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CONTENTS

PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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FLOOD FLOW FORMULAS

By H. B. Kinnison, Member*

(Presented at a meeting of the Hydraulics Section of the Boston Society of Civil Engineers held on May 2, 1945.)

Introduction

The age old expression, "as old as the hills", is applicable to the problems relating to floods and flood-flow formulas. The floods of this and bygone centuries were principally responsible for carving out of our land masses the valleys and flood plains of today. In the advance of civilization through the centuries, man has contended more and more with the tremendous forces of nature, one of which is expressed in the occasional floods that rush down river valleys and fill them from hill to hill.

As a result of the work of those floods, river valleys are particularly attractive to a number of human uses. The flood plains have been developed and enriched by the deposits of silt brought down by the rivers, thereby making the valleys ideal for agriculture. They provide easy grades for highways and railroads and convenient locations for homes and cities. The rivers themselves provide water for transportation, water power, city water supply, sewage disposal, fisheries, and many other human needs.

Owing to the many natural advantages possessed by the rivers and the valleys they occupy, there has always been a tendency for people to encroach upon the rivers and become co-occupants of the

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river channel. At times the river becomes an unfriendly bed fellow and, if not controlled, it attempts to force out or destroy all encroachments. At such times, human habitation is ruthlessly eliminated and structures built by men in the river bed and on the flood plains are subject to damage and destruction if not adequately designed. These structures include bridges, culverts, highways, railroads, factories, cities, homes, dams, water-supply and sewage-disposal works, and all of the other facilities that are commonly found encroaching on a river valley.

It behooves us, therefore, when designing any one of these various structures to give adequate consideration to the flood possibilities of the stream in whose valley the structures are to be built. The attempts of the engineers, present and past, to evaluate the flood-producing potentialities of a river are largely responsible for the formidable array of flood-flow formulas in our engineering literature.

Natural floods result from the integration of a series of natural phenomena intimately related to weather, climate, hydrology, topography, and geology. They are only in part the direct result of weather occurrences and are more unpredictable than New England weather. Our only way of predicting future floods is from a study of the records of the flood occurrences of the past. The records can be classified as follows:

(a) A flood record contained in continuous stream-flow records of considerable length at the site. This is the most desirable type of data and if of sufficient length (not yet determined), need not be supplemented by other data.

(b) Relatively short, or intermittent, stream-flow data supplemented, if possible, by high water marks. This type of data can be analyzed by means of the unit hydrograph method in conjunction with rainfall records. Practically all data now available are included in this class.

(c) Flood data contained in records of stream flow in a nearby basin of similar fundamental characteristics. Considerable care should be exercised in the application of flood records to another river basin as no two river basins are identical.

(d) No satisfactory stream-flow records or flood data at any place in the river basin. Only in cases such as the last two should resort be made to the use of general flood-flow formulas.

A flood-flow formula is a mathematical expression by means of which a determination can be made of the magnitude of probable future floods in a river basin. Practically all flood-flow formulas heretofore developed have been general in character and have employed coefficients of variable magnitude. They have been based on data covering relatively short periods of time or for one stream only. The collection in recent years of a mass of flood data has led to the general opinion among hydrologists that a general flood-flow formula is a poor substitute for a reliable record of flood flows covering a considerable period of time. Such records are becoming more numerous as the years go by.

General Flood Formulas

Flood-flow formulas of a general nature can be classified according to the purpose for which they were developed such as formulas for extreme floods, formulas for floods of a given frequency, and formulas for a specific application, such as for culverts and drains. Also formulas involving rainfall, and those for total runoff and formulas with the frequency relation expressed.

Formula for Extreme or High Flood

An example of this type of formula is $Q = CM^n$. This is the general formula for extreme floods in which Q is the discharge in c.f.s., C is a constant, and M is the area in square miles. The exponent n usually varies from 0.5 to 0.8. There are many such formulas listed in engineering handbooks. However, because of their limited significance and questionable value, they will not be enumerated.

Formulas with Flood Frequency Implied

Numerous formulas for floods of an implied frequency have been proposed, such as the modified Myer formula, and the Kuichling, Craig, and Burge formulas.

Modified Myer formula: $Q = 10{,}000\, P\sqrt{M}$ where $Q =$ peak discharge in c.f.s., $P =$ Myer percentage rating, and $M =$ drainage area in square miles.

The value of this formula, as well as many others of this type, depends upon the ability of the engineer to choose the proper percentage rating. Records of floods in the vicinity, and the known

characteristics of the drainage basin are used as a guide in the selection of the rating.

Kuichling formulas:

Frequent Floods: $$Q = \frac{44{,}000}{M + 170} + 20$$

Rare Floods: $$Q = \frac{127{,}000}{M + 370} + 7.4$$

In these formulas, Q = sec.-ft. per sq. mi. and M = drainage area in sq. mi. These formulas are based principally on records of rivers in India prior to 1900.

Craig formula: $Q = 440\, n\, B(\text{hyperbolic log } \frac{8L^2}{B})$ where Q = peak discharge in c.f.s., L = length of drainage basin in miles, B = average width of basin in miles, and n = a coefficient ranging from less than 1 to more than 2 depending on the rainfall and topography.

This formula was published in 1868 and was based on records of rivers in India. It represents the early efforts of engineers to introduce the physical characteristics of the drainage basin into the flood formulas.

Burge formula: $Q = 1{,}300 \frac{M}{L^{2/3}}$ where Q = peak discharge in c.f.s., M = area in square miles, and L = length of basin in miles.

This formula, which was also based on records of streams in India, is of early origin and is interesting historically because the length of the basin is included.

This group of formulas is given in order to illustrate the type of formulas developed generally prior to 1900 and to show the beginning of the use of the shape of the drainage basin in such studies. The flood data were in general very meagre and foreign to the United States. The Myer formula in its modified form has survived and has been widely used in the United States. It has been kept up to date by adjusting the percentage rating as more flood data were obtained.

Formulas for Culverts and Storm Drains

Among the best-known formulas that are used in the design of culverts and drains are those of Burkli-Ziegler, Talbot, and Fanning. Of these formulas, the Burkli-Ziegler is the only one that includes a term for rate of rainfall.

Burkli-Ziegler formula: $Q = RMC \sqrt[4]{\frac{S}{M}}$ where Q = discharge in c.f.s., R = maximum rate of rainfall in inches per hour, M = drainage area in acres, S = average slope of ground in feet per thousand, and C = a coefficient varying from 0.25 to 0.75 depending on the type of land being drained.

This formula is designed principally for urban areas, although it has been applied to rural areas as well.

Talbot formula: $A = C \sqrt[4]{M^3}$ where A = the area of the waterway in square feet, C = a factor depending on the topography and varies from 1/5 to 1, and M = area in acres.

The Talbot formula is based on observations in the Mississippi Valley, and has been used in railroad and highway design.

Fanning formula: $A = 0.23 \sqrt[6]{M^5}$ where A = the area of the waterway in square feet, and M = area in acres.

In this formula no allowance is made for variations in conditions which affect the flow.

Formulas of this type are useful only as rough guides in making an intelligent guess. They should be used only when no specific information exists, and only to the extent that the formulas and coefficients are known, from long usage and observation of results, to fit local conditions.

Formulas Involving Rainfall

Among the recent formulas developed by investigators are the following that take rainfall into account.

Grunsky formula:

$$q = \frac{3{,}200a\,R}{t^{1/2}} \text{ where } a = \frac{60}{60 + C\sqrt[3]{t}},$$

q = c.s.m., t = the critical time, in minutes, during the continuance of a rainstorm for the area under consideration, within which the rain

will produce the maximum rate of runoff, R = maximum rainfall for one hour, and C = a coefficient, varying from 0.5 to 250.

This formula was published by C. E. Grunsky in 1922, and is applicable to California streams (A.S.C.E. Trans., Vol. 85, 1922).

Lillie formula: $Q = VR\lambda\Sigma\,(\theta L)$ where Q = peak discharge in c.f.s., V = standard mean velocity of river in flood,

$$R = 2 + \frac{\text{annual rainfall}}{15}.$$

λ = a factor based on length of basin, and $\Sigma(\theta L)$ is proportional to the width of the basin.

This formula, published by G. E. Lillie, contains provision for both rainfall and shape of basin (Inst. Civil Eng., Vol. 217, 1924).

Pettis formula: $Q = CPW^{1.25}$ where Q = peak of flood in c.f.s., C = a coefficient representing the combined influence of other factors such as drainage basin characteristics, P = a rainfall coefficient, and W = average width of basin in miles.

The Pettis formula was published privately by C. R. Pettis in 1927 and was modified in June 1934 to $Q = C(PW)^{1.25}$

Besson formula: $Q_m = P_m T G_m A^x$ where Q_m = maximum flood flow in c.f.s., P_m = maximum precipitation during the flood period, T = factor representing topography, G_m = factor representing influence of ground conditions, A = drainage area, and X = an exponent to be determined for each drainage area.

This general formula by F. S. Besson (Military Engineer, Sept.-Oct. 1933) has rather involved methods of obtaining the various factors, and requires considerable flood-flow data to be useful.

Switzer and Miller formula: $Q = P_c C W^n$ where Q = the 24 hr. average flood in c.f.s., P_c = rainfall factor, W = width of basin in miles, and C and n are empherical constants.

This formula (Floods: Cornell Univ. Eng. Exper. Sta., Bull. 13, 1929) was developed by F. G. Switzer and H. G. Miller from a study of 47 rivers in different parts of the United States.

Formulas for Total Runoff

Following the very severe New England flood of Nov. 1927, the Boston Society of Civil Engineers appointed a committee of engineers to investigate the causes and extent of the flood and report their

findings. In this report* the committee published the following formula that involves drainage basin characteristics where a flood hydrograph of the river being studied is available.

Boston Society of Civil Engineers formula: $Q = C_f R \sqrt{A}$ where Q = peak flow in c.f.s., C_f = coefficient, based on drainage basin characteristics, R = total flood runoff in inches, and A = drainage area in square miles.

The maximum coefficient suggested for New England streams is $C_f = 1000$ with $R = 8$. Where flood data are available C_f can be replaced by the expression $\frac{1290}{T} A$, where T = the total flood period in hours. By this means the flood characteristics of a basin are brought effectively into the formulas.

Formulas with Frequency Relation Expressed

In a number of flood formulas primary consideration has been given to the frequency with which floods of any given magnitude may be expected on a certain stream. In the following formulas a direct relation has been expressed between the magnitude or intensity of flood peaks and the expected frequency of their occurrence.

W. E. Fuller (1914) formulas:

$Q_{ave.} = CA^{0.8}$; $Q = Q_{ave.}\,(1 + 0.8 \log_{10} T)$; $Q_{max.} = Q\,(1 + 2A^{-0.3})$.

$Q_{ave.}$ = average of the annual 24-hr. floods in c.f.s., Q = maximum 1-day flood in c.f.s., $Q_{max.}$ = peak discharge in c.f.s., A = drainage area in square miles, T = number of years in period considered, and C = coefficient, constant for a given point on a stream. (A.S.C.E. Trans., vol. 77, 1914.)

R. E. Horton formula: $q = 4021.5 \left(\frac{T^{0.25}}{A}\right)$ where q = flood "equaled or exceeded" in the average interval of T years, and A = drainage area in square miles.

This formula applies to Pennsylvania streams and is based on records prior to 1914. (A.S.C.E. Trans., vol. 77, 1914.)

E. W. Lane (1926) formula: $q = K(\log I + B)$ where q = discharge in c.s.m. which would be equaled or exceeded once in I years, K is a constant for the station in question, and B is a constant for New England streams. (A.S.C.E. Trans., vol. 89.)

*Journal of the Boston Society of Civil Engineers, Sept. 1930.

W. P. Creager formula (Hydroelectric Handbook, p. 55, 1927):

$$Q = CA^{0.5}\left[\frac{2 - e^{-.04}A^{0.3}}{3}\left(1 - \frac{\log 0.1T}{3}\right) + \frac{\log 0.1T}{3}\right]$$

where Q = peak flow in c.f.s., A = drainage area in square miles, C = base of Naperian Logarithms, T = frequency in years, and C = coefficient depending on characteristics.

The above are only a few of the great number of formulas developed by engineers in recent years to serve as a rough guide for the judgment of investigators. A more complete discussion of many of these formulas can be found in Water Supply Paper 771.

In his "Elements of Hydrology," p. 366, edition of 1928, Adolph F. Meyer states: "When meteorological and hydrological data are entirely wanting, so that the cause of floods in the given stream cannot be studied, the use of such formulas as Fuller's may be justifiable, in that they serve as a rough guide."

As Fuller's paper was presented in 1914, it can be stated that he could utilize only the flood data available at that time. The amount of flood data collected since then, however, has greatly increased and in the last few years complete records have been obtained on all of the recent great floods, many of which have been the greatest ever known. It is probable, therefore, that should Fuller undertake a similar study today he would arrive at a formula entirely different from that originally presented.

Most of the formulas contain factors or coefficients, the values of which cannot be precisely determined. The engineer using the formula must form his own judgment as to their value after taking into account his knowledge of the character and accuracy of the base data, the general character of the drainage basin involved, and the purpose for which the flood estimate is being prepared. After full consideration of all pertinent conditions, the engineer must choose a value for the coefficient that will determine the flood discharge. Too often it is the final value of discharge that is the basis of his judgment, the coefficient being merely a means by which he justifies his conclusions as to what he considers a reasonable figure. Under those circumstances he might as well have selected the value of the discharge in the beginning.

Flood Formulas for Massachusetts

From studies of recent great floods in New England, it is apparent that equal rainfall with similar intensities produce flood peaks in New England rivers that have a wide variation in magnitude. In fact, assuming equal drainage areas, we have as many different flood peaks as we have rivers. It is known that this variation is caused to a great extent by differences in drainage basin characteristics. The Boston office of the Geological Survey recognized the possibility of developing a method by means of which the magnitude of flood peaks could be determined for each drainage basin from a study of the drainage basin characteristics.

A cooperative project was initiated in cooperation with the Massachusetts Department of Public Works and the Works Progress Administration. The project was divided into four principal parts:

First, the determination of the magnitude of flood peaks of various frequencies at each stream gaging station in the State of Massachusetts and adjacent areas.

Second, the accurate measurement and evaluation of a large number of drainage basin characteristics.

Third, the determination of the relation between all significant basin characteristics and flood peaks of various magnitudes and the elimination from consideration of physical characteristics that did not correlate with the flood peaks.

Fourth, the determination of flood flow formulas for Massachusetts streams determined by the relationships developed from correlation curves between the significant physical drainage basin characteristics and the resulting flood peaks of various magnitudes and frequencies.

These flood-flow formulas were to be different from previous formulas in that they were to be without coefficients or other unknown quantities and could be determined precisely for any drainage basin in Massachusetts. All estimating of the value of coefficients and other unknown values was to be eliminated. Furthermore, they were to give the peak discharge for a flood of any frequency in any stream.

Flood Frequencies

In determining the magnitude of floods of various frequencies, it was necessary to designate floods of various frequencies by giving

them a name and also to designate their frequency in terms of recurrence interval. It was concluded that floods of four different magnitudes or frequency would cover the range satisfactorily. These are as follows: minor flood, or one that occurred with a 15-year frequency; major flood, or one with a frequency of 100 years; rare flood, or one that occurs with a frequency of 1,000 years; and the maximum possible flood, the frequency of which approaches an unlimited period.

If all stream flow records in Massachusetts were of sufficient length, the best method of determining the magnitude of the 15, 100, and 1,000-year floods would be from a frequency study of all floods that had occurred in each basin. However, flood records are so short that only the magnitude of the 15-year flood could be determined by means of the frequency study of past floods. In order to determine the magnitude of the 100-year floods on each stream, a study of the relationship between rainfall and runoff was made. Several rainfall records over 100 years in length were available for this study. The runoff resulting from storms having a 100-year frequency was determined and by means of the unit hydrograph the resulting flood peak was determined for each gaging station. This point was plotted on the frequency curve for each station and extended by means of logarithmic plotting through the point for a flood of 1,000-year frequency to the magnitude of the maximum possible flood. See Fig. 1.

