the area covered by the storm were usually much greater than ever before known.
3. All rivers and tributaries, without exception, situated in the storm area were seriously affected by the flood.

THE MERRIMACK RIVER AT LOWELL

By Arthur T. Safford, Member*

For many years we have been getting records from up country of the flood conditions on the Merrimack River system. At this particular time, prior to the Great Flood there was a rise at Plymouth, New Hampshire, of from 2 feet to 22 feet on March 12. Therefore a moderate freshet was expected. That first freshet, from March 12 to March

* Chief Engineer, Proprietors of Locks and Canals on Merrimack River, Lowell, Mass.

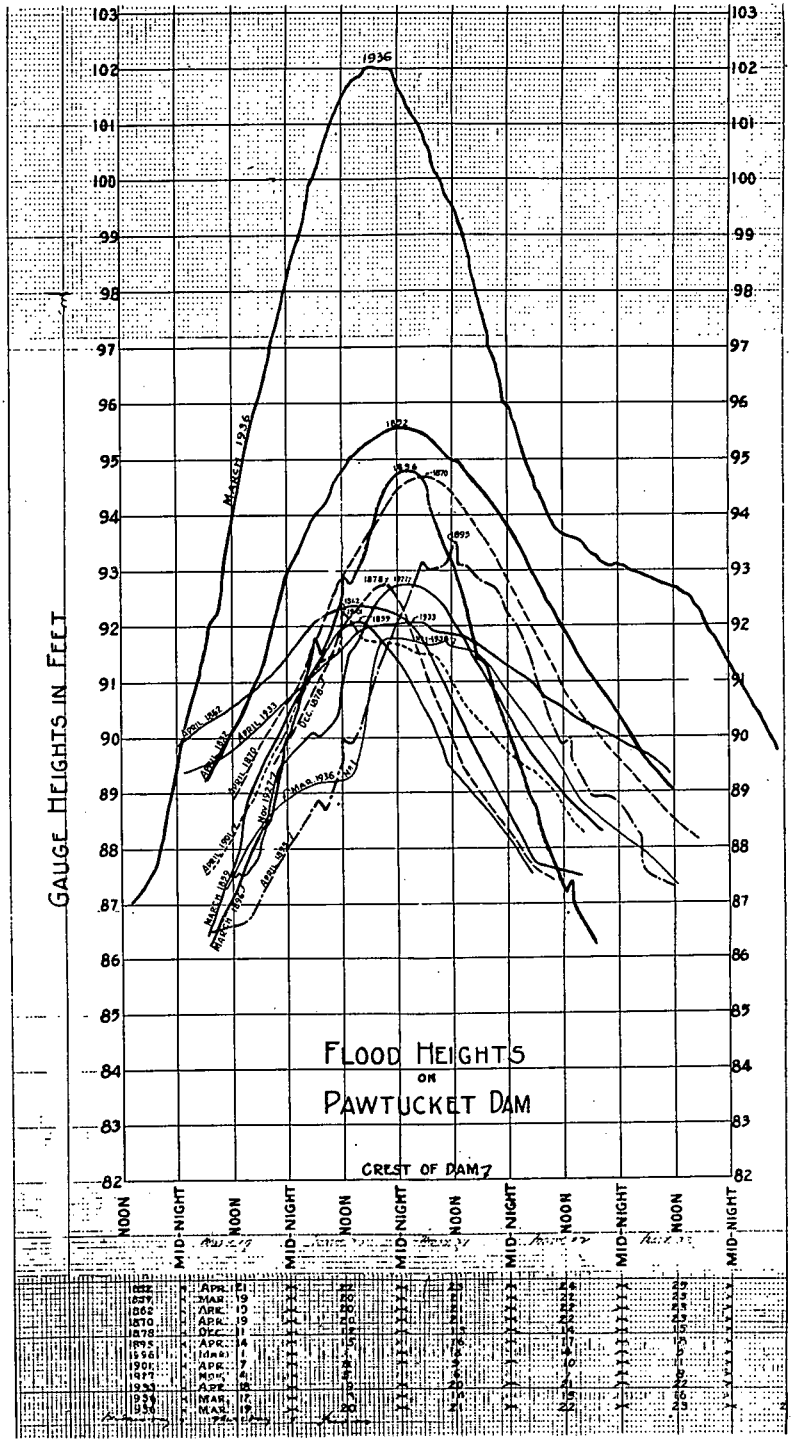

FIG. 5. — FLOOD HEIGHTS OF MERRIMACK RIVER AT PAWTUCKET DAM AT LOWELL, MASS.

16, which started with a rise of 22 feet at Plymouth was the "ice freshet." Very fortunately the ice went out and the river cleared itself, so that when immediately after that there came this tremendous amount of rain in the upper river, we were in a way prepared for the Great Flood.

The diagram (Fig. 5) indicates the 1927, 1896 and 1852 floods all plotted to the same vertical and horizontal scales, the latter scale showing the time of the day. On Wednesday night, March 18, the rate of rise of the 1936 freshet was about 50 per cent higher than the freshet of 1852. It was perfectly evident a serious flood condition was imminent. Mr. Kent and I had agreed that early Thursday morning we would drop the "Francis Gate," and put on flashboards, backed up by sandbags, which raised the barrier 3 feet above the Francis Gate. At the same time, while the river was rising at this increased rate, information was obtained by telephone with more or less regularity every half hour from Ayers Island, Bristol, Garvins Falls, and the Amoskeag Manufacturing Company, Manchester, New Hampshire, which indicated that the peak would be very high and long, and that it would be necessary to watch all the time for unforeseen conditions on the upper river and for breaks.

The peak of the 1936 flood was 6.4 feet in height above the 1852 freshet over Pawtucket Dam; and, in amount of water, approximately 160,000 cubic feet per second as against 100,000. This was a freshet 60 per cent higher than the amount of water which had ever successfully been met before.

In 1896 my company took the necessary levels and plotted the areas in the city of Lowell flooded by the 1896 freshet. A study was made of the area that would be included by a flood possibly 5 feet above that freshet. The irregular outline in Fig. 6 indicates the 1936 flooded area.

The Southern Division of the Boston & Maine Railroad from Nashua enters Lowell beside the river, where the land is very low. After the 1896 freshet, upon our advice the Boston & Maine Railroad raised its tracks 2 feet.

This year the Boston & Maine Railroad, with hardly any notion of what was going on, found itself barricaded by a dam about 8 feet high backed up by timbers, and the water, which ultimately reached its peak of 102.00 (20 feet above the crest of Pawtucket Dam), actually rose between 4 and 5 feet behind that dam.