The maximum possible flood was computed directly from the U. S. Weather Bureau's determination of the maximum possible precipitation occurring when conditions would create the maximum possible runoff. It is a flood that can be approached but can never be exceeded under our present concept of meteorology. It is the upper limit of all possible floods. Under present concepts of floods and their causes, it is recognized that there is an upper limit to the magnitude of possible floods on a drainage basin. It is just as important for an engineer designing flood control works or other hydraulic structures on a river to know the magnitude of the maximum possible flood at a given point on a river as it is for the designing engineer to know the ultimate strength of the materials with which he is building a structure. The magnitude of the maximum possible flood for each gaging station is determined by applying the proper unit hydrograph for that station to the maximum possible rainfall with complete runoff. Unit hydrographs have a tendency to have higher, sharper peaks as the

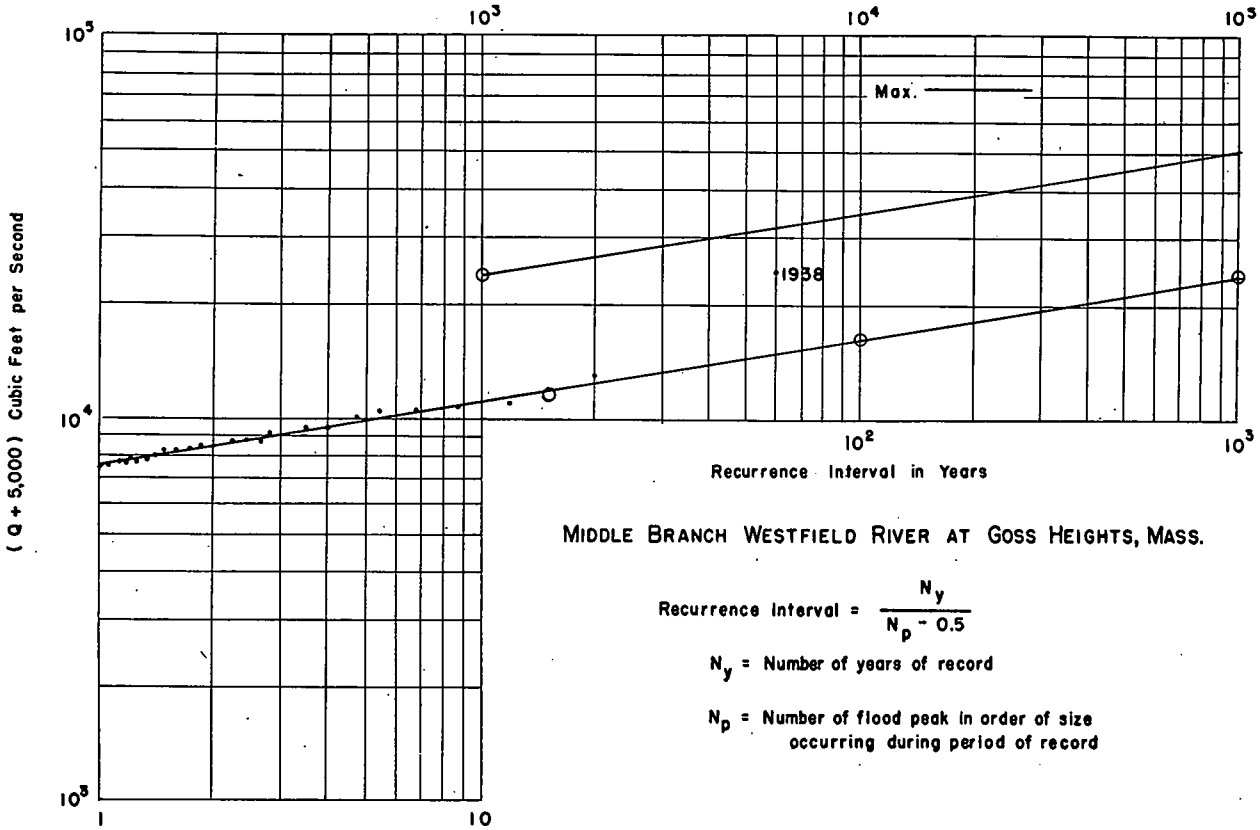
10⁵
10⁴
10³
1
10
10²
10³
10⁴
10⁵
Max.
1938
(Q + 5,000) Cubic Feet per Second
Recurrence Interval in Years
MIDDLE BRANCH WESTFIELD RIVER AT GOSS HEIGHTS, MASS.
Recurrence Interval = $\frac{N_y}{N_p - 0.5}$
N_y = Number of years of record
N_p = Number of flood peak in order of size occurring during period of record

FIG. 1.

magnitude of the flood increases. Proper care and judgment should be exercised in selecting the proper unit hydrograph for the specific point on the river under consideration.

Drainage Basin Characteristics

In conducting the study of drainage basin characteristics, all characteristics that could conceivably affect the runoff of flood waters on a drainage basin were considered. The following is a list of the characteristics determined and tabulated for each basin above each stream gaging station:

Areas.—(1) Drainage area; (2) area of lake, reservoir, and pond surfaces; and (3) area of swamps.

Altitude.—(1) Maximum altitude; (2) minimum altitude (altitude at the outlet); (3) median altitude at which 50% of the drainage area is higher and 50% is lower); and (4) mean altitude.

Slopes.—(1) East-west land slope; (2) north-south land slope; (3) average land slope; (4) slope of principal streams (a principal stream being defined as a watercourse that drains more than 10% of the total area of the basin); and (5) slope of tributary streams (tributary streams being defined as all watercourses contributing to a principal stream).

Configuration of Basin and Streams.—(1) Length of principal streams; (2) stream density (total length of all streams divided by the drainage area); (3) average distance runoff must travel to the outlet; (4) length of basin (twice average distance water must travel; (5) width of basin (drainage area divided by length of basin); ((6) form factor (length of basin divided by width of basin); and (7) branch index (length of stream channels, (selected according to an arbitrary criterion), divided by length of basin).

General Type of Vegetal Cover.—(1) Percentage of cropped land; (2) percentage of wooded land; and (3) percentage of grass land not cropped.

The work of determining these basin characteristics was extensive and involved many man years provided by a W.P.A. project operating under the supervision of the Boston office of the Geological Survey. U. S. Geological Survey topographic maps provided the basis for this study with the exception of vegetal cover which was based on information supplied by the National Resources Committee.

Correlations

The correlations were determined for floods of a given class by plotting on logarithmic paper all flood peaks of that class against the drainage basin characteristics. Some characteristics were found not to correlate and were discarded. Four significant characteristics were found to correlate and their influence was determined from the correlation equations. These characteristics were: M, or drainage area; L, or average distance water had to travel from its origin in the basin to point of measurement; S, or median elevation of basin above outlet; and a, or percentage of the area in lakes, ponds and swamps.

Flood Flow Formulas

The following equations were indicated by the correlations for the different classes of flood peaks:

For minor floods: $$Q = (0.000036\, s^{2.4} + 124)\, \frac{M^{0.95}}{a^{0.04}\, L^{0.7}}$$

for major floods: $$Q = (0.0344\, s^{1.5} + 200)\, \frac{M^{0.85}}{L^{0.5}}$$

for rare floods: $$Q = (0.0595\, s^{1.5} + 342)\, \frac{M^{0.95}}{L^{0.7}}$$

for maximum floods: $$Q = (0.128\, s^{1.6} + 1{,}800)\, \frac{M^{0.9}}{L^{0.7}}$$

In these equations Q = the peak discharge in cubic feet per second; a = percentage that lake pond and reservoir surface is to the total drainage area; s = median altitude of the drainage basin in feet above the outlet; M = drainage area in square miles; and L = average distance, in miles, which water from runoff over the basin must travel to the outlet.

Considering a river as a conduit passing flood flows, M can be considered as a measure of the volume of water to be discharged; S is a measure of the fall in the conduit; L is the distance the water travels or the length of the conduit; and a is a measure of the effect of storage on the shape of the flood hydrograph and should always be assumed to equal or exceed .05% even though the topographic map shows little or no water surface of lakes, ponds, or reservoirs.

It should be borne in mind that these formulas are intended for

TABLE I.—PHYSICAL CHARACTERISTICS AND FLOOD PEAKS

Outlet of drainage area	Maximum Discharge of Record			Physical Characteristics				Computed Flood Peaks in Cu. Ft. per Second			
	Cu. ft. per sec.	Date	Frequency in Yrs.	M (sq. mi.)	L (mi.)	s (ft.)	a (%)	From Flood Formulas Minor	Major	Rare	Maximum
N. Nashua R. near Leominster, Mass.	16,300	3-18-36	600	107	10.9	578	2.2	4,310	11,000	18,600	65,400
Concord R. at Lowell, Mass.	3,790	7-29-38		405	30.5	203	2.8	3,580	8,940	14,100	49,000
Ipswich R. near Ipswich, Mass.	2,610	3-15-36	30	124	13.8	91	1.1	1,940	3,710	6,080	23,900
Aberjona R. at Winchester, Mass.	——	——		23.3	4.2	109	2.3	900	1,680	2,990	12,600
Charles R. at Charles R. Village, Mass.	3,170	3-19-36		183	19.8	132	1.6	2,180	4,750	7,480	28,500
Neponset R. at Norwood, Mass.	——	——		35.2	5.7	153	2.9	1.090	2,280	3,970	16,000
Taunton R. at State Farm, Mass.[1]	3,050	4-14-35		211	14.0	90	1.9	3,110	5,810	10,000	38,600
Wading R. near Norton, Mass.	1,030	3-12-36	15	42.2	7.8	125	2.2	1,030	2,130	3,540	14,400
Blackstone R. at Worcester, Mass.	2,520	3-18-36		31.3	5.2	266	4.3	1,160	2,860	5,000	19,500
Blackstone R. at Northbridge, Mass.	——	——		139	13.4	341	4.2	2,780	7,530	12,700	44,400
Blackstone R. at Woonsocket, R. I.	15,100	7-24-38		417	19.6	365	3.2	6,390	16,600	29,100	97,200
Quinebaug R. at Westville, Mass.	——	——		93.8	10.6	285	3.1	2,060	5,410	8,970	32,900
Sugar R. at West Claremont, N. H.[2]	14,000	3-19-36		224	17.9	870	0.7	11,900	25,500	42,400	143,000
West River at Newfane, Vt.	52,300	9-21-38	170	308	22.2	1,225	0.3	29,100	46,200	76,100	258,000
Ashuelot R. near Gilsum, N. H.	5,220*	9-21-38		71.2	10.6	727	2.6	4,100	10,000	16,500	59,200
Ashuelot R. at Hinsdale, N. H.	18,000*	3-29-20		420	28.5	984	1.3	19,900	40,000	65,000	213,000
Otter Br. near Keene, N. H.[3]	6,130*	9-21-38		37.6	6.3	690	1.1	3,080	7,160	12,300	45,000
Otter Br. near Keene, N. H.[4]	6,130*	9-21-38		24.2	5.0	580	0.3	1,950	4,520	7,850	29,500
South Branch of Ashuelot River near Marlboro, N. H.	5,960	9-21-38	100	36.4	5.6	577	1.2	2,510	6,050	10,700	39,400
Millers R. near Winchendon, Mass.	8,500	9-22-38	600	83.8	7.9	260	4.0	2,180	5,280	9,320	34,600
Millers R. at S. Royalston, Mass.	——	——		186	12.0	285	3.0	3,680	9,000	15,900	55,800
Millers R. at Erving, Mass.	29,000	9-22 38	600	370	21.7	498	2.1	7,150	19,000	32,000	106,000
Sip Pond Br. near Winchendon, Mass.	2,630	9-21-38	500	19.0	3.9	173	3.2	800	1,720	3,020	12,600
Priest Br. near Winchendon, Mass.	3,000	9-21-38	1,700	18.7	5.8	243	0.3	710	1,660	2,670	10,800

*Peak reduced by abnormal storage.
[1]Excluding Assawompsett Pond.
[2]Excluding Sunapee Lake.
[3]Excluding Granite Lake.
[4]Excluding Granite Lake and Ellis Reservoir.

TABLE I.—(*continued*)

Outlet of drainage area	Maximum Discharge of Record			Physical Characteristics				Computed Flood Peaks in Cu. Ft. per Second			
	Cu. ft. per sec.	Date	Frequency in Yrs.	M (sq. mi.)	L (mi.)	s (ft.)	a (%)	Minor	From Flood Formulas Major	Rare	Maximum
East Br. Tully R. near Athol, Mass.	5,140	9-21-38	90	49.9	6.9	430	1.1	2,120	5,350	9,260	34,000
Moss Br. at Wendell Depot, Mass.	1,540	3-19-36	60	12.2	3.8	343	1.5	700	1,800	3,040	12,100
Deerfield R. at Charlemont, Mass.	56,300	9-21-38		362	27.2	1,446	1.3	40,100	60,200	97,100	326,000
Deerfield R. at Charlemont, Mass.[5]	56,300	9-21-38		178	13.6	1,192	0.2	23,400	35,700	61,400	212,000
North R. at Shattuckville, Mass.	——	——		88.4	10.1	978	0.2	9,890	17,500	30,300	107,000
Green R. at Greenfield, Mass.	——	——		89.4	12.6	868	0.4	6,690	13,800	22,500	79,600
Mill River at Northampton, Mass.	——	——		53.3	8.4	730	0.6	3,950	8,860	14,900	53,700
Ware River at Cold Brook, Mass.	14,000	9-21-38	2,000	100	9.2	315	2.4	2,550	6,450	11,300	40,800
Ware River at Gibbs Crossing, Mass.	22,700	9-21-38	2,000	201	20.0	510	1.5	4,420	12,200	19,500	65,700
Chicopee R. at Bircham Bend, Mass.	45,200	9-21-38	200	702	32.3	668	1.4	14,900	36,400	60,500	193,000
Swift River at West Ware, Mass.	7,590*	3-19-36		188	16.0	410	0.9	3,980	10,400	17,300	59,500
Quaboag River at W. Brimfield, Mass.	8,470*	9-21-38		149	16.2	460	2.6	3,380	9,410	15,400	53,300
Westfield R. at Knightville, Mass.	37,900	9-21-38	900	162	15.8	966	0.2	12,700	23,300	38,800	133,000
Westfield River near Westfield, Mass.	55,500	9-21-38	100	497	24.6	1,075	1.0	30,900	55,700	93,600	305,000
Middle Branch of Westfield River at Goss Heights, Mass.	19,900	9-21-38	1,300	52.6	10.3	998	0.05	6,620	11,600	18,800	67,900
W. Br. Westfield R. at Huntington, Mass.	21,800	9-21-38	200	93.7	11.2	1,020	1.0	9,900	18,600	31,200	112,000
Scantic River at Broad Brook, Conn.	7,360	9-21-38	150	98.4	11.5	387	0.2	2,750	6,760	11,200	40,100
Farmington R. near New Boston, Mass.[6]	18,500	9-21-38	500	75.3	7.7	728	0.8	5,690	12,400	21,900	78,000
Burlington Br. near Burlington, Conn.	645	9-21-38	80	4.1	1.8	180	0.05	380	700	1,230	5,470
Hockanum River near E. Hartford, Conn.	5,160	9-21-38	80	74.5	10.1	372	1.8	2,070	5,500	9,170	33,400
Housatonic R. at Coltsville, Mass.	6,400	9-21-38		57.1	7.1	648	1.0	3,850	8,980	15,700	56,200
Housatonic R. at Gr. Barrington, Mass.	11,500	9-22-38		280	24.4	712	1.7	8,310	20,700	33,200	109,000
Hoosic River excluding Hoosic Lake, at Adams, Mass.	5,080	9-21-38		31.4	4.5	860	0.05	5,450	9,400	17,000	63,300
N. Br. of Hoosic River at North Adams, Mass.	9,980	11-?-27	35	39.1	5.4	965	0.2	6,910	11,900	21,300	78,500

[5]Excluding Harriman Reservoir.
[6]Excluding Otis Reservoir.

application only to areas in Massachusetts or vicinity, and to areas ranging in size from one to about 700 square miles, although they probably could be used for areas up to 1,000 square miles. They have been developed from a study of all available precipitation and stream-flow data for this region. After additional years of stream-flow records have been added to the data now available, it is possible that the formulas should be modified.