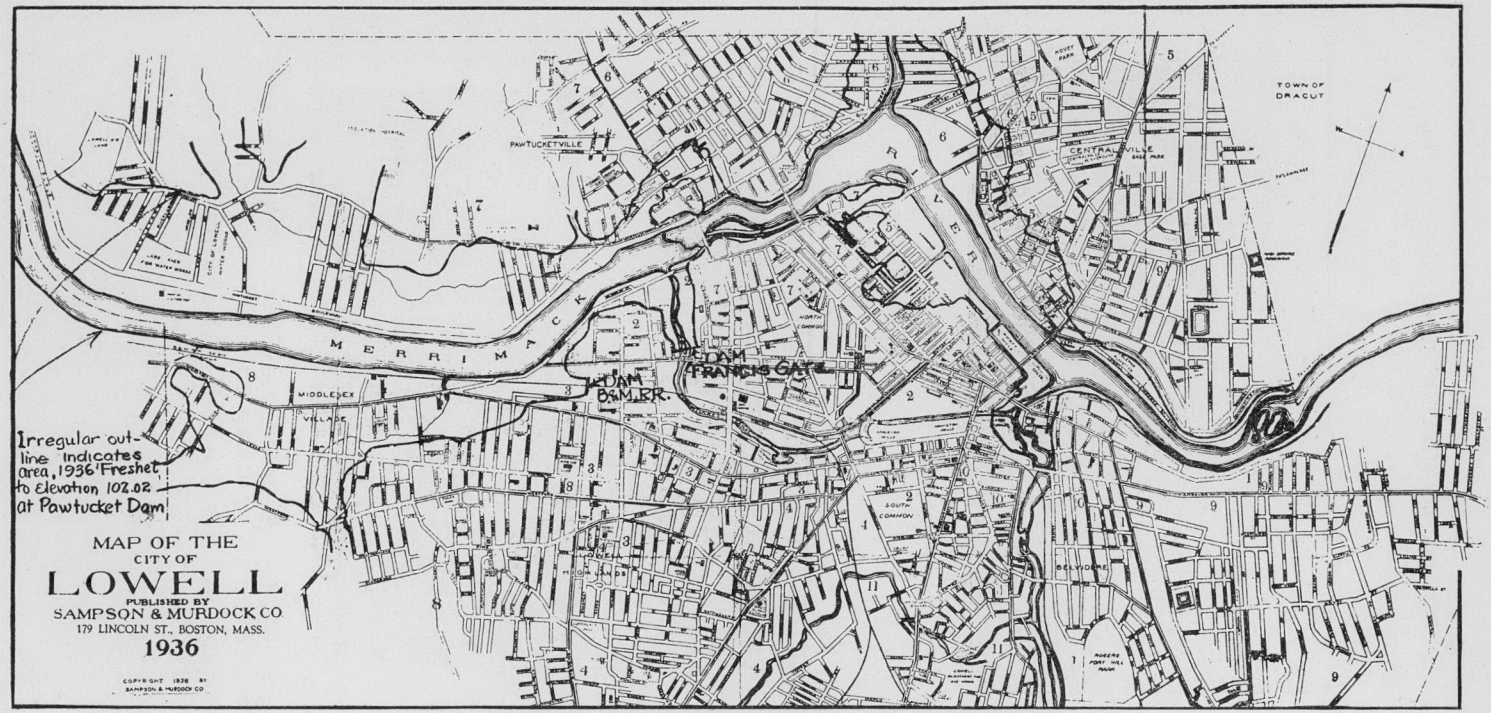

FIG. 6. — FLOODED AREA IN CITY OF LOWELL, MARCH, 1936, FLOOD

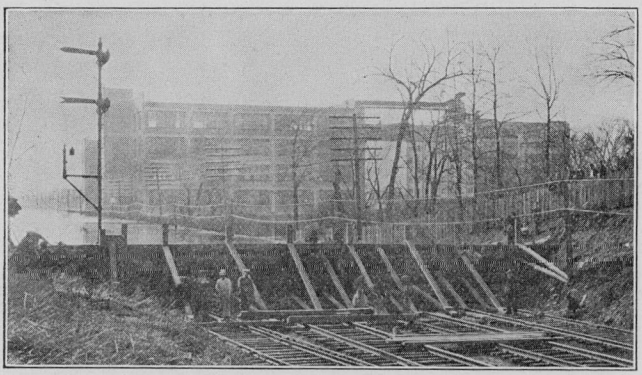

FIG. 7. — EMERGENCY DAM ACROSS TRACKS OF THE BOSTON & MAINE RAILROAD AT LOWELL

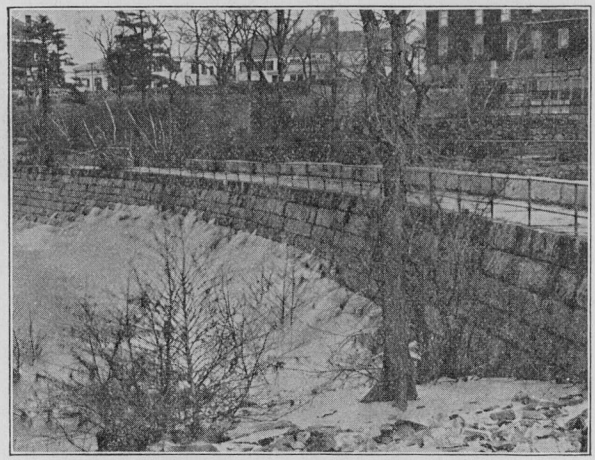

FIG. 8. — NORTHERN CANAL WALL ABOVE MOODY STREET, LOWELL

Showing leakage through the wall

During the night of Wednesday, when it was evident that the flood would rise to a height and flood of water we had never had before, warnings were sent out to all the city departments, particularly to the police and the fire departments. Wonderful work was done by them in getting boats and providing help, and they began to take people out of the houses. At the same time, the W. P. A. responded with a great many men; and even women who were working on the sewing projects in some way got hold of an enormous amount of blue denim, and made thousands of sandbags. I think without the W. P. A. it would have been difficult to have assembled enough people to have met the test that came the next day.

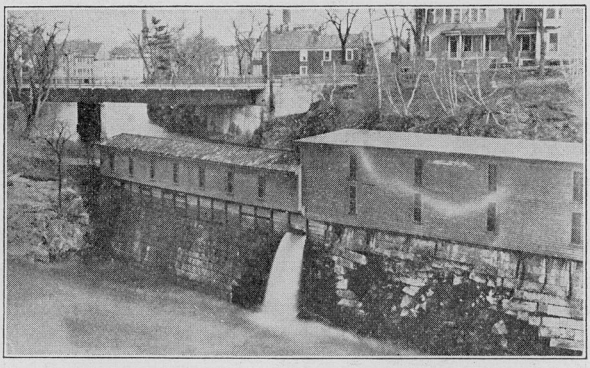

FIG. 9. — MOODY STREET WASTE GATE ON NORTHERN CANAL

Showing foundation washed away by freshet

There were just two places where the water might break in through the business district. One was at our Old Guard Locks, where the Francis Gate is located, and the other was in the Boston & Maine cut where the railroad enters the city.

The map of the flooded districts (Fig. 6) shows a number of low places where attempts were made to keep out the water.

Efforts were made to put up barricades on the north side of the river (known as Pawtucketville), but to no avail. There was no time or opportunity to build permanent structures, merely piles of sandbags which proved to be of no use, as the river pushed them over, broke through that part of the city, and flooded and ruined a great many houses. The people, of course, had to be taken out and cared for elsewhere, which was done early and in a sympathetic and efficient manner.

Also the current of the river broke through Centralville. If one has seen any of the results of the freshet, it would be perfectly evident how madly the river had rushed through the streets and ripped and torn everything to pieces. Even some of the abutments of the Merrimack River bridges were undermined.

It was appalling to see a mile of mills, located directly on the Merrimack River below the dam, whose basements were all flooded, the first floors flooded to a large extent and pumps put out of commission; and to be faced with the constant danger that even with all that was done we probably could not meet the test.

To my mind the amazing part of it is that from the upper Pemigewasset we are only now getting the record-breaking records of rain and

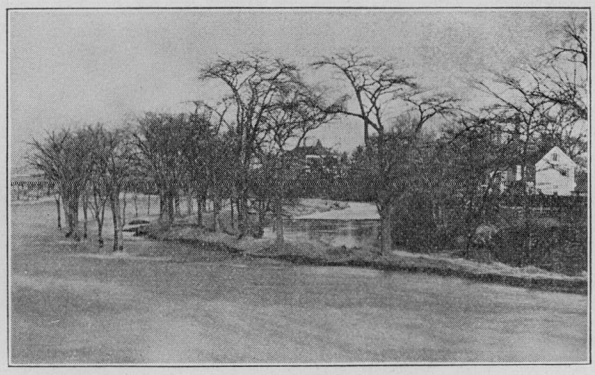

FIG. 10. — MERRIMACK RIVER BELOW SCHOOL STREET BRIDGE, LOWELL

Showing high water flowing into Northern Canal. Elevation of water at Pawtucket Dam, 101.32 at 3 P.M., March 20, 1936

snow water content that caused the trouble. And it makes one stop and think, to realize that large areas had 10, 12 and even 18 inches of rain; and there were reports that snow in the valleys was 2, 3 and 4 feet in depth, and we practically got it all between the two freshets. I have no doubt that 12 inches of run-off must be about what went down the river.

Now there is not much more to be said, except that I am wondering if we are not faced with a little different set of conditions. We never saw such rain as that before. Every winter there is the snow, but if

these rains are to come in the future, it is imperative to build higher and more permanently.

I presume that the 20 feet on the dam at Lowell was probably increased by a concrete arched bridge, just below the dam. When it is noted that the velocity down the river was 10 feet per second, the velocity over the dam 20, and the velocity through this bridge at the gorge perhaps 25, one must consider the head required to maintain those velocities.