Table 1 gives for a number of gaging stations in or near Massachusetts the computed drainage basin characteristics used in the formulas, the maximum flood peak of record, and the peaks of the four floods as determined from the flood formulas.

Computations

The application of the formulas to a given area is accomplished by determining the four applicable drainage basin characteristics from topographic maps of the drainage basin and substituting those values in the equations. In determining these characteristics the drainage basin is delineated on the topographic map and the area measured by means of a planimeter to obtain the value of M. The area of the swamps, lakes and reservoirs in the basin is measured in the same manner to determine the value of a. L is determined from the same maps by dividing the basin into a number of small basins of approximately equal size and measuring the distance from the center of gravity of each sub-basin to the point of measurement. These distances are then weighted against their area and the weighted distance for the basin determined. S can best be determined by dividing the area into square-mile blocks, then finding the median elevation for each square mile and computing their average. Each equation can be solved in a few minutes by the use of a pair of dividers applied to Fig. 1 in Mr. Bigwood's following discussion.

Rare Floods of Record

As previously mentioned, it is likely that the greatest floods of recent history have been experienced over portions of southern and central New England since 1927. Although the storms producing these record-breaking floods were extremely severe and covered large areas, yet at only four gaging stations were floods measured that equaled or exceeded the 1,000-year or rare flood at that station.

These stations are located as follows: Priest Brook near Winchendon, Mass.; Ware River at Cold Brook, Mass.; Ware River at Gibbs Crossing, Mass.; and Middle Branch of the Westfield River at Goss Heights, Mass. It is probable that other ungaged areas in the vicinity of these stations also experienced floods of similar magnitude. The exact location of the center of a great storm depends largely on the vagaries of the weather. Without doubt, by changing atmospheric conditions only slightly, floods of equal or greater severity could have been experienced on other nearby areas where floods of much lesser magnitude were experienced during the great storms of 1927, 1936, and 1938. The Ashuelot River, for example, could have experienced a rare flood as did the Ware River, yet the greatest flood recorded during the 38 years of record on the Ashuelot River has a frequency slightly less than a major flood. Any area in the region covered by these studies may experience a rare flood at any time when the conditions favor its occurrence. It is interesting to speculate as to what may happen when these valleys which have not yet experienced great floods eventually have their 1,000-year flood. We cannot say that it was not expected. Any one of these valleys heretofore immune from great floods is likely to be due for a rare flood before the basins now being protected by reservoirs receive their second rare flood. When they do, we may say that the reservoirs were located in the wrong valleys. Undoubtedly an attempt will be made at that time to build flood-protecting reservoirs in those valleys after the rare flood of the future has occurred and the damage has been done. Before the great flood of the future occurs, and the destruction is wrought, is the proper time to build the dams for the control of the future rare floods which will eventually occur in the heretofore favored valleys. Studies should now be made for the purpose of determining drainage basins in which extensive flood damages would result if floods of rare magnitude were experienced. Reservoirs should now be built where possible flood damage would warrant. If a basin never has experienced a rare flood, that fact should not be considered conclusive evidence that it never will, because in the natural course of events it may be the next one to be affected. Fig. 2 shows the relation between the maximum flood of record and the computed rare flood at the same location. This indicates that many of the gaging stations are located on streams on which a record of a rare flood has not yet been obtained. It also shows the variation between rare floods as

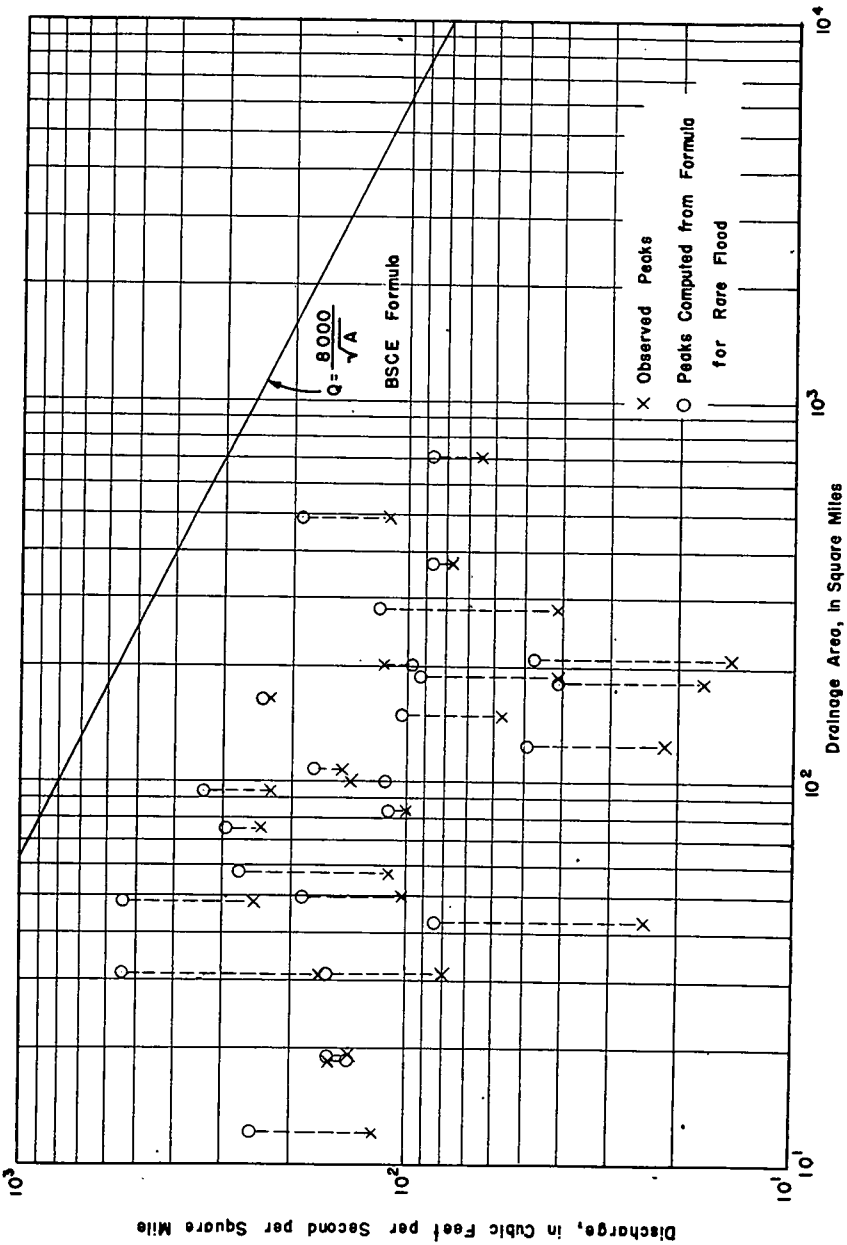
Q = 8,000 / √A
BSCE Formula
× Observed Peaks
○ Peaks Computed from Formula for Rare Flood
Drainage Area, in Square Miles
Discharge, in Cubic Feet per Second per Square Mile
10¹
10²
10³
10⁴

FIG. 2.

computed for the various gaging stations by means of the formula for rare floods and also the departure of these computed values from the maximum flood as computed by means of the formula proposed by the Boston Society of Civil Engineers.

Practical considerations must govern the design of flood-control works. Some of these considerations are the cost of providing flood protection in relation to the value of the property to be protected; the nature of the improvements located in the river valleys below the reservoir; the frequency of the floods for which the control works are designed; and the possibility of the loss of human life in the valley downstream.

Acknowledgments

Grateful acknowledgment is made to the work of Bruce R. Colby for his careful and able analysis of the data on which this paper is based and to Charles C. McDonald for his able assistance in arranging the material for this paper. The study in Massachusetts was conducted in cooperation with the Department of Public Works, General Richard K. Hale, Director, Division of Waterways. The study of drainage basin characteristics was outlined by the Division of Water Utilization, U. S. Geological Survey, R. W. Davenport, Chief.

APPLICATIONS OF THE KINNISON-COLBY FLOOD FORMULAS

By B. L. Bigwood*

(A discussion of Paper by H. B. Kinnison at meeting of Hydraulics Section, Boston Society of Civil Engineers, held on May 2, 1945.)

The Kinnison-Colby formulas, obviously, are the fruits of a great deal of coordinated effort and thought. Highly creditable to all concerned is the evident, original foresight which lead to the computation and compilation of the mass of drainage basin statistics, or "characteristics", which are primary elements in the formulas. The subsequent work on derivation of flood magnitudes and frequencies, and the correlation of these to the basin characteristics, represent much diligent individual research and thought on the part of Messrs. Kinnison and Colby. That basin characteristics—some of them at least—are major controlling factors in the magnitude of flood discharges from our streams, has, of course, been widely recognized, but it seems doubtful that few previous investigators have gone as far into the intricacies of this subject—and come out with as much—as have Messrs. Kinnison and Colby. Time may be expected to weigh the full merits of their work, after study, trial and experiment with the formulas have established their strengths, and if any, their weaknesses.

Messrs. Kinnison and Colby, with every good reason, warn that the formulas should not be used outside of Massachusetts without careful analysis of stream-flow data in the area under consideration. However, it seemed probable that hydrologic conditions in Connecticut are, in general, about as closely analogous to Massachusetts conditions as may be found and, therefore, the formulas could be expected to apply to Connecticut watersheds with some degree of reliability. Proceeding on this basis, application of the formulas was made to seventeen watersheds in Connecticut—seven in the Thames River basin, four in the Connecticut River basin, five in the Housatonic River basin, and to the Quinnipiac River basin. Base data and results

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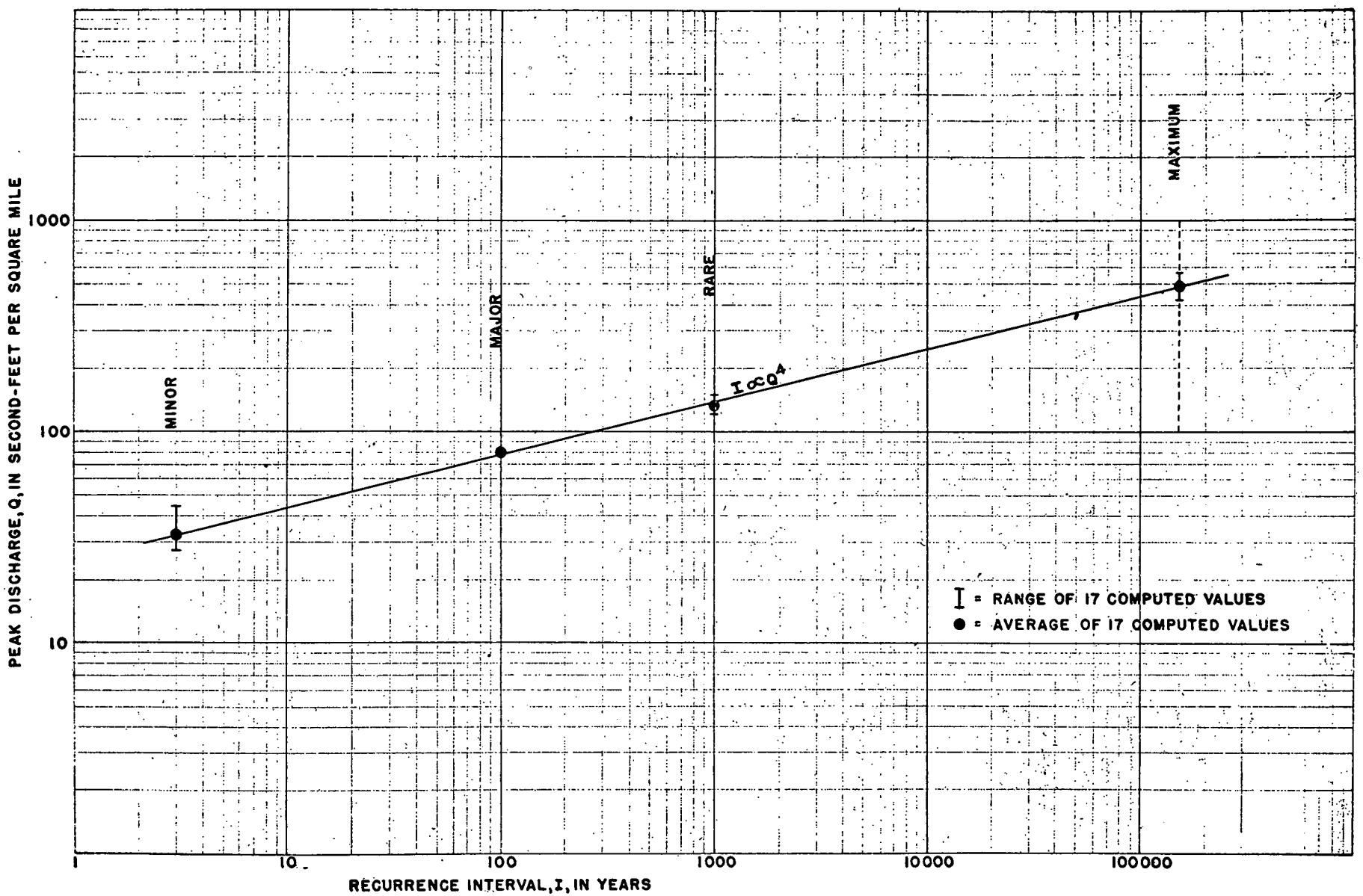

FIG. 2.—THE APPARENT STANDARD FREQUENCY DISTRIBUTION OF KINNISON-COLBY TYPE FLOODS, FOR 17 CONNECTICUT STREAMS.

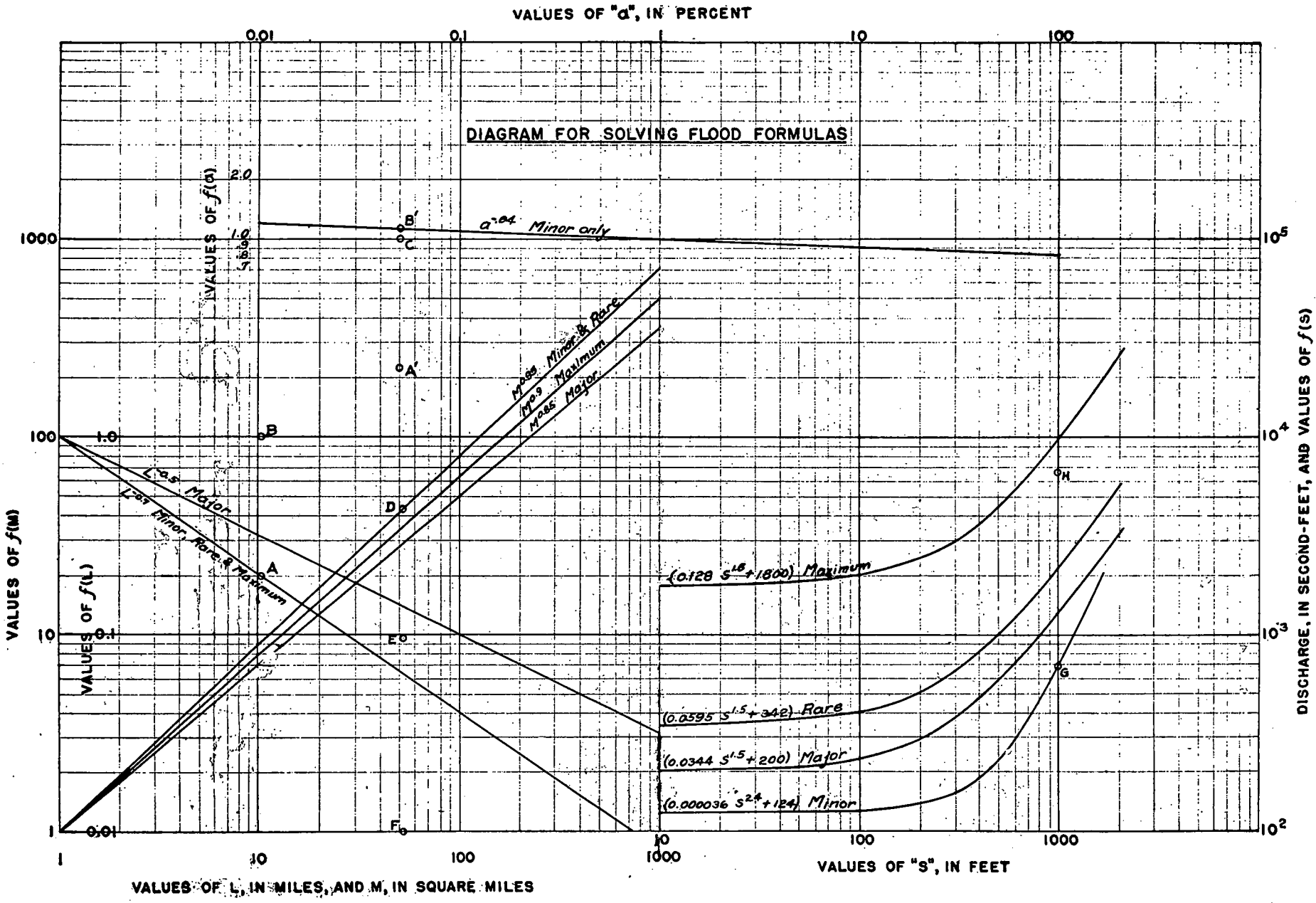

FIG. 1.—DIAGRAM FOR SOLVING FLOOD FORMULAS.

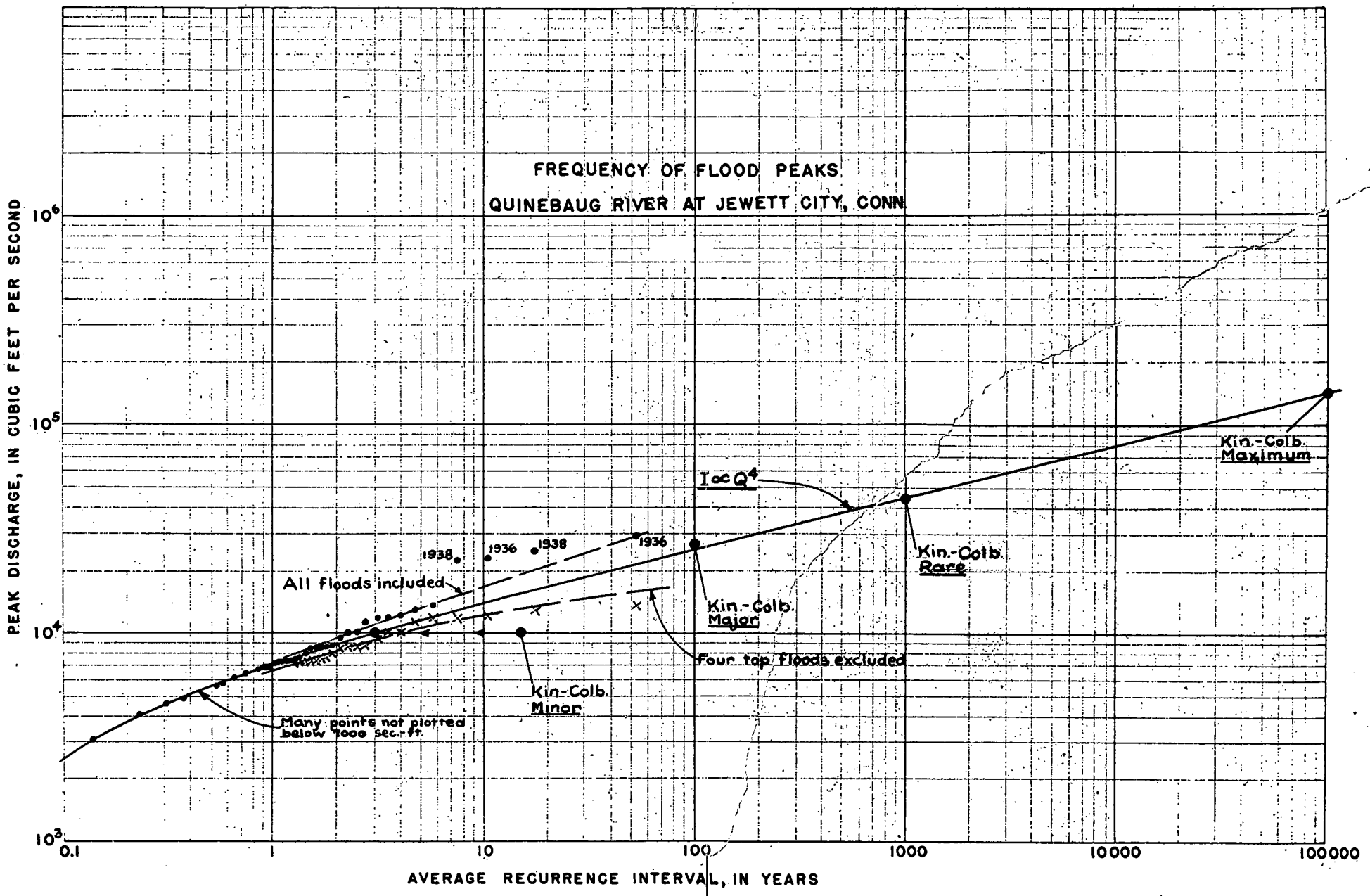

FIG. 3.—FREQUENCY OF FLOOD PEAKS, QUINNEBAUG RIVER AT JEWETT CITY, CONN.

obtained are given in Table I. For the Farmington River at Riverton, Pequabuck River at Forestville and Park River (conduit inlet) at Hartford, valves of "L", "s" and "a" were computed in the Hartford office of the Geological Survey in connection with the preparation of this discussion. For all other Connecticut streams listed in Table I, these physical characteristics were computed contemporaneously with those for many other New England streams by personnel of the Works Projects Administration under the supervision of the Boston office of the Survey. A forthcoming water-supply paper of the Geological Survey will contain these data.[1]

An early finding in connection with the application of the formulas was that a single log.-log. diagram can be prepared, similar to Fig. 13 in the authors' article in March, 1944, Proceedings of the American Society of Civil Engineers, from which, by use of a pair of dividers, flow values can be determined with time-saving ease and rapidity for any class of flood. Such a diagram is shown in Fig. 1. Using the data for Middle Branch of Westfield River at Goss Heights, Mass., the determination of the Kinnison-Colby minor flood for this stream may be made from the diagram as follows:

Physical characteristics: $L=10.3$; $a=0.05$; $M=52.6$; $s=998$.

1. Set one point of dividers at $L = 10.3$ on curve for $L^{-0.7}$, or at point marked "A" in Fig. 1. Expand dividers to $L = 10.3$ at $f(L) = 1.0$, or to point B.

2. Lay off distance AB vertically downward from $a = 0.05$ on curve for $a^{-.01}$; shown as A'B' in Fig. 1. Holding at A' close dividers to $a = 0.05$ at $f(a) = 1.0$, or to point C.

3. Lay off distance A'C vertically downward from $M = 52.6$ on curve for $M^{0.95}$; shown as DE on Fig. 1. Rotate dividers 180° about point E and expand to $M = 52.6$ at $f(M) = 1$, or to point F.

4. Lay off distance EF from $s=998$ on curve for "$0.000036\ s^{2.4} + 124$"; shown as GH on Fig. 1. Read value of point H in second-feet on discharge scale. H = 6,600 second-feet. Q.E.D.

Use the dividers in similar manner and sequence with curves for major, rare and maximum floods, to determine the discharge values for those floods, except that step No. 2 is eliminated since characteristic "a" is considered to have negligible effect in these floods. The distance (such as AB) determined under step No. 1 is then used

[1]Topographic Characteristics of Drainage Basins and Their Hydrologic Significance, by W. B. Langbein and others.

TABLE I.—PHYSICAL CHARACTERISTICS AND FLOOD PEAKS

Outlet of drainage area	Yrs. of Record	Maximum Observed Peak Discharge C.f.s.	Per sq. mi.	Date	Physical Characteristics M (sq. mi.)	L (mi.)	s (ft.)	a (%)	Kinnison-Colby Flood Peaks Minor C.f.s.	Per sq. mi.	Major C.f.s.	Per sq. mi.	Rare C.f.s.	Per sq. mi.	Maximum C.f.s.	Per sq. mi.
Willimantic River near S. Coventry, Conn.	13	14,900†	123	9-21-38	121	14.7	448	1.5	2,950	24	8,150	67	13,100	108	46,700	386
Shetucket River near Willimantic, Conn.	17	52,200	130	9-21-38	401	17.0	460	1.0	8,750	22	21,700	54	38,200	95	127,000	317
Hop River near Columbia, Conn.	12	6,450	84.6	9-21-38	76.2	9.3	325	1.2	2,050	27	5,300	70	8,900	117	32,800	430
Natchaug River at Willimantic, Conn.	14	27,200†	161	9-21-38	169	13.7	424	0.8	4,200	25	10,900	65	18,200	108	63,000	373
Quinebaug River at Jewett City, Conn.	26	29,200	41.1	3-19-36	711	31.6	473	2.3	10,000	14	26,800	38	44,500	63	142,000	200
Five Mile River at Killingly, Conn.	7	2,480	42.6	7-24-38	58.2	10.5	316	2.8	1,350	23	3,950	68	6,050	104	23,300	400
Yantic River at Yantic, Conn.	14	13,500	152	9-21-38	88.6	8.3	296	2.3	2,450	28	6,000	68	10,600	120	38,600	436
Farmington River at Riverton, Conn.	15	37,100	186	9-21-38	199*	10.2	825	0.9	14,500	73	28,500	143	53,500	269	180,000	905
Pequabuck River at Forestville, Conn.	4	3,800	84.1	9-38	45.2	6.5	472	0.6	2,250	50	5,450	121	9,600	212	36,000	796
Park River (conduit inlet) at Hartford, Conn.	8	5,700[a]	76.0	1-25-38	75	6.5	123	1.2	2,100	28	3,800	51	6,800	91	27,200	363
Salmon River near E. Hampton, Conn.	16	12,400	118	9-21-38	105	7.8	410	0.9	3,800	36	9,350	89	16,400	156	59,000	562
Quinnipiac River at Wallingford, Conn.	14	5,230	48.0	9-21-38	109	12.2	270	0.9	2,200	20	5,550	51	8,900	82	32,900	302
Tenmile River near Gaylordsville, Conn.	14	12,500	61.3	9-22-38	204	17.1	529	0.7	5,500	27	14,100	69	23,600	116	80,600	395
Still River near Lanesville, Conn.	13	4,410	64.4	9-22-38	68.5	12.0	358	1.4	1,650	24	4,600	67	7,200	105	26,800	391
Naugatuck River near Thomaston, Conn.	14	9,970	139	9-21-38	71.9	11.5	606	0.3	3,250	45	7,950	111	13,000	181	46,500	647
Naugatuck River near Naugatuck, Conn.	22	17,000	69.1	9-21-38	246	18.2	641	0.7	7,850	32	19,100	78	32,300	131	107,000	435
Leadmine Brook near Thomaston, Conn.	14	3,050	127	9-21-38	24.0	5.0	450	0.4	1,450	60	3,550	148	6,050	252	23,000	958

†Natural peak, after adjustment for dam failure upstream.
*Excluding Otis Reservoir drainage.
[a]Based on peak discharge at gaging station 0.9 mi. upstream.

directly in starting step No. 3. It seems quite likely that this use of a diagram has occurred to others also.

The results obtained for the 17 watersheds listed in Table 1, exhibit a consistency and reasonableness which sway one toward a favorable opinion as to their general reliability. For instance, the Farmington, Salmon and Naugatuck Rivers in Connecticut are considered "flashy" streams, relative at least to most other Connecticut streams, and for all classes of floods on these streams the formulas give considerably higher unit rates of peak discharge—a logical expectation. That the Pequabuck River at Forestville may also be classed as a "flashy" stream appears to be indicated by the magnitude of the unit rates of peak discharge obtained by application of the flood formulas to its drainage basin.

The flood discharge values obtained for the Farmington River at Riverton, Conn. (excluding Otis Reservoir), as given in Table I, compare very favorably with the authors' results for Farmington River at New Boston, Mass. (also excluding Otis Reservoir), and appear to demonstrate the consistency of the formulas in application to different sites on the same streams. For a New Boston minor flood the authors show a result of 76 second-feet per square mile; the result for Riverton, with an effective drainage area about 2½ times greater, is 73 second-feet per square mile. For the major flood at New Boston the authors give 165 second-feet per square mile; the figure of Riverton is 143 second-feet per square mile. For the rare flood the comparison is 291 second-feet per square mile against 269 second-feet per square mile, and for the maximum possible flood 1,036 second-feet per square mile against 905 second-feet per square mile.

The maximum flood of record at both of these stations occurred in September, 1938. The observed peak flow at New Boston for this flood was 18,500 second-feet and at Riverton, 37,100 second-feet. In each case this flood falls in the range between the Kinnison-Colby major and rare floods and the formula results indicate that in this range peak flows at Riverton should be around 2 1/3 times the peak rates at New Boston. The actual figures for the September, 1938, flood show a Riverton peak discharge rate 2 times that of New Boston, which again appears to check the consistency of results by the formulas as compared to actual occurrences.

The Willimantic, Hop and Natchaug Rivers are the major headwater feeders of the Shetucket River. The gaging stations on these

three streams measure the runoff from 366 of the 401 square miles of drainage area above the gaging station on the main Shetucket River. There is, then, a close relationship between the combined flow of the three headwater streams and the flow at the Shetucket River gaging station. A study of several floods of record in the Shetucket River basin indicates that the curve of relation between peak discharges at the Shetucket River gage and the summation of peak discharges for the same floods at the three headwater gaging stations is a straight line on log.-log. coordinates. The curve is well defined for floods up to about 55,000 second-feet. Applying a like procedure to the four computed Kinnison-Colby type floods for these same gaging stations, i.e., summing the computed flow rates in each flood class for the three headwater streams and plotting these values against the computed rate in each corresponding flood class for the main Shetucket River, a curve of relation is defined (by the four points) that is also a straight line on log.-log. coordinates, parallel to and within 6 per cent of the curve of relation based on recorded floods. Inconsistencies in computed and observed results represented by the 6-per cent spread between the two curves of relation seem of almost negligible import and it would appear that here again the flood formulas give very reasonable results in application to inter-related watersheds, judged on basis of actual, observed occurrences.

The designers of the Park River pressure conduit in Hartford undoubtedly delved deeply into the question of possible flood flow from that 75-square-mile watershed. Prior to construction of the conduit, model tests were made at the Alden Hydraulic Laboratory at Worcester Polytechnic Institute,[2] to confirm the computed discharge capacities under varying differential heads. The rating curves for the conduit, reproduced in the report of these tests, indicate that a flow of 28,000 second-feet may be conveyed at maximum desirable headwater elevation, with the Connecticut River at low stage (10 feet m.s.l. datum). For the same headwater elevation but with the Connecticut River at so-called flood stage (about 15.5 feet, m.s.l. datum), 26,000 second-feet would flow through the twin tubes, while with the Connecticut River at 27 feet (m.s.l. datum), about 21,000 second-feet could be carried. (The latter stipulations as to headwater

[2]See Report of Hydraulic Model Tests of Park River Conduit, Connecticut and Park Rivers, Hartford, Conn. Alden Hydraulic Laboratory, Worcester Polytechnic Institute, C. M. Allen, Hydraulic Engineer. War Department, Corps of Engineers, U. S. Army, Providence, R. I. May 1941.

and tailwater elevations with a computed carrying capacity of 19,700 second-feet constituted the controlling design requirements. Hence, the model test indicted that the discharge capacity of the conduit was somewhat in excess of the design requirements). At an extraordinary Connecticut River stage of 45 feet (m.s.l. datum) the conduit would still convey a Park River flow of 8,000 second-feet without exceeding maximum desirable headwater elevation. As shown in Table 1, the Kinnison-Colby rare flood for the Park River above the conduit inlet is 6,800 second-feet and the maximum possible flood is 27,200 second-feet. Thus, with the Connecticut River at the highest stage conceivable the conduit will convey a flow nearly 20 per cent greater than the Kinnison-Colby rare, or 1,000-year flood, while, with the Connecticut River at stages that are more likely to prevail the conduit capacity approaches, and under most favorable conditions, equals the Kinnison-Colby maximum possible flood. There appears to be, then, a reasonable accord between flood formulas and conduit designers as to the hydrologic possibilities and probabilities on this watershed.