As a matter of fact, in 14 miles of river, the water at the Jackson Company, at Nashua, New Hampshire, stood 40 feet above the dam at

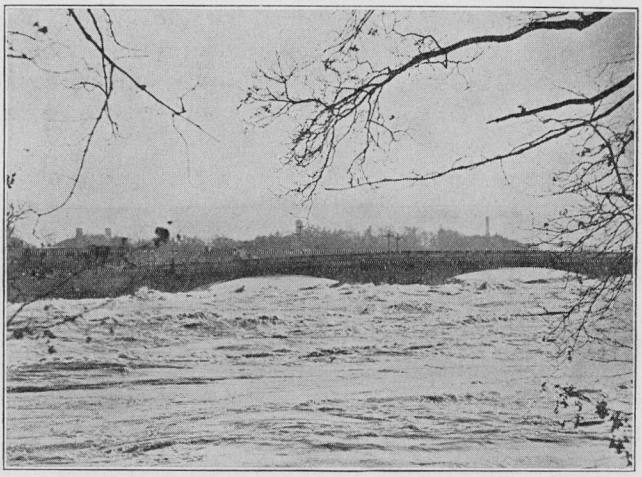

FIG. 11. — MERRIMACK RIVER AT LOWELL THE DAY AFTER THE PEAK, THE WATER SURFACE AT PAWTUCKET DAM BEING AT ABOUT ELEVATION 100. PAWTUCKET BRIDGE STILL AN OBSTRUCTION

Lowell. In other words, there was a slope of 20 feet in 14 miles, or about a foot and a half to the mile.

The thing of most interest to people in Lowell and to visitors was the Francis Gate. In 1850 Mr. Francis, from a study of previous floods, made up his mind that he should make greater provision against serious floods. Consequently he built this gate, called "Francis Folly," to close the lock when dropped, and carried it to a height that was well above the 1852 freshet. When it is noted that the water went a foot over the Francis Gate and came within 2 feet of the top of the protective dam above this,

one can understand what we were up against. All the work of building the skeleton dams was directed by Mr. Stanley Kent, my assistant, as I had to stay in the office to answer questions, such as, "Has the Amoskeag Dam gone out?" and "Has it been blown up?" questions of the sort which simply added to our anxiety.

I might add that the Plymouth peak was four hours long, the Lowell peak eight hours long; but for sixty hours the water stood above the height of the 1852 freshet.

As a result of this great flood, the conditions leading up to it, and of a preliminary study of some of the wrecked structures, the following appear to me conclusive:

Torrential rains in New England with heavy snow may become a more common condition for the late winter than before.

Abutments should be higher, even if the dams are stable; most of the danger at the main river dams was from washing around the ends.

Bridges and approaches should be higher, or the chance of their being carried away taken deliberately.

Key places in settled communities, where water is likely to break through, should be protected by skeleton dams.

Reinforced concrete arched bridges should not be built just below dams so as to become the controlling constriction in the flow.

The velocity of water approaching and leaving the main river dams, for the first time, scoured and moved large amounts of material — something to be expected only from mountain brooks.

Permits to build should exclude locations within known flood areas.

For the future, public service companies should be looked to for flood warnings, because of their greater property stake, better records, and skilled continuous operating crews.

At the present time too many organizations are simply collecting data; there is no definite responsibility even with government agencies.

THE FRANCIS GATE, AT LOWELL

By S. Stanley Kent, Member*

The so-called Francis Gate was built at a strategic location in the canal system by Mr. Francis in 1850. The gate was suspended across the lock at what is called the "Old Guard Locks," at the head of the Pawtucket Canal, which was the original navigation canal built in 1792. This canal runs down through the heart of the business and manufacturing districts of Lowell. A failure of the head works at the "Old Guard Locks" would permit a substantial part of the flood waters to pour down through the business and manufacturing districts.

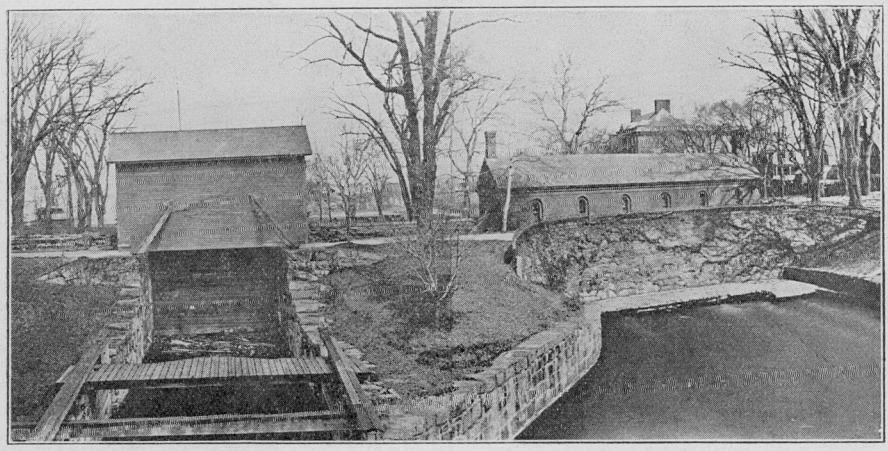

Fig. 12. — Old Guard Locks and Francis Gate, Lowell

The gate itself was not quite 27 feet wide, about 25 feet high, and was built of 18-inch white pine timbers that were kyanized. It weighs about 20 tons. It was suspended from a cast-iron cradle, supported by timbers leaning together at the top, and it had been hanging there ever since it had been hoisted after the flood of 1852.

Early Thursday morning the water reached a point where it was evident that the gate would be needed. Our men were called out at half past four, and at quarter of six they started cutting the shackle which

* Assistant Engineer, Proprietors of the Locks and Canals on Merrimack River, Lowell, Mass.

held the gate. This shackle was a wrought-iron forging, and the strap was about 4 inches wide, and three quarters of an inch thick. The tools, had been left on the platform above the gate for this purpose. The cutting process took about fifteen minutes. The gate dropped absolutely straight, landed in about 7 feet of water with a splash and crash, and came to rest.

At the peak of the flood the depth of water on the upstream side was a little over 26 feet, and on the downstream side, about 6 feet, giving a pressure of approximately 20 feet on the bottom part of the gate. It is figured that the stress on those white pine timbers, some eighty-five years old, was about 2,200 pounds per square inch. I believe the breaking strength for white pine is given as about 4,400 pounds, so that it is evi-

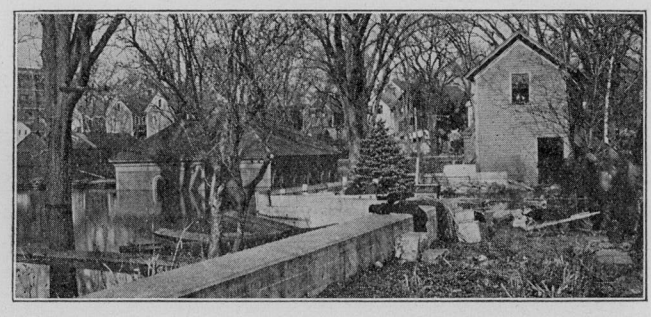

FIG. 13. — FRANCIS GATEHOUSE. PERMANENT DAM BUILT AFTER 1927 FLOOD. WATER NEARLY AT PEAK OF 1936 FLOOD

dent that those timbers were well stressed. The computed deflection for these conditions is about 2½ inches. It looked as though there was fully that much.

Mention may be made of one other point in Lowell where there was trouble. The Northern Canal wall follows the river for about 800 feet above the Moody Street bridge. It was evident early Thursday morning that at the peak of the flood the river would overtop this canal wall and the overflow would have to be taken care of through the canal system. All the head gates were closed at the upper end of both canals, with the exception of 5 feet on one gate at the "Old Guard Locks," which was left up to make certain of maintaining the water level on the lower side of the Francis Gate. When the river overtopped the Northern Canal wall the various waste dams on the canal system were opened gradually. At four o'clock Friday afternoon, as the river neared its

peak, there must have been between 6,500 and 7,000 cubic feet per second going over the wall into the canals.

Then, at about five o'clock Friday afternoon, something like 200 feet of capstones on top of the canal wall were pushed into the canal, probably by floating débris. This added some 1,500 to 2,000 second feet to the flow through the canal system. At about the same time one of the ice booms at the principal waste dam in the heart of the system broke loose and closed off thirteen bays, which shut off the discharge of about 2,000 second feet of water. The result was that there were some three to four thousand second feet more than the canal system could handle. This excess water went over what is called the "Little Canada District" and found its way back to the river. Fortunately, it did little damage.

MERRIMACK RIVER DAMS IN NEW HAMPSHIRE AND THE 1936 FLOOD

By Richard S. Holmgren, Member*

In times of flood those living near the lower reaches of a large river are continually hearing rumors of dams above which have failed, releasing huge volumes of water to add to the destruction in the valley below. The recent flood on the Merrimack River was no exception to this, for during the early stages of the flood rumors were everywhere telling of dams in New Hampshire which had failed. After the flood had subsided and communications had been re-established, it was found that although heroic measures had been necessary, the dams along the Pemigewasset and the Merrimack rivers had passed the flood with very little serious damage, although the flood peaks on these rivers had exceeded all previous records.