Excluding the previously-mentioned Farmington, Salmon, Naugatuck and Pequabuck Rivers, the minor flood indicated by the formula, for most Connecticut streams, averages around 24 second-feet per square mile. The major flood averages about 60 second-feet per square mile, the rare flood about 100 second-feet per square mile, and the maximum possible flood around 360 second-feet per square mile. These values relate about as 1 to 2½ to 4 to 15.

The peak flows given by the formulas for the different flood classes indicate that actual floods of record on the streams in this section have very seldom equalled or exceeded the rare flood type. Of the 17 streams in Connecticut to which the formulas were applied, four have exceeded the computed rare floods—in September 1938. These are the Willimantic, Natchaug, Shetucket and Yantic Rivers, which drain the western side of the Thames River basin. In these instances the 1938 peak discharge exceeds the Kinnison-Colby rare flood by 14%, 49%, 37% and 27%, respectively.

Frequency of Floods

A complete study of flood frequencies has been made only in the case of the Quinebaug River at Jewett City, Conn. The 26-year record for this gaging station contains 192 flood rises in which the peak rate exceeded 3,000 second-feet (the approximate minimum annual flood of record).

From this complete study, and subsequent incomplete studies for a few other Connecticut streams, it seems fairly certain that, in Connecticut, the Kinnison-Colby minor flood has a recurrence interval of around three years, as against the 15-year interval found for Massachusetts streams. At Jewett City, the Quinebaug River has equalled or exceeded the computed minor flood twelve times in the 26-year period of record, giving this flood a recurrence interval of 2.3 years.

Preferably, this divergence should be checked by additional complete frequency studies of Connecticut records, before being accepted as fact. However, assuming that this finding is correct, it seems apparently that the frequency curve of depth of runoff, Curve C, as pictured in Fig. 7 of the authors' article in March Proceedings of the American Society of Civil Engineers, gives 15-year depths which are too low for many Connecticut streams. Or, putting it another way, the 15-year depth of runoff shown by Curve C. has a much shorter recurrence interval in Connecticut. Comparing the Jewett City flood record with that for Millers River at Erving, Mass., for example, bears out this apparent Connecticut divergence. The floods at Jewett City which on 12 occasions exceeded the computed minor flood, at Erving surpassed the minor flood for that stream on only two occasions. An actual variance in meteorological conditions over coastal areas, representative of Connecticut, and inland areas, representative of central New England, probably accounts for this situation in respect to the minor type flood.

On the other hand, the major, rare and maximum possible flood rates for Connecticut streams, as derived from the formulas, seem reasonable in comparison with observed flood magnitudes and with results for Massachusetts streams. Also, the standard recurrence intervals of about 100 and 1000 years for the major and rare floods, respectively, appear to fit Connecticut conditions with a degree of reliableness commensurate with Massachusetts findings. This may perhaps be accounted for in the fact that major and rare type floods usually are caused by meteorologic disturbances of wide extent which, therefore, should have nearly equal frequencies over large regions embracing several drainage basins.

The standard recurrence interval for the Kinnison-Colby minor flood on Connecticut streams appears, then, to be about 3 years, and for the major and rare floods 100 and 1000 years, respectively. Now,

in studying the derived frequency curves for the 17 trial streams in Connecticut it was found, rather surprisingly, that in nearly every instance, the recurrence interval, *I*, varied about as the fourth power of the discharge, *Q*, or vice versa, the discharge, *Q*, varied as the fourth root of the recurrence interval, *I*. Extending the derived frequency curves on this basis to the maximum possible flood rates, it was found that the standard recurrence interval for this flood class was between about 100,000 and 150,000 years in each case. In short, there appears to be a standard shape, or. form, to the derived frequency curves for these trial Connecticut streams.

This can be demonstrated by adjusting the curves to pass through a common point, as shown in Fig. 2. Expressing the discharge in second-feet per square mile, the point common to all curves was selected at "Q" equal to 80 second-feet per square mile and "I" equal to 100 years. The value 80 is the average 100-year (major) flood for the Connecticut streams considered. Each stream class-value was then multiplied by the coefficient which made its 100-year (major) flood equal 80 second-feet per square mile. The resulting class values for each of the 17 trial streams, when plotted on log.-log. coordinates against the standard recurrence intervals of 3, 100, 1,000 and 150,000 years, were found to group closely on a single straight line which best fitted the groups when drawn on a 1 to 4 power relationship of "I" to "Q".

The possibility indicated is that the defined portion of a flood-frequency curve for a given stream may be extended on the standard basis that at the higher frequencies "I" will vary as the fourth power of "Q". It works beautifully in the case of Quinebaug River at Jewett City, as shown in Fig. 3. There is, however, no particular advantage to be gained in resorting to this procedure, but I have mentioned it here at some length because it was a rather surprising feature of the application of the formulas to Connecticut streams, and may perhaps have some significance. Figure 3 demonstrates, too, that the Kinnison-Colby minor flood appears to have considerably less than a 15-year recurrence interval in the Connecticut area.

This about terminates the observations which I have to make regarding these flood formulas and their use. I have not studied them with near the completeness they require, and hence, this discussion is neither extensive nor comprehensive. I have, perhaps, one mild criticism, which is, that the authors, in my opinion, did not sufficiently

stress the fact that there is a limit to the area over which the formulas will apply. I made a rough esimate, for example, of the probable peak flow of the lower Connecticut River, on the assumption that the contributory discharges had poured from everyone of its tributaries at a peak rate equivalent to the Kinnison-Colby maximum possible flood. The resulting down-river discharge would do credit to the Mississippi. It was, I am sure, a wholly improper application of the formulas but it emphasizes the point made in my criticism.

In conclusion, I want to express my appreciation to the Hydraulics Section of your Society and its chairman, Mr. McDonald, for the courtesy extended me in the invitation to take part in this program. It has indeed been a pleasure and a privilege to meet with you and to join with Mr. Kinnison in this discussion. The subject redounds with credit to him and to his collaborator, Mr. Colby.

GAS ENGINE POWER FOR SEWAGE TREATMENT PLANTS

By Allen J. Burdoin, Member*

(Presented at a meeting of the Sanitary Section, Boston Society of Civil Engineers, held on October 3, 1945.)

The installation of gas engines in sewage treatment works for the production of power and heat from sewage gas has become common practice in large and medium sized plants having a considerable power load. Phenomenal savings have been reported due to the use of sewage gas for power production, and where gas engines have been installed in existing plants they have paid for themselves in a very few years. Sewage treatment plants are peculiarly constituted in that they can use not only the power produced by the engine but the heat rejected to the cooling water and to the exhaust, so that the efficiency of the engine as a producer of usable power and heat far exceeds the 25 to 28% thermal efficiency based on power output only. Diesel engines have proved to be economical prime movers for small municipal and industrial power plants where fuel has to be purchased, operators employed to care for the engines, and the heat in the engine jacket water is thrown away. It is no wonder, therefore, that gas engines have made such a record in sewage treatment plants where the fuel is a by-product, the operator is required anyway and looks after the engines in addition to his other duties, and the waste heat can be used to heat the digesters and buildings.

It should not be necessary to point out that the utilization of gas engines will not produce any better effluent, nor make the processing and disposal of the sewage sludge any simpler. However, the proper application of gas engines will grealty reduce the operating costs of sewage treatment which in many cases are a heavy burden on the community. In some instances this factor may be decisive in obtaining public approval of a proposed project. Whether or not to install gas engines in any particular case should be determined only after a detailed study of the cost of supplying the power requirements of

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the plant with gas engines or by purchased power, including the cost of adequate stand-by facilities in both cases to insure continuity of service. In some cases the electric power company has been willing to make a rate so attractive that the additional first cost of installing gas engines was not warranted. In other cases, especially in the smaller plants, the power load may be so small that the additional complication of gas engines is not justified. This is particularly true where an operator is on duty during the day shift only. The situation is complicated further by the fact that the amount of gas produced is usually not sufficient to produce all the power needed, except in the simpler plants providing primary treatment only; and when sufficient, it is seldom abundant.

This paper is an attempt to outline the various factors which must be considered in planning a gas engine installation. The production of power and the effect of the new dual fuel engines on the design of the power plant will be considered first, and will be followed by a discussion of various methods of waste heat recovery and utilization.

Power Requirements

The largest uses of power in sewage treatment plants are for sewage pumping, and for air compression for the activated sludge process. To these may be added the heat drying and incineration of sewage sludge, for although the power requirements are not large, the amount of gas required for fuel is so great in most cases as to leave hardly any for power production. The author does not know of any plant including incineration or heat drying of the sludge where the power requirements for both sewage pumping and aeration have been met by sewage gas engines, although this was proposed by Metcalf & Eddy for Long Beach, California. In this case the municipal gas plant was located adjacent to the site selected for the sewage treatment works, and it was contemplated that considerable quantities of city gas would be used as auxiliary fuel. The proposed Los Angeles plant contemplated both incineration of sewage sludge and gas engine driven blowers; but the sewage would not require pumping, and the plant was to utilize the modified aeration process rather than full activated sludge treatment.

Power Available from Gas Production

The stronger the sewage, the more power will be available per m.g.d. from gas production. Since the main pumping load is proportional to the quantity of sewage and not to the strength of the sewage, a plant with strong sewage is more apt to be self-sufficient from a power standpoint than a plant with weak sewage. The quantity of power available from the digestion of sewage solids is shown in Fig. 1. This is based on the assumption that the raw sludge contains

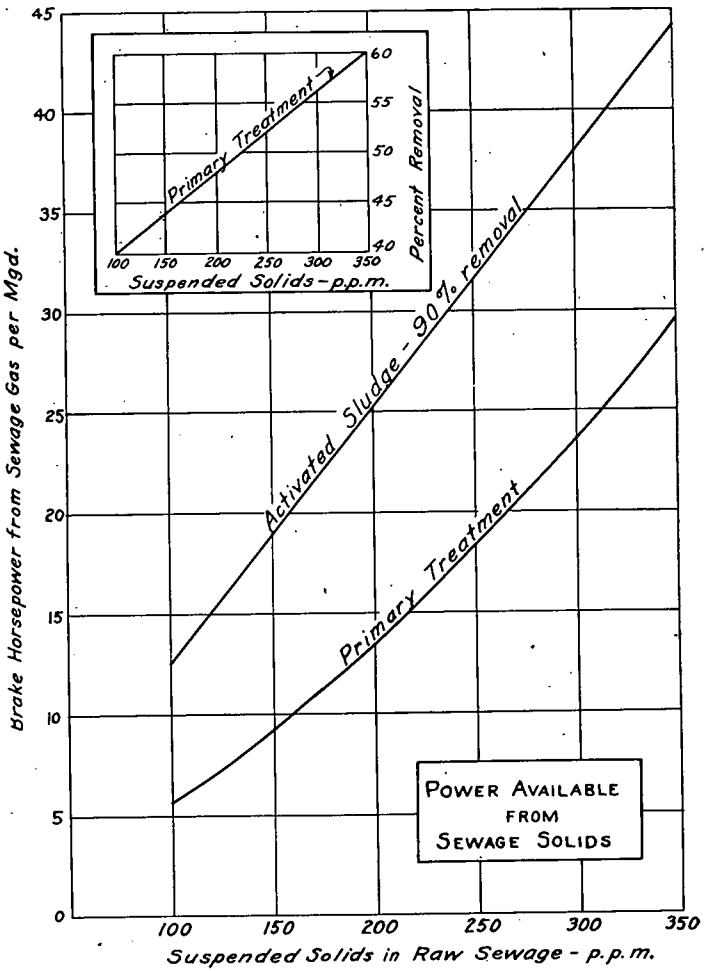

Fig. 1.

70% volatile solids, of which 60% are destroyed by digestion with the production of 16 cu. ft. of gas per lb. of volatile matter added to the digesters. Allowance is made for the fact that removal of suspended solids by primary treatment is greater for strong sewage than for weak sewage,—40% removal being assumed for weak sewage with 100 p.p.m. suspended solids, and 60% removal for strong sewage with 350 p.p.m. suspended solids. The figure shows that for primary treatment only with variation in strength of sewage from 100 p.p.m. to 350 p.p.m. suspended solids, the average power which can be developed will vary from 5.6 to 29.4 brake horsepower per m.g.d.; while for activated sludge treatment with digestion of the mixed raw and activated sludges, the power available will vary from 12.6 to 44.1 brake horsepower per m.g.d. These figures are based on gas with a low heating value of 600 B.t.u. per cu. ft., and 10,000 B.t.u. per horsepower hour, which is approximately correct for gas engines operating at or near full load. Where gas engines drive electric generators operating under part load these values must be reduced, and generator and motor efficiencies allowed for. An addition of 25% to the estimated power requirements will cover motor and generator efficiencies, but the reduction in power output under part load will depend on the engine load factor.

Power Requirements for Sewage Pumping

The power required for pumping will depend on the capacity, lift, and efficiency of the pumps. The proportion of gas power available which is required to take care of the pumping load will depend upon the strength of the sewage as well as upon the lift, and efficiency. Fig. 2 shows the horsepower required per m.g.d. by pumps with efficiencies varying from 50 to 80%, and with lifts up to 120 feet. The horizontal lines indicate the horsepower available from the digestion of primary sludge for sewages of different strength. The solid lines indicate the power available for direct drive, and the dotted lines the available power at the pump coupling for electric drive assuming the generators well loaded. This figure illustrates quite well the effect of the strength of the sewage. For example, if the sewage strength as measured by suspended solids is 100 p.p.m. the sewage can be lifted only 25 ft. with the most efficient engine driven pumps, whereas the lift can be 61 ft. for 200 p.p.m. and 134 ft. for 350 p.p.m. of suspended solids.

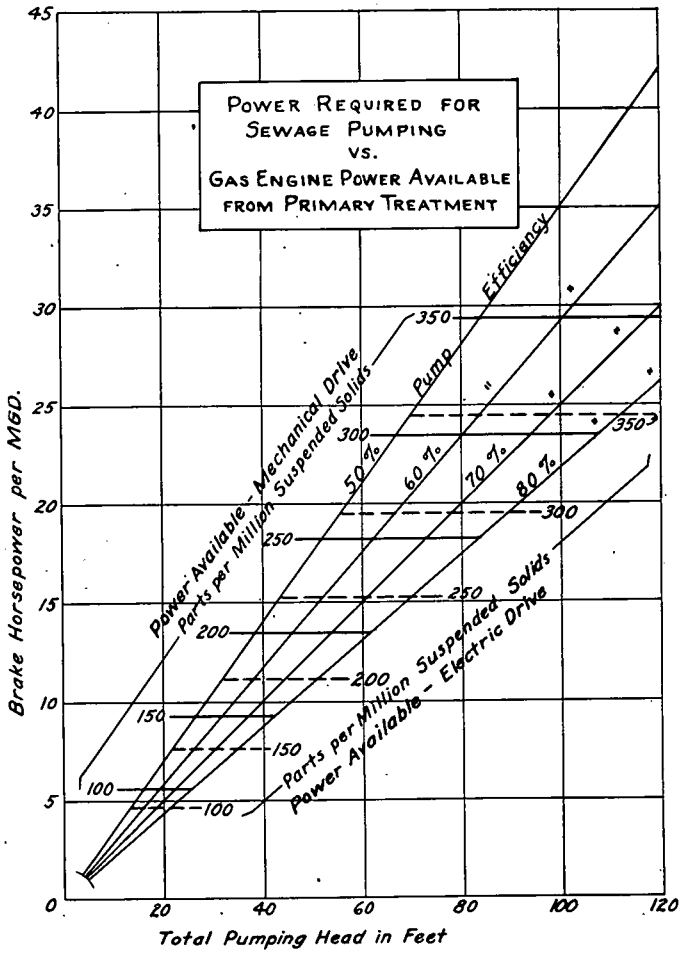

FIG. 2.

POWER REQUIRED FOR AERATION

The power required for aeration will depend on the depth of the aeration tanks which determines the air pressure required, and the air used per gallon of sewage. The latter varies within wide limits and depends on the tank design, detention period, sewage strength, degree of nitrification desired in the effluent, and the quantity and condition of the return sludge. In order to obtain prelimi-

nary figures for use in estimating power requirements, the author made up Fig. 3, which tends to show that the air requirements vary directly in a straight line relationship with sewage strength measured as p.p.m. of suspended solids in the raw sewage.* The data plotted were taken in large measure from "The Operation and Control of Activated Sludge Sewage Treatment Works" by Langdon Pearse and Committee, published in the January 1942 issue of the Sewage Works Journal. Using the average line from Fig. 3, the power required by the blowers is shown in Fig. 4 for discharge pressures varying from 6.0 to 8.0 p.s.i. gage, assuming an adiabatic efficiency of 65%. (The efficiency of blowers used in sewage treatment plants usually lies between 60 and 70%.) Fig. 4 also includes two lines showing the approximate upper and lower limits of power required for the spread of points plotted on Fig. 3.

The total gas engine power available is plotted on Fig. 4, and also the power available from the activated sludge solids alone. The scale of abscissae of Fig. 4 is calibrated in both cu. ft. of air per gallon of sewage and parts per million of suspended solids in the raw sewage. In cases where the average curve of Fig. 3 does not apply, one scale can be used to find the power requirements, and the other scale to find the power available. These curves indicate that sufficient gas should be available except under unusual conditions for direct connected gas engine driven blowers, if the gas is not used for any other purpose. If the air pressure can be kept low, and the amount of air kept down to the average curve of Fig. 3 or somewhat below, and the pumping lift is low, it may be possible to pump the sewage as well as compress the air by gas engine power. This is the case at Tallman's Island in New York City where the blowers are Roots-Connersville positive pressure blowers driven by direct connected Worthington gas engines, and the main sewage pumps are driven by Worthington gas engines through bevel gears. A connection with the gas company supplies city gas to the engines in case of a deficiency of sewage gas. The production of sewage gas is almost sufficient to supply the load, but some gas has had to be purchased for this purpose. All engine auxiliaries, which are electric motor driven, and all plant motors including the sewage pumps of the Powell's Cove Pumping Station (which are included in the same building and serve a

*This may not be true where the B.O.D. does not bear an approximately fixed relationship to the suspended solids.

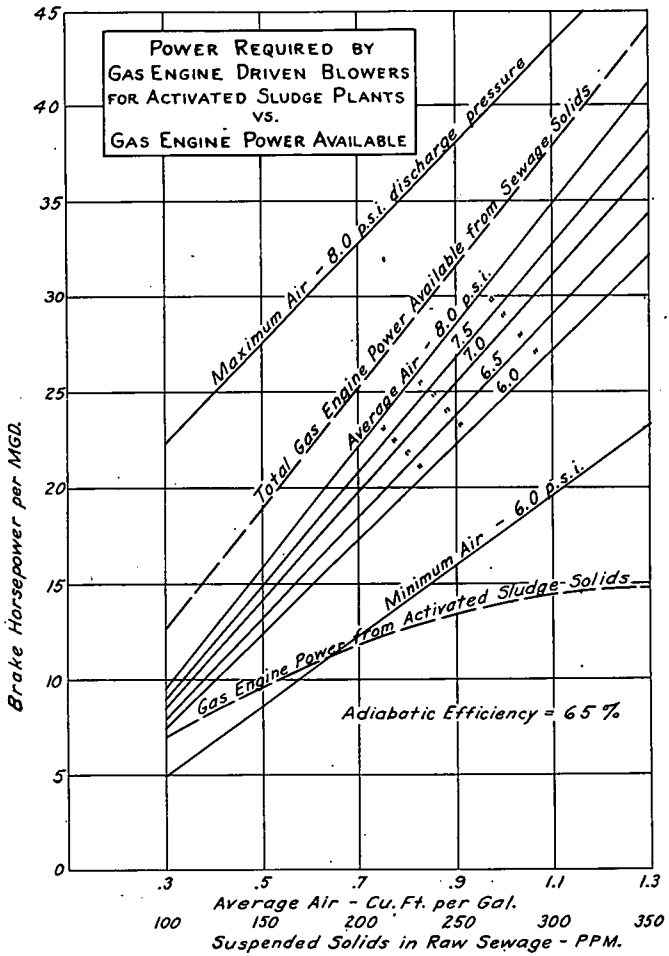

FIG. 4.

small drainage area) are supplied with electric power by the public utility. The total electric load amounts to about 20% of the main power load. The gas production at Tallman's Island amounts to 10.5 to 11 cu. ft. of gas per pound of volatile matter added, due in part to the large digestion capacity available with a sewage flow of only 15 m.g.d.

Heat Drying and Incineration of Digested Sludge

The gas requirements of sludge driers and incinerators for auxiliary heat are so great that, normally, gas engines are not considered when they are installed. Assuming 70% moisture in the filter cake, heat drying requires from 60 to 95% of the gas produced as auxiliary fuel. Incineration of the sludge may be self-sustaining without auxiliary heat, but usually some heat is required, which with wet cake may amount to 70 to 90% of the gas production. Many incinerator installations do not include digestion of the sludge, in which case there is no gas and oil is used as auxiliary fuel. Published data indicate that the power requirements may vary from 16 to 66 Kwh. per ton of dry solids.

Miscellaneous Motors and Auxiliaries

Sewage plants necessarily include a great number of small motors which operate only a few minutes or a few hours a day. The auxiliary motors will operate at a low load factor, rarely exceeding 20% of the installed horsepower or 30% of the main power load.

Peak Loads

The power system must have adequate capacity to provide for peak loads; but where large motors are attached to manually operated shredders or infrequently operated sludge pumps it should be recognized that the plant operator controls both power production and power use, and can, when necessary, curtail the use of equipment to cut down the peak load without detriment to the process. Gas may be stored in low pressure gasometers or high pressure holders to provide fuel for peak loads. A storage capacity equal to 6 to 12 hours' gas production is usually required.

Power Requirements Must Be Estimated

In any given problem the total power requirements must be studied carefully on the basis of performance curves submitted by the manufacturers for the main items of equipment, and an estimate of the power load of the auxiliaries. The above figures and diagrams have been given as an approximate guide, only, so that a quick determination can be made of the schemes which should be given detailed study; and also to outline the situation regarding the installation of

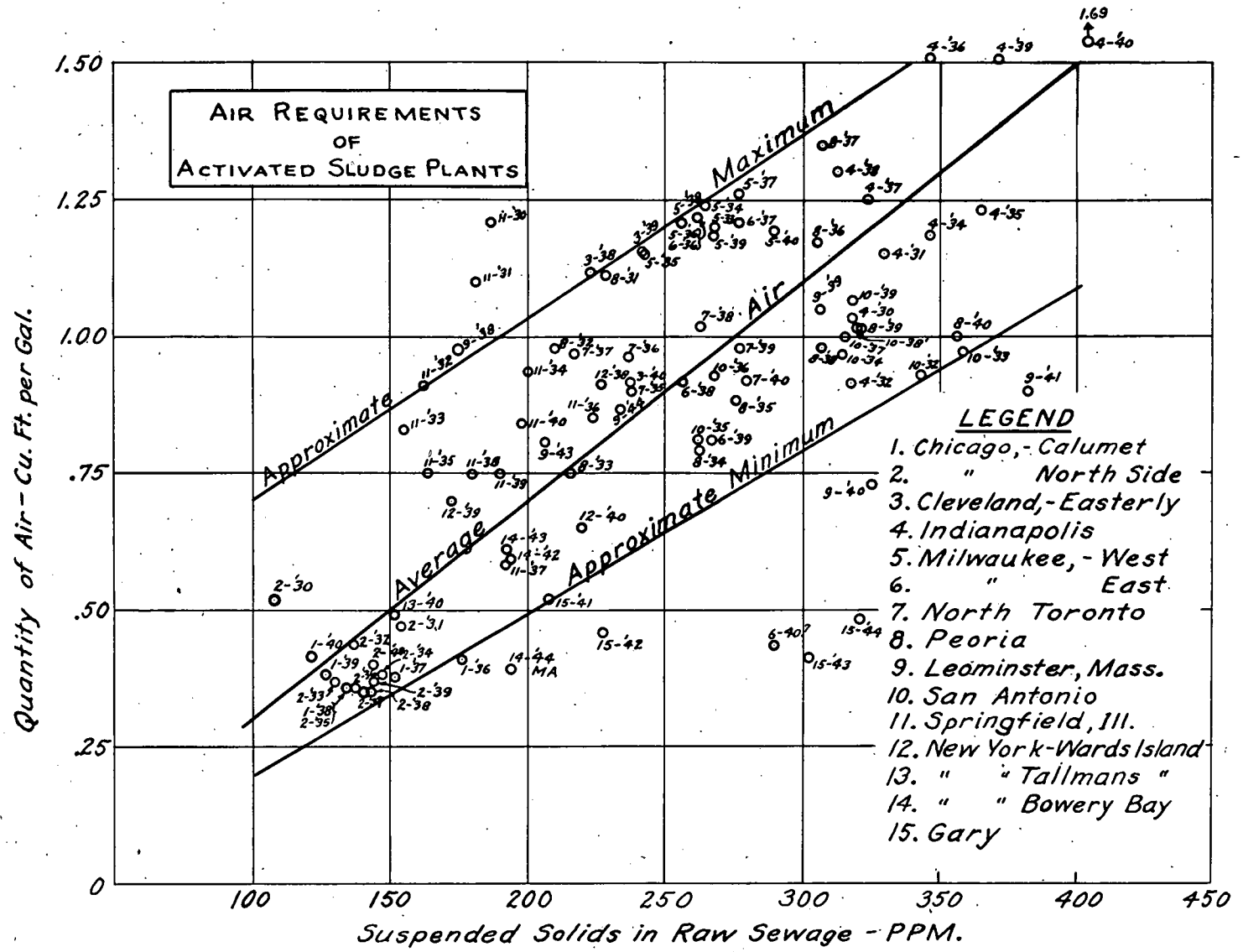
AIR REQUIREMENTS
OF
ACTIVATED SLUDGE PLANTS
Approximate Maximum
Average Air
Approximate Minimum
LEGEND
1. Chicago,- Calumet
2. " North Side
3. Cleveland,- Easterly
4. Indianapolis
5. Milwaukee,- West
6. " East
7. North Toronto
8. Peoria
9. Leominster, Mass.
10. San Antonio
11. Springfield, Ill.
12. New York-Wards Island
13. " " Tallmans "
14. " " Bowery Bay
15. Gary
Quantity of Air - Cu. Ft. per Gal.
Suspended Solids in Raw Sewage - PPM.
0
.25
.50
.75
1.00
1.25
1.50
100
150
200
250
300
350
400
450
1.69

FIG. 3.

gas engines in sewage treatment plants, which at the present time amounts to this: Gas engines are very efficient and economical, but the fuel supply is often inadequate, and stand-by charges are high.

Types of Installations and Stand-by Power Provisions

The variety and complexity of modern sewage treatment plants, combined with the problem of a generally inadequate fuel supply and the need for assured continuity of service, has led to a large variety of gas engine installations. The various types of plants are listed in tabular form in Table I. It will be noted that they fall into two main classifications,—those that generate electricity, and those where all or part of the main power consuming equipment is gas engine driven.

Gas engine driven generators are popular in primary treatment and trickling filter plants because there is usually more than enough power to pump the sewage, and the extra power generated can be used to supply the auxiliary load of small motors. Often the power generated may be sufficient to carry the entire plant load, as at the Coney Island plant in New York City.

For activated sludge plants engine driven pumps and blowers are popular, first, because the available gas supply goes farther that way and it is too much to hope that if electricity were generated any would be left over for the plant auxiliaries, and second, because engine drive has certain advantages. The speed of engine driven pumps and blowers may be varied at will, and the speed of engine driven pumps may be float controlled, with the result that the pumping rate may be made to follow the sewage flow closely without starting and stopping equipment, and the air flow can be adjusted to the minimum required by the quantity and strength of the sewage with resultant economy. This feature may dictate the use of direct drive where otherwise frequent starting and stopping of large pumps may overtax generator capacity or require expensive starting equipment.

The flexibility afforded by electric motor drive, however, has led the installation of electric generators even in activated sludge plants, as witness the Bowery Bay and Jamaica plants of New York City. The electric drive costs no more in the case of large units than direct drive, and has the advantage, considering two large pumping units, that Generator No. 1 can supply power to either Pump No. 1 or Pump No. 2, as can Generator No. 2; which means in effect that

TABLE I

TYPES OF GAS ENGINE POWER PLANTS FOR SEWAGE TREATMENT WORKS

I. ELECTRIC POWER GENERATION BY GAS ENGINES - ALL EQUIPMENT ELECTRIC MOTOR DRIVEN

STAND-BY SERVICE

A. Alternate Engine Fuels

1. City Gas (ex.- Coney Island)
2. Gasoline (ex.- Rahway Valley Joint Meeting)
3. Bottled Gas
4. Diesel Oil (Convertible Engines)

B. Electric Utility Connection

1. Generators phased in with electric utility units with provisions for buying and selling power (ex.- Dist. of Columbia, Cedar Rapids, Ia.)
2. Provisions for connecting all units to engine generator bus or electric utility bus
3. Equipment divided permanently between engine generator bus and electric utility bus (ex.- Bowery Bay)

C. Diesel Engine Driven Generators (ex.- Greenville, S.C.)

II. MAIN POWER LOAD (PUMPS AND BLOWERS) GAS ENGINE DRIVEN WITH AUXILIARY EQUIPMENT ELECTRIC MOTOR DRIVEN

STAND-BY SERVICE FOR MAIN ENGINE DRIVEN UNITS

A. Alternate Engine Fuels

1. City Gas (ex.- Tallman's Island)
2. Gasoline (ex.- Aurora, Ill.)
3. Bottled Gas (ex.- Pueblo, Colo.)
4. Diesel Oil (Convertible Engines)

B. Electric Motor Driven Stand-by Units supplied by the Electric Utility (ex.- Gary, Ind.)

ELECTRIC SERVICE FOR AUXILIARY MOTORS

A. Electric Utility Connection (ex.- Tallman's Island)

B. Gas Engine Driven Generators

1. Alternate Engine Fuels
   a. City Gas
   b. Gasoline (ex.- Aurora, Ill.)
   c. Bottled Gas
   d. Diesel Oil (Convertible Engines)
2. Electric Utility Connection (ex.- Springfield, Ill.)

C. Diesel Engine Driven Generators

one additional pumping unit is available as assurance against a complete shutdown in the case of electric drive.

Stand-by power may consist in the case of electric generation of either an additional fuel supply for the engines, a connection to the electric utility, or a diesel-generator set. In New York City, most of the plants have a connection with the gas company. This is particularly advantageous in New York because the plant can be started on city gas, but the gas company is not allowed to make a demand charge. This may not be true in other places, however, such as Boston, and city gas is expensive, although not as expensive as purchased electricity, and at many treatment plant locations may not be available. In such cases gas engines can be supplied with gasoline carburetors and the engines operated on gasoline until the gas production becomes adequate, or engines convertible to either gas or diesel operation may be installed.