There are six dams on the Pemigewasset and Merrimack rivers below Plymouth. Four of them are owned by the Public Service Company of New Hampshire, namely, the Ayers Island Dam at Bristol, the Eastman Dam at Franklin, the Garvins Falls Dam just below Concord, and the Hooksett Dam in Hooksett. The Concord Electric Company owns a dam at Sewalls Falls, just above Concord, and the Amoskeag Manufacturing Company owns the sixth and lowermost dam at Manchester.

* Chief Engineer, New Hampshire Water Resources Board, Concord, N. H.

The Ayers Island Dam at Bristol is an Ambursen type built in 1930. The crest length is 263 feet, with a nominal freeboard of 18 feet, which was more than enough for the maximum flood height reached of 15.0 feet. The company had built a three-foot parapet wall on top of the abutments as a protection to visitors, which also gives additional freeboard at times of extreme flood. This dam has a sluice gate 16 feet by 28 feet, and this gate passed approximately one seventh of the peak flow when it was open 14 feet. The spillway crest is equipped with hinged flashboards which were all released during the flood. The power house was in operation during the entire flood period, except for a brief shutdown during the first flood due to ice-blasting in the forebay.

The next dam down the Pemigewasset River is the Eastman Dam of the Public Service Company of New Hampshire at Franklin. Of considerable height, this old timber crib dam has been the victim of many scare stories, but still stands, although in a battered condition. The 338-foot spillway has a freeboard of 12 feet, and although the water rose 2 feet above that, extensive sandbagging prevented erosion at the abutments. The power house was completely put out of business, and was off the line for at least a month. The maximum stage of 14 feet was the same as in 1927, but when the pond was drawn down, the apron was seen to be removed entirely, and the crest of the dam had settled a maximum of 3.1 feet for some two thirds of its length, so comparisons are not possible.

Just above Concord is the Sewalls Falls Dam of the Concord Electric Company, a timber-crib dam, built in 1892. This structure has withstood the ravages of 1895 and 1896, 1927 and 1936. Only 14 feet high, this dam caused but a sizable wave in the river, for at the maximum stage of 16.8 feet, the tailwater was some 10 feet higher than the crest of the dam. Although the spillway is 497 feet long, the freeboard of 14 feet was inadequate. A hard-working force of W. P. A. workers kept half a sandbag ahead of the rapidly rising waters, and their task was made more praiseworthy because the abutment walls are very narrow. A vicious eddy directly below the dam was eating its way into the canal embankment and also required attention. Had the situation gotten beyond control, the river might have made short work of the gravel banks at each end of the dam. The power plant escaped with a mere foot and a half of water on the floor. It was 24 feet above normal in tailrace. The electrical equipment was not damaged, and service was resumed as soon as the transmission lines were replaced and a head could be developed. The two decks of the dam, below the crest eleva-

tion, are completely devoid of their protective steel plates, but otherwise the dam came through unscathed.

The next plant downstream is at Garvins Falls. This, too, is a plant with a canal, and although the 13.7 feet of water over the spillway did not seriously endanger the 15-foot freeboard, the problem of keeping the water from entering the canal through the double-track railroad right-of-way was of great importance. A cut-off wall had been built under the tracks, but plans for providing abutments to carry stop logs across the tracks were never constructed. The dam is of rubble stone masonry, built in 1903, and is about 20 feet high. There is an ogee spillway 475 feet long, and a 75-foot section, 2 feet higher. Although it was not apparent at the time of the flood, later, when the pond was

FIG. 14. — SEWALLS FALLS DAM, MERRIMACK RIVER, AT CONCORD, N. H.

Showing water after the peak had passed

drawn down, it was seen that three rows of capstones had been taken off by the flood for practically the entire length of the ogee section. These blocks, some weighing 6 tons, are scattered from 100 to 300 feet below the dam. The water rose 24 feet below the power house, and the two older horizontal generators were completely covered, but the newer, vertical generators remained dry.

The plant of the Public Service Company of New Hampshire at Hooksett was in the middle of the severest washout in the Merrimack Valley. The dam is of stone masonry and creates a 14-foot head, but at the time of the flood peak there was only a 3-foot drop over the crest. The generator was not flooded, being on a high pedestal, but the

station itself was completely washed out. Switchboards and apparatus were soaked, and after the flood, everything in the place, except the generator, had to be taken apart in order to get rid of the sand. The forebay and tailrace were also completely filled with sand.

Although not related to the subject of dams, one is forced to note what the Merrimack River did to Hooksett Village. Starting at the dam, the village extends down the stream for three quarters of a mile, on the outer edge of the sharp bend. With water some 18 to 20 feet flowing through the main street of a village, pole lines, barns, houses, stores, schools, roads and sewer systems were quickly swept away. Along with these went several hundred feet of double track railroad fill, some 20 feet high. It required 25 bents of pile trestle to get a train from the east bank to the abutment of the three-span bridge which was left unharmed in the middle of the river after the flood receded.

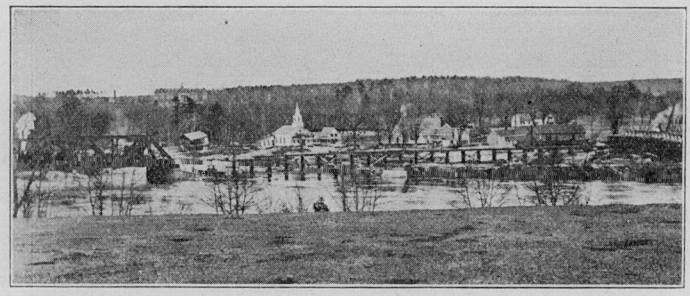

FIG. 15. — WASHOUT AT HOOKSETT, N. H.

Building trestle to reach bridge

To add to the sum total of trouble at this point, there was a three-span wooden railroad bridge which crossed the river just above the Hooksett Dam. Being on an abandoned branch of the railroad, the bridges had been scheduled for removal this spring. When it was apparent that a real flood was in progress, people for miles around lined the banks to see these bridges float away. They were not disappointed. One by one, the bridges left their abutments. Over the dam they went but remained intact. The first section stranded on the bank directly above the bridges. The next section removed a span of the steel highway bridge, but did no damage to the railroad structure just below. And the biggest one of all missed what remained of the highway bridge, sailed through the site of the approach to the railroad bridge, and grounded itself on the bank below.

At Manchester is located the last dam on the New Hampshire part of the Merrimack, and here occurred the most thrilling fight of all. All day Thursday the water rose, at times, at the rate of 8 inches per hour. The dam built in 1923 is of concrete, about 20 feet high, with spillway section 340 feet long at elevation 70, a 400-foot section at elevation 72, and a short 10-foot section at elevation 74. In spite of this length of spillway, the 12-foot freeboard was inadequate for the 17.1 foot stage which was reached at the gate house. As soon as it was realized that the 1927 flood was to be surpassed, all available National Guard and W. P. A. workers were mobilized at the armory and at fire stations. It is estimated that 500 men placed half a million sandbags in defence

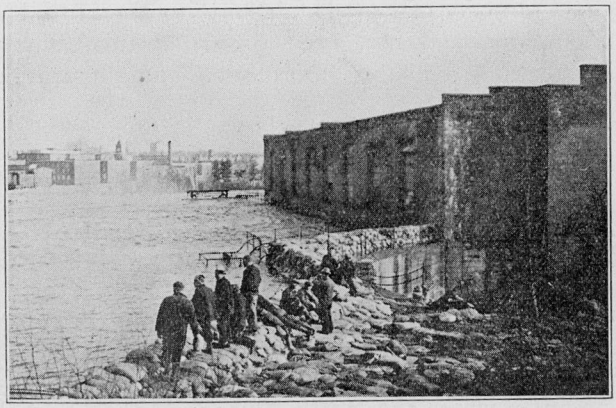

FIG. 16. — BUILDING SANDBAG DAM TO PROTECT POWER HOUSE AT AMOSKEAG DAM, MANCHESTER, N. H.

of the Amoskeag Dam. As at Garvins, a railroad cut lower than the dam abutments had to be filled in as well as raising the abutment with bags. On the west side of the river, some 4 feet of sandbags kept water from flooding the power house. With hard work the men kept about 2 feet ahead of the flood at all times. Fortunately the Amoskeag bridge directly above the dam was able to be used by sand trucks and emergency cars during the entire flood period. The power house and the dam came through the flood unscathed. Due to peculiar channel conditions, the water was 3 feet higher on the east bank than at the hydroplant on the west end.