A diesel engine driven generator is an economical means of furnishing the extra power required, especially when combined with ample gas storage. Sufficient diesel engine capacity should be installed to operate the plant in case of failure of the gas supply.

Probably the commonest form of stand-by is a connection with the electric company. Where the company will permit, it is desirable to operate the sewage treatment plant generators in parallel with the electric company's generators on a give and take basis, the company supplying electricity to provide for peak demands and during emergency shutdowns, and the plant feeding electricity into the utility company system when engine capacity and gas production exceed plant requirements. This system works very well at Washington, D. C., and Cedar Rapids, Iowa.

However, the company may not agree to such an arrangement. In that case, if the gas supply is adequate, the electric connection may serve as a stand-by only, and the plant feeders and large motors be connected by double throw switches to either the plant generator bus or the utility bus. If the gas supply is inadequate, so that some power must be purchased most of the time, it may be preferable to divide the main plant equipment permanently between the plant generator bus and the utility company bus. This is the case at the Bowery Bay plant of New York City, where the rates for any other arrangements were prohibitive.

In case the pumps or blowers or both are gas engine driven, elec-

tricity must be supplied to the auxiliaries and small plant motors. This may be provided by auxiliary engine driven generators as at Aurora, Illinois, by generators belted to the main engines as at Springfield, Ill., or by an electric connection with the utility company. With this set-up, provision must be made for an auxiliary fuel supply for the pump and blower engines, such as a connection with the gas company, bottled gas, or gasoline carburetors, unless additional motor driven stand-by units are provided as at Gary, Ind. Stand-by electric power for the auxiliaries may be provided by means of duplicate feeders from the electric company, or duplicate engine driven generators with provisions for an auxiliary fuel supply other than sewage gas.

One fact that must not be overlooked by the designer is that when the plant first goes into operation there will be no gas. This means that stand-by facilities will have to be adequate for the entire plant load; and if electricity or gas is purchased from the utilities, a high peak demand will be established during the first few months of operation which may affect power charges for several years.

The Dual Fuel or Gas Diesel Engine

A recent engine development of extreme importance to sanitary engineers is the dual fuel engine, capable of burning either gas or fuel oil on the diesel cycle, and with diesel efficiencies. This development originated in Germany and England just prior to the war and resulted in the commercial introduction of engines in which the fuel gas and intake air were introduced into the cylinder at about atmospheric pressure, compressed to diesel pressures of about 450 p.s.i., and ignited by the injection of a small quantity of fuel oil. In this country the dual fuel engine was pioneered by Worthington, and is now being offered by three manufacturers with the strong likelihood that other engine builders will manufacture it in the near future. This is inevitable on account of the sales advantages possessed by such engines and the fact that the patent on the fundamental principle of operation, which was issued to Dr. Diesel on April 30, 1901, has long since expired. A curious side light on this new engine is that, following the failure of Diesel's original work to reach commercial application and the success of Otto cycle gas engines, the belief that pre-ignition would occur if a gas-air mixture were compressed to diesel levels proved a stumbling block which effectually

sealed this avenue of research until recently. How effective this was is illustrated by the fact that the only commercial gas diesel in the United States, prior to 1944, was the Nordberg engine which utilizes high pressure gas injection. It is convertible to either gas or fuel oil, but is not a dual fuel engine in the same sense as the new engines, which can shift from gas to oil or vice versa without shutting down or losing speed, and can also operate on any proportion of gas and oil under full automatic control of the governor. Details of design and control systems are of course patented. It is claimed that the automatic governing system of the new Worthington engines is responsive to the relative proportion of the fuels available as well as to speed and load requirements, so that they will use gas to the limit of the amount available, filling in with fuel oil as required. Both supercharged and unsupercharged engines have been developed and are available in a limited range of sizes. Fundamentally the engines are heavy duty diesel engines with the addition of gas and pilot oil injection equipment and new governing systems.

The importance of this development to the design of power plants for sewage treatment works can hardly be overestimated. In the first place it means that 20 to 25% more power will be available from the same amount of gas as shown by the published performance curves (Fig. 5). In the second place fuel deficiencies can be made up by burning five-cent fuel oil automatically as required in the same equipment. It should be possible with these engines to abandon with perfect safety all stand-by connections with gas and electric companies, for the engines will have available at all times a cheap and plentiful fuel supply. Additional power will be obtained practically for the fuel cost (equal to 4.2 mils per kwh.), while stand-by charges will consist of the fixed charges on the oil tanks and spare engine. Under these conditions, it may happen that medium and large size sewage treatment plants will in the future tend to standardize on electric generation of power produced by dual fuel engines without utility company connections except in those cases where excess electricity can be sold to the power company.

The Engine Heat Balance

When you buy power from the electric company, power is all you get; but when you make your own power with gas engines, you get from 6000 to 7500 B.t.u. per kwh. bonus in the form of usable

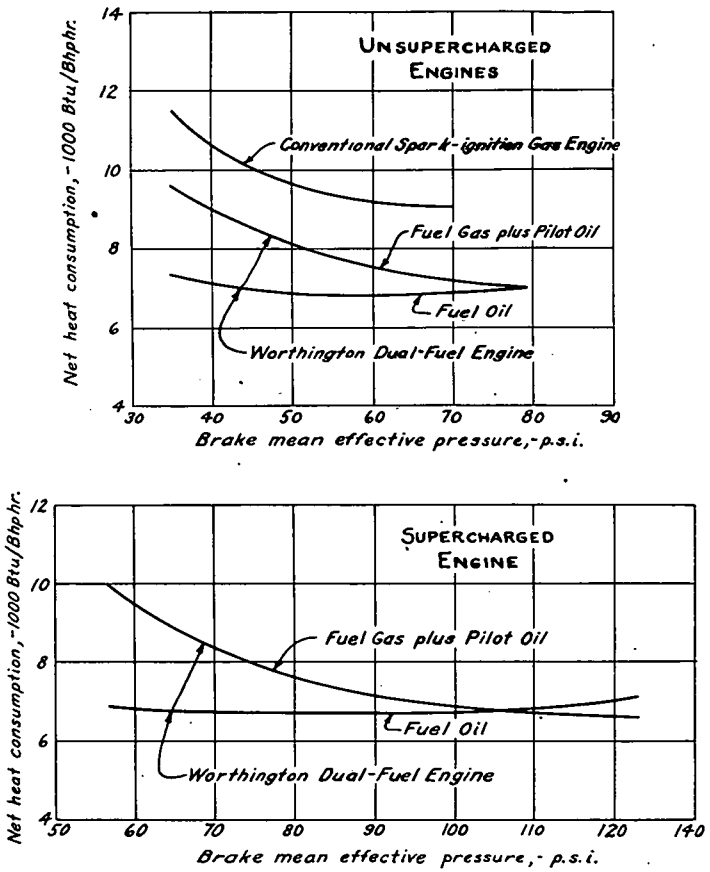

FIG. 5.

heat in the engine jacket water and in the exhaust. If you burn the gas under boilers you will have more gas than you can use during half the year and will have to waste the excess, and you won't get any power. The preferable arrangement would appear to be the installation of gas engines which have a steady year round demand for the gas, with the purchase of oil for building heating during the winter.

Fig. 6 shows typical performance curves for a 300 hp., 4 cycle, 400 r.p.m., heavy duty sewage gas engine. From these curves, Fig. 7 was computed by the author which shows graphically the proportion

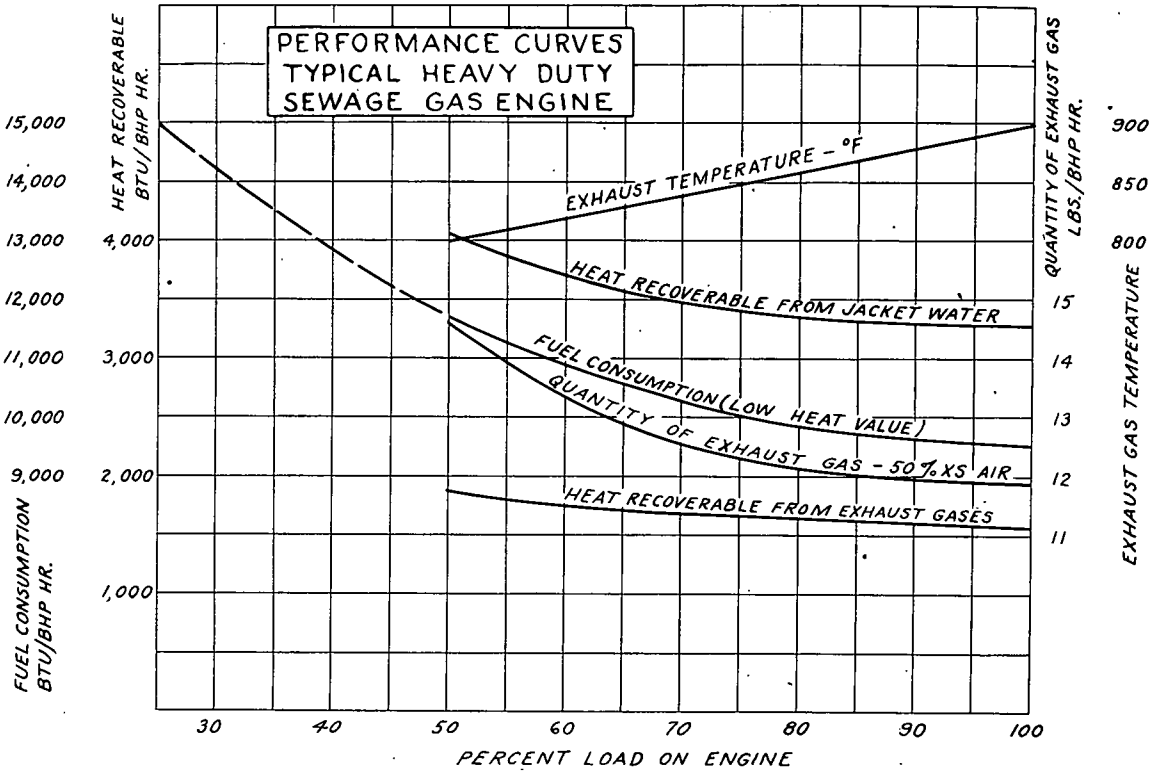
PERFORMANCE CURVES
TYPICAL HEAVY DUTY
SEWAGE GAS ENGINE
HEAT RECOVERABLE BTU/BHP HR.
FUEL CONSUMPTION BTU/BHP HR.
QUANTITY OF EXHAUST GAS LBS./BHP HR.
EXHAUST GAS TEMPERATURE
15,000
14,000
13,000
12,000
11,000
10,000
9,000
4,000
3,000
2,000
1,000
900
850
800
15
14
13
12
11
EXHAUST TEMPERATURE - °F
HEAT RECOVERABLE FROM JACKET WATER
FUEL CONSUMPTION (LOW HEAT VALUE)
QUANTITY OF EXHAUST GAS - 50% XS AIR
HEAT RECOVERABLE FROM EXHAUST GASES
30
40
50
60
70
80
90
100
PERCENT LOAD ON ENGINE

FIG. 6.

of usable power and heat from the same engine. The proportions will vary somewhat for different engines, and especially for different types of engines. Where exact figures are desired, data should be obtained from the engine builder. Table II shows the heat balance at ½, ¾,

TABLE II.—ENGINE HEAT BALANCE.

	½ Load	¾ Load	Full Load	As given by Walraven
	Percent of Heat Input			
Brake Horsepower	21.7	25.4	26.8	21-26
Jacket Water Heat	34.4	34.0	34.6	30-37
Exhaust Heat Recoverable	16.0	16.6	16.6	17-22
Useable Power and Heat	72.1	76.0	78.0	75-81
Exhaust Gas Loss	7.9	9.2	10.4	19-25
Friction and Radiation	20.0	14.8	11.6	
Total	100.0	100.0	100.0	100

and full load, obtained from Fig. 7, and also figures given by Walraven for Springfield, Ill., as reported by Keefer. These engines are equipped with oil coolers and water jacketed exhaust manifolds which increase the heat removed by the cooling water.

AVAILABILITY OF WASTE HEAT

The amount of water circulated through the engine jackets should be sufficient so that the temperature rise will not exceed 20 to 25° F. at full load. The temperature of the water leaving the engine should be limited to the value recommended by the manufacturer, which in general will not exceed 140° F. Since the optimum temperature of digestion tanks lies between 80° and 90° F. and the temperature of the water circulated through the heating coils should preferably not exceed 130° F. if incrustration of the outside of the coils is to be avoided, it will be seen that the heat in the jacket water can be fully utilized.

The temperature of the exhaust gases will be approximately as shown in Fig. 8, which shows also the exhaust gas temperatures of four stroke cycle mechanical injection diesel engines. It will be noted that the temperature of the exhaust from the gas engine, due to throttling control, is almost constant with varying load and consider-

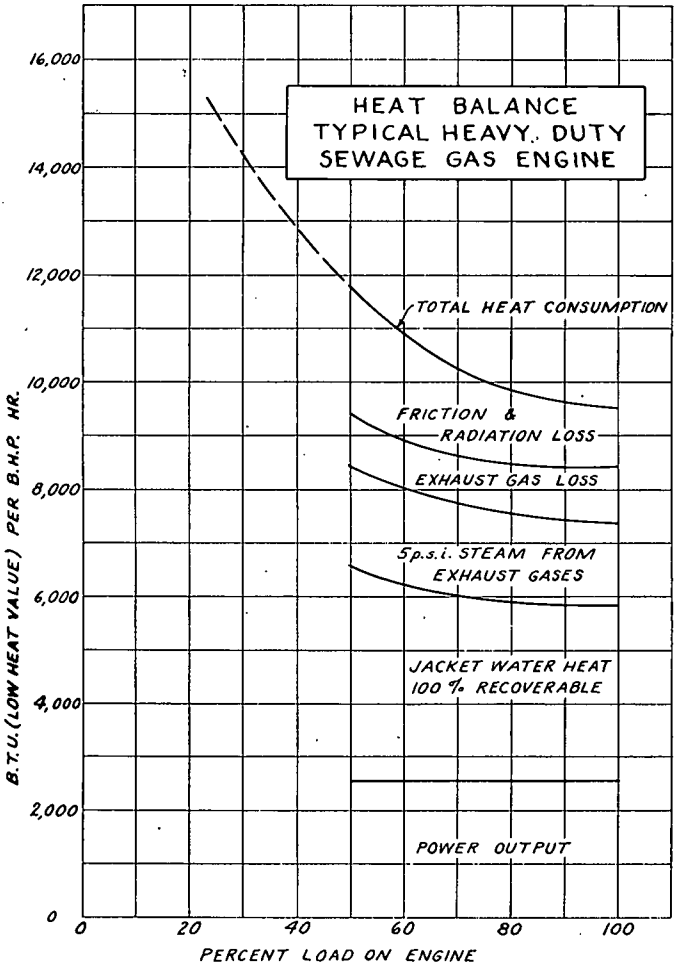

FIG. 7.

ably higher than the exhaust from the diesel engine. It is expected that the exhaust gas temperatures of dual fuel engines and diesel engines, will be similar, although no data are yet available.