The mills below the dam were badly flooded in 1927 and 1895 and 1896. At the lowest mill the water was 15 feet higher, and at the site

of the McGregor bridge the flood topped 1927 elevation by 16.3 feet. In one of the mills is a monument marking the stages of the big floods. In 1895 the peak was at 37.5 feet, in 1896 it was at 38.8, in 1927 it was at 36.0, and in 1936 it went off the monument, up on the floor above until it was at elevation 52.3 or 13.5 feet above the all-time high.

From this account it will be seen that the weakest points in the dams considered were inadequate spillway capacities due to lack of sufficient freeboard. Fortunately, by the use of sandbags these abutments were raised sufficiently to prevent overwashing. Permanent repairs are now in progress at practically all dams, which will assure sufficient capacity to pass similar floods in the future.

Acknowledgment is hereby given to Mr. William Noyes of the Public Service Company of New Hampshire, to Mr. Alfred Vose of the Amoskeag Company, and Mr. Fred Brown of the Concord Electric Company for making available information and illustrations used in this paper.

SOME EXPERIENCES IN THE NEW ENGLAND POWER ASSOCIATION TERRITORY DURING THE FLOOD OF 1936

By Harry M. Nelson*

Some of the methods used to combat the flood were rather unique. At Shelburne Falls, where the North River joins the Deerfield River, a wooden highway bridge on the North River looked as if it might be taken out by the high water at any minute. The selectmen held a meeting and decided that the situation demanded that the bridge should be set on fire. The fire was well under way when the river picked up the bridge and carried it downstream. The bridge, in flames, first landed against a footbridge belonging to the New England Power Company and set that afire in several places. It then went under the footbridge and continued downstream, alarming the residents with houses near the river, until it landed against an abandoned concrete trolley bridge in the center of Shelburne Falls Village. After burning for some time it broke up and passed down the river.

At Bellows Falls the river was watched closely. When it was found that a record flood was likely, plans were made to prepare for it. Probably the most important item of this preparation was to plug the rail-

* Hydraulic Engineer, New England Power Service Company, 89 Broad Street, Boston, Mass.

road tunnel, which is a stone masonry lined structure in earth under the business section of the town. In 1927 water started to go through this tunnel, but was stopped when the construction crew, which at that time was engaged in building the Bellows Falls Hydro-Electric plant, placed a timber and sandbag cofferdam at the entrance. If the water had been allowed to pass through this structure it would probably have been wrecked and a great amount of damage to the buildings and streets over it would have been done. The plugging of the tunnel in March of this year was much simpler, due to the lesson learned in 1927, as timbers for the purpose had been framed and placed under a small shed at the entrance. Concrete slots to receive the timbers had also been built in the masonry portal.

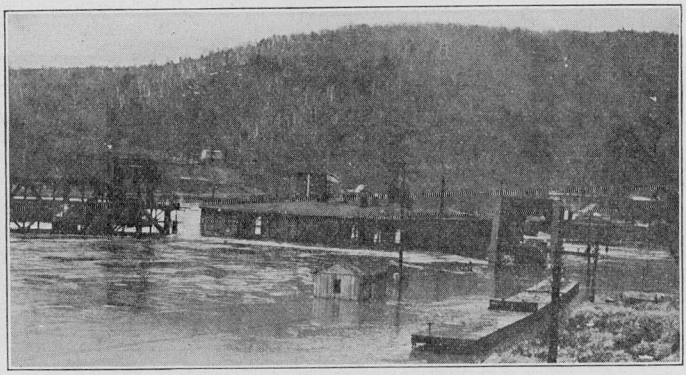

FIG. 17. — CONNECTICUT RIVER AT BELLOWS FALLS, VT., MARCH 19, 1936, AT 1.45 P.M.

At the height of the flood, with both roller gates on the Bellows Falls Dam raised as high as possible, and all three bays of the flashboards off, there were about 29 feet of water passing over the sills at the roller gate openings, and 24 feet over the sills at the flashboard openings.

Immediately downstream from the dam there is a bridge across the river, owned by the Boston & Maine Railroad Company, which was loaded with gondola cars. The water was between 6 and 7 feet above the bottom of the lower chord and floor system of this bridge, and the upstream side was badly hammered by the ice, trees, houses and barns that came down the river. The chord is made up of two channels about 18 inches deep, latticed together. The floating débris pounded these channels together so that their flanges were nearly in contact and the lattice bars were bent up into the shape of bows.

The Rutland Railroad Company bridge crosses the power canal just below its entrance. This bridge was loaded with gondola cars and suffered no damage, partly due to the protection afforded by the ice boom belonging to the power company. The water passed between the end of the dam and the Rutland Railroad roundhouse. As a result the upstream side of the roundhouse was pushed in and the downstream side was pushed out.

The heavy boom at the entrance to the power canal took a lot of punishment from ice and floating débris, but it survived and is still functioning, with the exception of a light section of the tail end next to the dam, which it was expected might go out.

Near the upper end of the Bellows Falls gorge there is a railroad bridge composed of two masonry arches with a pier in the center of the gorge. The two arch openings were nearly filled to capacity with the

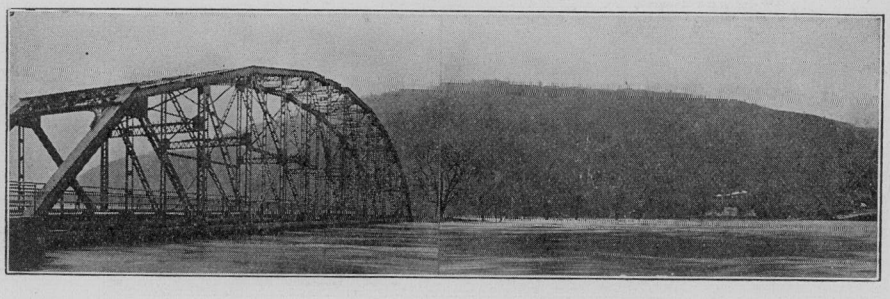

Fig. 18. — Connecticut River at Brattleboro, March 19, 1936, at 3.45 p.m.

water, which was so swift that it piled up at least 10 feet high above the general surface of the river at the point where it struck the center pier. This surging, foaming water made a beautiful sight.

The tailwater below the hydro-electric plant came up over both the 66,000 and 110,000 volt switch yards and made it necessary to cut the plant loose from the rest of the transmission system of the New England Power Association, but fortunately it was possible to keep one unit running to supply power to the Bellows Falls district. At the height of the flood we had two pumps of our own, a fire pump from North Walpole and another one from Bellows Falls, working to keep pumped out the wheel pit of the only unit capable of being run. This pit had been protected with sandbag cofferdams to keep out the water which had found its way into the basement of the power house. In spite of all this pumping, the water got within about an inch of the oiling system. Had it reached this

system it would have been necessary to shut down this plant, and Bellows Falls would have been without power, light and probably telephone service.

At Brattleboro the island which is crossed by the highway was completely covered, and a considerable portion of the island was washed away. An inspection of the first United States Geological Survey map of this district indicates that at one time this island was about 1,500 feet long. It is now practically gone except the portion on which the road is

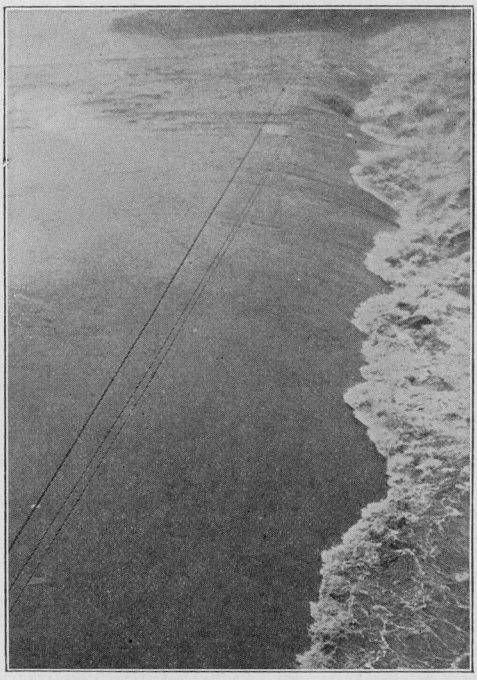

FIG. 19. — WATER OVER CREST OF VERNON DAM, MARCH 19, 1936, AT 9 A.M.

built. At one time the upper end was used for an amusement park, and up to the 1927 flood there were three or four houses on it which were inhabited. Most of the highway fill on the island approach to the bridge was washed out, but was quickly replaced when the water subsided so that traffic could be resumed.

The Central Vermont Railroad Company yard tracks at Brattleboro were practically all covered with water during the flood, and the round-

house was flooded. A great amount of silt and sand was deposited on the yard tracks, which had to be shoveled off by hand before the yard could be put back into service.

The Vernon Dam spillway is 600 feet long. During the height of the flood the tailwater rose about 36 feet above normal and was between 10 and 11 feet above the crest of the dam. The headwater was about 19 feet over the crest. The ordinary normal operating head at this plant is about 34 feet, but with tailwater above the crest of the dam, the plant was flooded and out of commission.

At the New Hampshire end of the Vernon Dam it was feared that the water would go over the concrete abutment and the earth fill in back of it. The power company forces started to put in some sand-

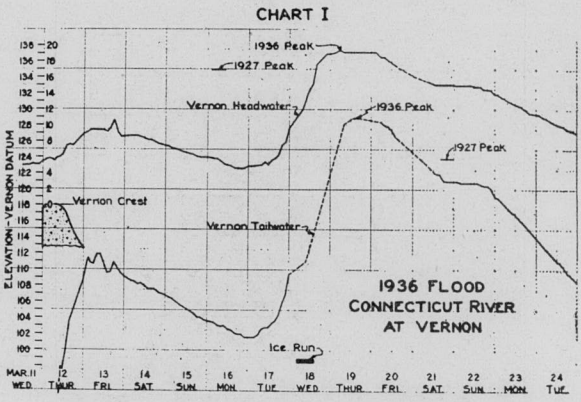

FIG. 20. — PEAK ELEVATIONS OF MARCH, 1936, FLOOD ON CONNECTICUT RIVER AT VERNON DAM, N. H.

bags, but the water came up so rapidly that the executives of the company became alarmed and made arrangements with the proper authorities in Keene to obtain men from the National Guard and American Legion to cope with the situation. A sandbag barrier several feet high was built by these men, so that no damage occurred to the abutment wall and embankment.

There were several places between Bellows Falls and Greenfield where the Connecticut River cut across its bends and formed a new channel. At the Vermont end of the Walpole–Westminster bridge a new channel was scoured out, and in the process the approach to the bridge was removed for a distance of perhaps three or four hundred feet.

A large frame building used for storing materials for a wreath factory, and which was located at the upper end of this new channel, disappeared entirely.

THE MARCH, 1936, FLOOD AT HOLYOKE, MASSACHUSETTS

By A. W. Ladd*

In order to get a complete, although somewhat condensed, picture of the flood of March, 1936, in and around Holyoke, it would seem necessary to commence, first, with the circumstances and effect of a very serious ice jam occurring in the Connecticut River about six and one-half miles above the dam at Holyoke. This ice jam formed about five days prior to the real flood, and for a period of two and one-half days caused considerable worry as to what would happen when it gave way. The ice jam formed on March 13 and held until the evening of March 15. During this period of about two and one-half days the river was diverted from its normal channel, overflowing low meadow land to the east and returning to its normal channel at the downstream end of the ice jam. The ice apparently made a complete barrier, jamming clear to the river bottom, at which point the channel is in the vicinity of 30 feet deep.

About 7 in the evening of March 15 the entire mass of ice began moving downstream at a rate of about six and one-half miles an hour, passing over the Holyoke Dam at a river stage of 9½ feet above its crest. About a month later, when the river flow had decreased sufficiently to permit partial inspection, it was found that for about 1,000 feet of length of the dam the granite crest had been taken off to a depth of 5 feet.

After the ice flow passed by Holyoke, the river began to fall off, but on the 17th it commenced to rise again, due to heavy rains in the upper Connecticut River Valley and warm weather throughout the watershed area. From a gauge reading of 7½ feet on March 17, at noon, the river climbed to a peak of 16.8 feet at 8 P.M. on March 19, at which point it remained until the following morning at 9 A.M. and then began to recede very slowly. The flow at the peak was estimated to be 225,000 cubic feet per second.

The gauge reading was just about 2 feet higher than the peak of the 1927 flood, at which time the gauge at the dam indicated 14.75.

* Hydraulic Engineer, Holyoke Water Power Company, Holyoke, Mass.

These stages are in terms of feet above the crest of the dam. In the northern section of Holyoke the river stage was about 6 feet above the 1927 level, while in the southerly portion of Holyoke the height was about 7 feet above that of 1927.

"Bottle-necks" in the river above and below Holyoke in all probability caused these comparative differences of river stages between the 1927 and 1936 floods. The loss of the crest of the Holyoke Dam also, possibly, had some bearing upon the matter.

The Holyoke Water Power Company had three important locations along the river to protect and guard closely during the vital flood period. The gatehouse and railroad cut on the Holyoke side of the dam was extremely important. Sandbag barriers were built along the entire

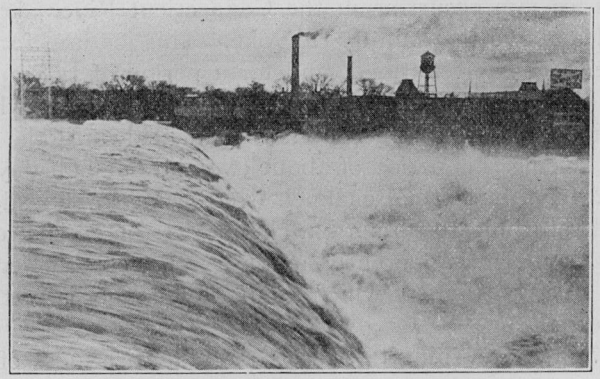

FIG. 21. — HOLYOKE DAM, HOLYOKE, MASS.

The maximum flow over the crest of the dam was about 3 feet higher than shown in this picture, taken March 22, 1936

upstream side of the gatehouse and across a railroad right of way, which prevented the flood waters from entering the canal system, consequently saving many thousands of dollars' damage. Sandbag barriers were also built up on the South Hadley abutment and around these gatehouses to prevent the flood waters from entering the South Hadley canal. At both of these places concrete walls had been built after the 1927 flood to an elevation of 16 feet above the crest of the dam. However, as before stated, the March, 1936, flood level was nearly 1 foot above the top of these walls. Therefore it became necessary to place sandbags upon them when reports indicated unknown and extreme stages were to be reached. These barriers were built up well ahead of the rising river.

At the railroad cut a wall of sandbags about 9 feet high and nearly 25 feet wide at the bottom was built. This particular barrier prevented water from flowing around one end of the gatehouse and entering the canal system. With these protective measures around the gatehouses, and by giving proper orders as to draft of water from the canals, it was possible to maintain control of the water level in two power canals, with a total length of three and one-half miles. The third level, or lower canal, in the southern part of the city was completely submerged. The river overflowed its banks so that at the peak the lower power canal was raised 7½ feet above its normal level.

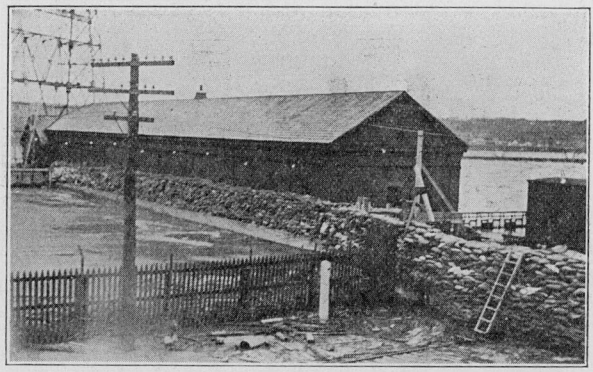

FIG. 22. — HOLYOKE WATER POWER COMPANY GATEHOUSE, HOLYOKE SIDE

Showing sandbag barriers on the upstream side of the gatehouse and across the railroad cut. The water surface had dropped about 3 feet below the peak when this picture was taken.

The third location, which required an immense amount of protective means, was at our hydro and steam electric station, located upon the river bank. For considerable time, and after the river had reached a height of many feet above the generator floor, water was kept out of a greater portion of the station. However, after sandbag barriers had been placed in all windows and doors and at other apparently low and weak points, the water pressure broke through a small window, which had been sandbagged, into a small generator building adjacent to the main power plant. The rising waters then entered the main building by means of a cable tunnel, and consequently flooded the entire station, covering the main generator floor to a depth of about 17 feet. By this time the entire

plant, of course, was completely out of operation, four hydro-generators being under water and all boiler fires put out.

At the three locations where several hundred men worked twenty-four hours a day for practically four days, over 100,000 sandbags, with several thousand feet of lumber, were used. These temporary barriers, hastily thrown up, for the most part, served their purpose extremely well, and many thousands of dollars' worth of property were saved. The spirit and willingness of the men engaged in this emergency work, with never a thought of the possible risk and danger involved, was very gratifying to those who were supervising the protective measures.

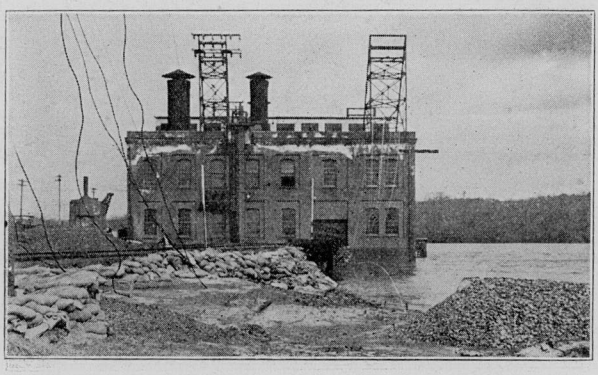

FIG. 23. — HOLYOKE WATER POWER COMPANY, NO. 1, POWER STATION, SOUTH END

Communication between strategic points during such a period as a flood is, of course, very vital and important. If in one location where emergency work is being carried on, that place becomes isolated, some means must be had to communicate with headquarters. When telephone service across the river to our South Hadley gatehouse could no longer be maintained, short-wave radio communication was established. The importance of such a method of communication was readily and quickly realized. We not only kept in touch with our own work in an isolated position, but it turned out that a good portion of an entire town could send messages and reports only by means of the short-wave radio service.

The experiences of the 1936 flood have taught, in so far as I see, two important principles: first, build protective and supporting devices, with as much strength and height as reasonably possible, with due regard for cost; and second, reliable communication between strategic points, in order that responsible persons can be in a position to transmit accurate information and reports to parties requiring such information.

DISCUSSIONS BY LETTER OF DR. JURGENSON'S PAPER ON "THE APPLICATION OF THEORIES OF ELASTICITY AND PLASTICITY TO FOUNDATION PROBLEMS."

Dr. Leo Jurgenson, in his paper "The Application of Theories of Elasticity and Plasticity to Foundation Problems," published in the July, 1934, issue of the Journal of the Boston Society of Civil Engineers, mentioned certain differences between the solutions of Mr. S. D. Carothers as given in the "Proceedings International Mathematical Congress," Toronto, 1924, Vol. 11, pp. 527–549, and those of Mr. E. Melan, published in "Beton and Eisen," 1919, for certain elastic problems.

The attention of these authors has been directed to the points at issue, and the following remarks will explain the differences in the two cases. — EDITOR.

PROFESSOR ERNST MELAN * (by letter): In the case of the plane problem, in which I have considered the boundaries to be presented by two parallel planes, on the surface plane the normal pressure is given according to the distribution of the load, which may be quite arbitrary; moreover, the shearing stress is zero here. On the other plane, at the depth $y=h$, also two conditions must be assumed (in the case I treated, vertical movement and at the same time shearing stresses are zero). Unless that is done, the solution is not entirely determined and an infinite manifoldness of solutions exists, if only one condition instead of two is stipulated. In this way my solution accords with a soft elastic layer on a rigid and slippery rest where the shearing stress as well as the vertical displacement is zero.

Concerning the complicated terms, which are obtained, the solution is figured out numerically only for the normal stresses on the rest due to a concentrated load on the surface plane.

Finally, it may be mentioned that Dr. K. Marguerre (Ingenieur-archiv, 1931) has considered this problem on the condition that any displacement, however, of any point of the rest plane is impossible.

* Technical University, Vienna, Austria.

Therefore he replaces the condition that the shearing stress is zero by the equation that the horizontal movement is zero. This case will be realized by a rigid rest, rough enough to prevent the horizontal displacement. I remark that my solutions and those of Mr. Marguerre give the limits between which the values of stresses may range in the case of a rigid support.

S. D. CAROTHERS, Esq.* (by letter): I have derived solutions for the stresses due to a uniform load on a long, narrow strip having parallel edges and of width 2a, and among others consider two special cases, as follows:

Case I. Where at the depth h below the surface there is a slippery plane.

Case II. Where at the depth h below the surface there is a hard rock surface where the vertical movement is zero and where friction is not restricted in value.

In Case I the movements of the slippery plane are not restricted in any way whatever, and the solution given would indeed lead to large vertical movements at this plane. All that is claimed by the author is that it is a possible simple solution which gives a set of stresses possibly more unstable than any other possible set.

In Case II the solution given is again claimed to be a simple, possible solution, consistent with unrestricted shear on the plane at the depth h below the surface. In the original paper it was stated that the vertical movement at this plane was zero. It is now found that this statement, written conjecturally, on account of the symmetry of both the positive and negative images about the plane $y=h$, does not bear examination. The plane $y=h$ does, in fact, move vertically.

I am indebted to Professor Melan for pointing out that the movement on the plane $y=h$ is not zero.

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* 2 Elm Road, Beckenham, Kent, England.

Of General Interest

Book Note

A New Edition of the Kent's "Mechanical Engineers Handbook" is expected to be ready for distribution in October, which will include in Volume I, a chapter on Power Generation. According to information obtained from the Old Corner Book Store, 50 Bromfield Street, the price of the book is expected to be $5 per copy.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

September 23, 1936. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers' Club, and was called to order by the President, H. K. Barrows. One hundred and ten members and guests were present. Ninety-two persons attended the buffet supper.

The President announced the deaths of the following members: Frank S. Hart, elected a Member May 19, 1886, died July 11, 1936; Edward H. Kitfield, elected a Member February 17, 1904, died June 26, 1936; Frank B. Rowell, elected a Member April 17, 1889, died May 31, 1936.

The Secretary reported on the election of new members, as follows:

Elected June 17, 1936:

Grade of Student: Orland T. Prichard, Frank L. Soule, John A. Sweeney.

Elected September 23, 1936:

Grade of Member: Harry I. Wright.

Grade of Associate: Samuel J. Tomasello.

Grade of Junior: Harry A. Carlson,* Harry R. Cattley,* Francis H. Dutra,* Kenneth T. Hallett,* Ernest Jacoby, Jeremiah E. B. Jennings, Alan K. Willard.*

The President then introduced the speaker of the evening, Mr. Joseph P. Dever, Chief Engineer and Director, Sewer Division, Metropolitan District Commission, who gave an interesting talk on the "North Metropolitan Sewerage Problems." The North Metropolitan Relief Sewer, or the Aberjona Sewer, as it is popularly called, is the largest of the 348 projects under construction or already completed in Massachusetts under the Public Works Administration.

The district served by this sewer is an important residential and industrial center. The present sanitary sewers are overloaded, and overflow into streets and streams, creating conditions not in keeping with the otherwise high-class character of the communities and detrimental to public health. This is particularly noticeable in the Aberjona Valley District, an important division of the Metropolitan Park System.

* Transfer from grade of student.

The upper limit of the sewer is in Stoneham, near Main Street. The line passes into Woburn between Maple and Hill streets, follows the Aberjona River and the Boston & Maine Railroad through Winchester, the Mystic River Valley through Medford, and discharges into the Mystic River below Craddock Dam, a total length of about ten miles.

A question period followed the talk. Moving pictures were used to indicate some of the conditions existing before the project was undertaken.

The meeting adjourned at 8.45 P.M.

EVERETT N. HUTCHINS, *Secretary.*

OCTOBER 21, 1936. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club, and was called to order by the President, Prof. H. K. Barrows. Seventy members and guests were present. Sixty persons attended the buffet supper.

The Secretary reported the election of the following new members:

Grade of Student: Robert Henry Lewis, Anton E. Hittl, John B. Cadogan, Arthur E. Gibson, Harold E. Sanford.

The President then introduced the speakers of the evening, Mr. Carroll A. Farwell, member of the firm of Fay, Spofford & Thorndike, Consulting Engineers, and Mr. Howard J. Williams of the same office, who gave talks on the "Rouses Point Bridge at Lake Champlain."

Rouses Point Bridge is now under construction across the extreme northerly end of Lake Champlain. It consists of a bridge structure about 1,800 feet in length and filled approaches about 4,000 feet in length. The bridge consists of twelve simple pony truss spans and a swing drawbridge with through trusses, all supported on concrete piers, which rest either directly, or through wooden piles, on hardpan or ledge.

The project is being carried out by the Lake Champlain Bridge Commission, an interstate body, and is financed by means of a Federal P. W. A. grant and a private loan.

Fay, Spofford & Thorndike are the designing and supervising engineers.

Tolls will be collected to cover operating expenses and amortization of indebtedness.

An informal question period followed the talk. The talks were illustrated with lantern slides.

The meeting adjourned at 8.45 P.M.

EVERETT N. HUTCHINS, *Secretary.*

Designers Section

MAY 13, 1936. — A meeting of the Designers Section was held this evening at 6.40 P.M., in the Society Rooms in Tremont Temple.

Mr. John R. Nichols, Consulting Engineer, spoke on the "Unusual Features of Structural Design Involved in the Design of the Harvard Astronomical Observatories."

The talk was illustrated with the reflectoscope, and described the design of the turret-top observatories constructed for Harvard University in South Africa, and at Harvard, Massachusetts. These structures, revolving on tracks, and with a continuous aperture slot in half the roof and down one side, permit the 60-inch reflecting telescope to be directed and held at any point in the sky above 15 degrees from the horizon. The turret shape was evolved and first used for these structures.

The attendance was fourteen, and the meeting adjourned at 8.15 P.M.

ANTHONY S. COOMBS, *Clerk.*

OCTOBER 14, 1936. — The first meeting of this season was held at the Society Rooms, and was called to order by the Chairman, Paul F. Howard, promptly at 6.30 P.M.

The minutes of the May meeting were read and approved.

The Chairman then presented the speaker, Mr. Irving B. Crosby, Consulting Geologist, whose subject was "Engineering Geology of the Passamaquoddy Project."

The talk was illustrated by lantern slides. Mr. Crosby outlined the various Passamaquoddy schemes in general, and then explained the geological conditions

which affected the choice of dam sites and storage reservoirs, and also the available sources of stone, sand and gravel.

A discussion period followed, and the meeting closed at 8.10 P.M., with an attendance of forty-five.

ANTHONY S. COOMBS, *Clerk.*

Highway Section

MAY 27, 1936. — The regular quarterly meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening in the Society Rooms in Tremont Temple.

In the absence of the Chairman, the meeting was called to order by the Vice-Chairman, Mr. A. J. Bone, at 7.40 P.M. As there was no other business to come before the meeting, Mr. Bone introduced the speaker of the evening, Mr. E. F. Copell, Traffic Engineer of the Massachusetts Department of Public Works, who gave an interesting talk on "Recent Developments in Safety Traffic Engineering."

Traffic engineering includes suspension of traffic control devices, the assembling and interpretation of accident data, the determination of measures to reduce hazards and prevent accidents, the direction of traffic on the highways, the collection of data on traffic volumes, and routing and co-operating with and co-ordinating law, and the efforts of local law-enforcing agencies.

Following the talk there was a discussion period.

The meeting adjourned at 8.55 P.M.

There were thirty members present.

T. C. COLEMAN, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[October 20, 1936]

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

BLANK, WESLEY H., Melrose, Mass. (Age 41, b. Winchester, Mass.) Was graduated in 1916 with degree of S.B. in civil engineering at Massachusetts Institute of Technology; assistant instructor in bridge design, 1922; 1916–18, with McClintic-Marshall Company, detailing, estimating, design and shop; 1918–19, lieutenant, United States Army, anti-aircraft artillery; 1919, 1920, 1921, assistant chief engineer of design, Shoemaker Bridge Company, Pottstown, Pa., preparing designs and estimates, in competition with other fabricating plants of building and bridge projects; 1922, instructor at Technology; 1923, 1924 and part of 1925, with Stone & Webster, Inc., executive engineer in charge of three projects, — Westinghouse Lamp Works, Belleville, N. J., Jeffrey Manufacturing Corporation, Columbus, Ohio, and Tampa National Bank, Florida; 1925–33, plant engineer and in charge of drafting room, sales, estimates and costs, with Babcock-Davis Corporation, South Boston, Mass.; 1933–35, private engineering during depression, including several municipal projects; garbage plant and police revolver range at Melrose; investigations for Boston plants on engineering problems; designed warehouse and miscellaneous buildings; bridge work with Boston & Maine Railroad, designing and estimating

several railroad bridges and piers; at present with Maurice A. Reidy, Boston, designing Calvin Coolidge bridge, 1,350 feet, across Connecticut River at Northampton to Hadley; also Pepperell bridge over Nashua River. Refers to *H. K. Barrows, C. B. Breed, K. R. Garland, C. M. Spofford.*

BOGUSLAVSKY, BORIS W., Cambridge, Mass. (Age 27, b. Vladivostok, Russia.) Was graduated from University of Washington, College of Engineering, with degree of B.S. in civil engineering, in March, 1934, and degree of M.S. in civil engineering in June, 1935; September, 1936, at Massachusetts Institute of Technology as a graduate student, pursuing studies leading towards degree of Sc.D. In 1934–35, employed as teaching fellow at University of Washington in civil engineering department; September, 1935, to September, 1936, with bridge department, city of Seattle, Washington, as structural designer. Refers to *W. M. Fife, G. Gilboy, C. M. Spofford, G. H. Strandberg.*

CLARK, LESTER M., Dorchester, Mass. (Age 26, b. Dorchester, Mass.) Was graduated from Mechanic Arts High School, 1927, and Northeastern University with degree of B.S. in civil engineering in 1931; 1927, clerk in maintenance department, Hunt Spiller Manufacturing Company; 1929, steel worker on hydro-electric plant, Connecticut River Development Company; transitman with E. M. Brooks, C.E., and G. A. Haskins; 1930–31, office and field work, town of Needham engineering department; 1932–34, general engineering work; 1935, construction engineer with G. P. Carver Engineering Company; 1936, construction engineer, Carter Coal Company and Public Works Department, United States Navy; now, engineering draftsman and estimator, Public Works Department, United States Navy. Refers to *Henry Brask, W. H. Fitzgerald, L. C. Hollis, N. M. Winch.*

FISHER, DEXTER O., Dorchester, Mass. (Age 26, b. Dorchester, Mass.) Attended Dorchester High School three years; Central Evening High School two years; was graduated from buildings course at Lowell Institute (under auspices of Massachusetts Institute of Technology) in 1931; advanced courses at Lowell Institute at Massachusetts Institute of Technology in following subjects: advanced concrete design, applied mechanics and analytical geometry, advanced testing materials laboratory, advanced structural design, hydraulics, heat treatment and metallography; 1926 to August, 1929, with Stone & Webster; August, 1929, to date, with Metropolitan District Water Supply Commission as senior civil engineering draftsman. Refers to *C. L. Coburn, L. M. Gentleman, K. R. Kennison, F. E. Winsor.*

LEDDEN, JOHN BERNARD, Fitchburg, Mass. (Age 23, b. Fitchburg, Mass.) Attended St. Bernard's High School, Fitchburg, 1926–30, and graduated as a member of the technical course; attended Norwich University, 1931–35; and graduated with B.S. in chemistry; August, 1935, to date, on Nashua River Valley survey as chemist on water analysis. Refers to *H. B. Allen, G. P. Edwards, R. A. Marble, C. E. Pethybridge.*

For Transfer from Grade of Junior

O'CONNOR, EDWIN JOSEPH, Danvers, Mass. (Age 28, b. Danvers, Mass.) Graduate of Danvers High School 1925, and Northeastern University, with degree in civil engineering, in 1931. During co-operative period worked for Aspinwall & Lincoln. In 1931–33, with E. C. Blanchard & Son, Danvers, Mass., as estimator; January, 1934, to date, transitman, chief of party, and supervisor Massachusetts Geodetic Survey, Public Works Department, Boston, Mass. Refers to *H. B. Alvord, C. O. Baird, L. A. Chase, J. H. Lowe, C. A. Wolfrum.*

SCHOFIELD, LLEWELLYN T., Holliston, Mass. (Age 29, b. Gaspereau, N. S.) Was graduated from high school, Weston, Mass., 1925, and Northeastern University, with degree of B.S. in civil engineering, 1929. Co-operative work: 1925–29, with Rowland H. Barnes and Henry F. Beale, C.E., Waltham and Newton, Mass., as rodman, transitman and chief of party;

1929–32, with George F. Clements, C.E., Hyannis, Mass., as chief of party and assistant engineer. Special engineering courses at Massachusetts Institute of Technology for three months in 1932; 1932 to date, private engineering offices in Framingham and Holliston, Mass. Refers to *N. Bearse, H. L. Burton, E. B. Cobb, A. M. Pillsbury, C. S. Richardson.*

ADDITIONS

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