PLANT HEATING LOAD

The heating load will consist of the heat required by the digesters, and the heat required for building heating and hot water. Al-

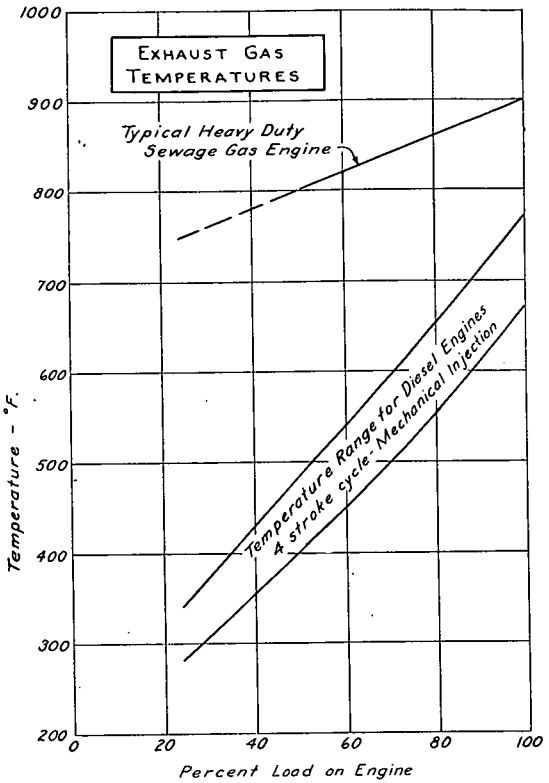

FIG. 8.

though digester heating requirements vary considerably depending upon size, concentration of sludge added, insulation, ground water level, etc., experience shows that where the bulk of the power requirements are furnished by gas engines, the heat in the jacket water and exhaust is ample to take care of digester requirements except in the coldest weather. During mild weather, the waste heat from the engines may be sufficient to provide for building heat requirements also. Computations for the proposed Rockaway Sewage Treatment Plant for New York City, designed by Metcalf & Eddy, indicate that the heat in the jacket water alone should be sufficient to keep the digesters at 90° F. The heat in the exhaust will be used to make low pressure steam and will supply about one-third of the heat requirements of the several buildings, seven in all. A study of the

records of the sewage treatment plant of the District of Columbia, also designed by Metcalf & Eddy, indicates that the digester heating requirements have been furnished almost entirely by the heat in the jacket water. At other plants where hot water systems are used for both building heating and digester heating, the available data indicate the need for supplementary heat, although the waste heat from the engines must furnish all the heat for the digesters plus a large proportion of the building heat. In some cases the supplementary heat is furnished by excess gas as at Aurora, Ill., and the District of Columbia.

Extremely cold weather, unless long sustained, will have little effect on digester temperatures on account of the tremendous heat capacity of the stored sludge. Computations for the Rockaway plant indicated that if no heat were applied during the coldest weather, the temperature drop on a real cold day (15° F.) would amount to only 1.5° F.

Methods of Heat Recovery

Heat recovery systems are of two general types, but the details are many and varied. Where hot water heating systems are used for the buildings, the common practice is to put the jacket water through an exhaust gas heat exchanger or hot water boiler, increasing the temperature approximately 10° F. to 150° F. The exhaust gas heat exchanger may be a combination boiler and silencer designed to operate with or without water, or the silencer and heat exchanger may be entirely separate, in which case provision is usually made to bypass the exhaust gases around the heat exchanger for summer operation. Fig. 9 shows in simplified diagrammatic form the heat recovery system used at Bowery Bay, which is of this general type. The cooling water system is a closed system, heat for the digesters and building heating systems being obtained from tubular heat exchangers. The high temperature building heating system heat exchanger is so connected that it can add heat to the jacket water system when more heat is required by the digesters, or it can serve as a feed water heater for the hot water boilers, thus furnishing heat for building heating. Additional cooling required during the summer months is furnished by a tubular cooler supplied with sewage effluent. This system has several variations,—in one the jacket water goes through the heating coils in the digesters, and in another, the digestion tank heating water

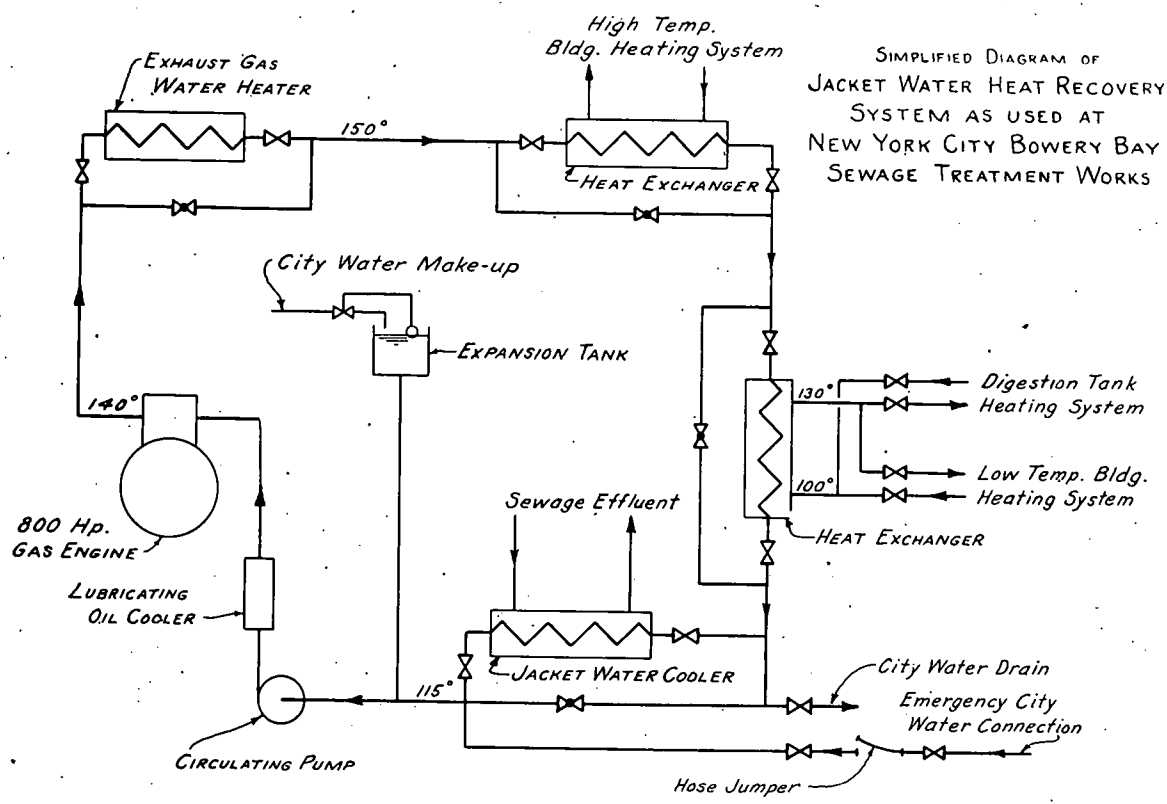
Simplified Diagram of
Jacket Water Heat Recovery
System as used at
New York City Bowery Bay
Sewage Treatment Works
Exhaust Gas
Water Heater
150°
High Temp.
Bldg. Heating System
Heat Exchanger
City Water Make-up
Expansion Tank
140°
800 Hp.
Gas Engine
Lubricating
Oil Cooler
Circulating Pump
Sewage Effluent
Jacket Water Cooler
115°
130°
100°
Digestion Tank
Heating System
Low Temp. Bldg.
Heating System
Heat Exchanger
City Water Drain
Emergency City
Water Connection
Hose Jumper

Fig. 9.

picks up heat from the jacket water in a heat exchanger and then flows through the exhaust gas hot water boiler.

Trouble has been experienced with this system at some plants due to corrosion of the exhaust gas heat exchangers. Due to the large percentage of moisture in the exhaust gases the dew point will be about 130° F. Condensation caused by cooling of the exhaust gases below the dew point has been the apparent cause of this corrosion. Sizing of the equipment for higher exhaust temperatures (300° F. recommended) combined with automatic controls now on the market should eliminate these troubles.

Where steam is used for building heating, the best arrangement is to install waste heat boilers to produce low pressure steam from the exhaust. Several varieties of waste heat boilers are on the market as well as combination boilers and silencers. The latter operate with a varying water level controlled by the steam pressure, and if economically sized, should give no trouble due to condensation. The waste heat boilers for the proposed Rockaway plant are of this type, and will operate with varying water level under automatic control during spring, summer and fall, and under full load with maximum water level during the winter, when they will operate in parallel with the plant heating boilers to which the automatic control will be shifted. Specifications call for the boilers to produce 1.5 lbs. of steam per brake horsepower hour under full load, and it is expected that with this size boiler exhaust gas temperatures will not drop below 300° F. even at ½ load.

Fig. 10 is a simplified schematic diagram of the engine jacket water and digestion tank heating system as designed for Rockaway. Evaporative coolers will be used for auxiliary cooling instead of tubular exchangers cooled by plant effluent. The evaporative coolers will have sufficient capacity operating wet to run two engines at full load, even if the digesters can take no heat, or the heating system is out of order. When operated dry the evaporative coolers will have about one-third* of their rated wet capacity, and during the spring and fall will discharge warm air to the engine room to help heat the power house at times of maximum load on the engines. Control of the temperatures in the system is fully automatic, the only manual operation being putting in or taking out of service additional equipment,

*Using recirculated air. With outside air the capacity would vary from 40 to 60% depending on the outside air temperature.

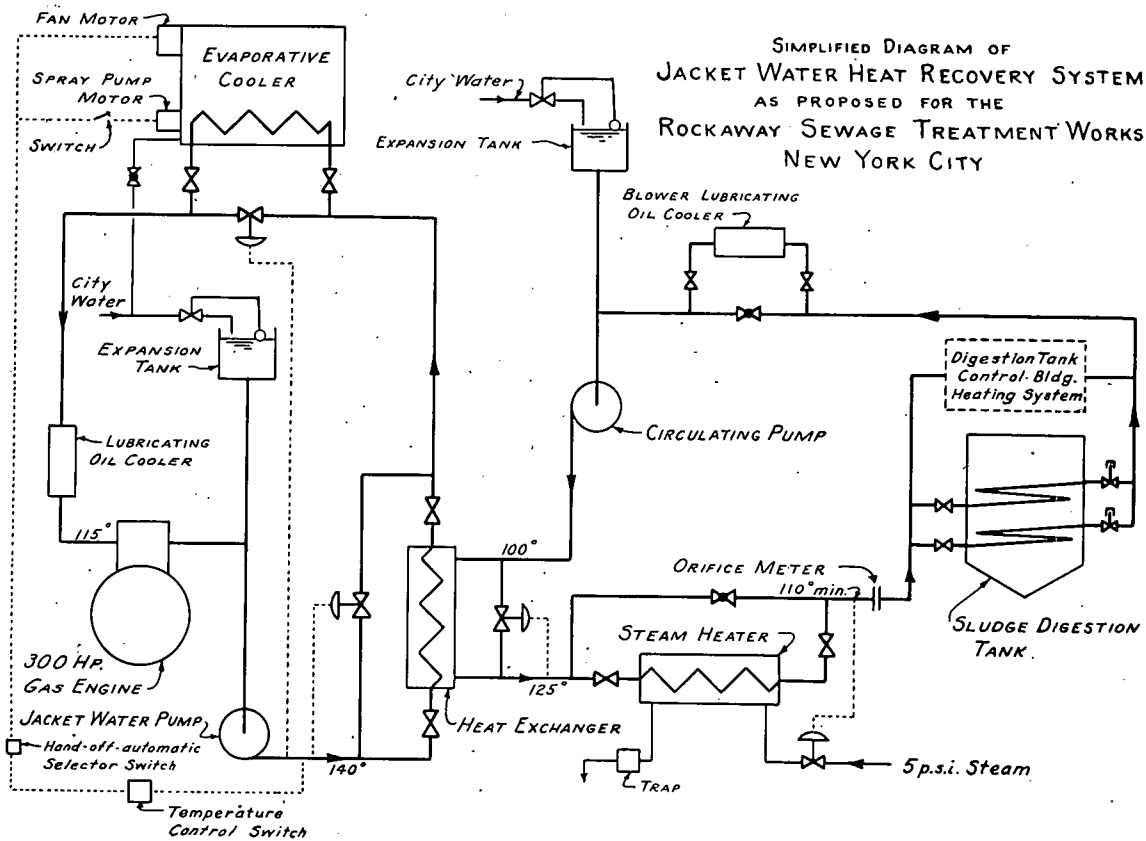
SIMPLIFIED DIAGRAM OF
JACKET WATER HEAT RECOVERY SYSTEM
AS PROPOSED FOR THE
ROCKAWAY SEWAGE TREATMENT WORKS
NEW YORK CITY
FAN MOTOR
EVAPORATIVE COOLER
SPRAY PUMP MOTOR
SWITCH
City Water
EXPANSION TANK
BLOWER LUBRICATING OIL COOLER
City Water
EXPANSION TANK
LUBRICATING OIL COOLER
115°
CIRCULATING PUMP
Digestion Tank Control Bldg. Heating System
100°
ORIFICE METER
110° min.
STEAM HEATER
SLUDGE DIGESTION TANK
300 HP. GAS ENGINE
125°
JACKET WATER PUMP
HEAT EXCHANGER
Hand-off-automatic Selector Switch
140°
TRAP
5 p.s.i. Steam
Temperature Control Switch

FIG. 10.

or throwing the switch to select wet or dry operation of the evaporative coolers. An indirect steam heater is provided for furnishing additional heat to the digestion tank circulating water when necessary.

Miscellaneous

It has been impossible to deal with many details of the design of gas engine installations in such a short paper, and I have selected what seemed to me to be of most interest to Sanitary Engineers. It seems appropriate, however, to add a word about automatic alarms and shut-down devices for the engines. These are available in a number of good designs, and their use in engine installations has become common practice. The duties of sewage plant operators are so numerous and the mechanical equipment they have under their charge is of such variety, that it is impossible to give minute attention to every piece of equipment. When things go wrong with an engine, the condition must be corrected promptly; yet it may be inconvenient to shut down the engine, especially if it is driving a generator which at the moment is furnishing all the plant electricity. For this reason we advocate the installation of alarms to warn the operator in case of high oil or water temperature or low oil or water pressure, as often the condition can be corrected without shutting down the engine. Devices should always be included for shutting down an engine automatically in case of overspeed, and they may be desirable also in case of low oil pressure.

Details of engine design and of power plant layout including the design of engine foundations, intake and exhaust piping, and fuel and lubricating oil systems have had to be omitted for lack of space. They will be found adequately covered in various books on engines.

Acknowledgments

The author desires to thank the Worthington Pump & Machinery Corporation for their permission to use Fig. 5 and for the data on which Fig. 6 and Fig. 7 were based. He also desires to thank the engineers of the Bureau of Sewage Disposal Design of New York City, particularly Mr. Wellington Donaldson, Chief, and Mr. John W. Van Denburg, Mechanical Engineer, for many courtesies extended the author, and particularly for permission to use Fig. 9 and Fig. 10.

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Of General Interest

Logan Airport Inspection Trip East Boston

[November 17, 1945]

On Saturday morning November 17, 1945 an inspection trip was conducted to the General Edward Lawrence Logan Airport at East Boston, through arrangements made with the Massachusetts Department of Public Works, which is carrying out the enlargement of this airport.

The group assembled in the Massachusetts National Guard Hangar building through the courtesy of Brig. Gen. Keville, Adj. General of Mass., where a number of speakers briefly explained the work of this project. President Carroll A. Farwell, of the Boston Society of Civil Engineers opened the meeting and then turned it over to Everett N. Hutchins, District Waterways Engineer of the Department of Public Works, who introduced the speakers after describing the project with the aid of maps and plans.

Commissioner Herman A. MacDonald outlined the history of the development and portrayed the vast expansion now under way and gave figures of the future activities at this airport.

Mr. William L. Shannon, consultant on runway construction and design, in speaking for himself and Prof. Arthur Casagrande, soils consultant, defined the relationship between the hydraulic fill—a heavy clay—which has been pumped into the area, and the granular fill and pavement that will be placed for the runways.

Mr. Julius Waldman, resident engineer of the Department, explained the nature of its runway cross-section and some of the field tests that have been conducted to ascertain the bearing value of the clay and the determination of the thickness of gravel and sand to be placed over the clay to provide a runway design for the heaviest airplanes.

Mr. Miles N. Clair, of the Thompson and Lichtner Co., consultant on the proposed permanent terminal building for this airport described the general layout of the building to provide adequate plane stations for the air traffic expected for a long period and emphasized the features of office spaces, and all service areas and facilities to handle passengers, baggage and motor vehicle access to the loading platforms.

Mr. Charles G. Loring, architect on the design for the temporary terminal at the airport described the building greatly needed now and at present under construction by the Marson Construction Co., to care for the rapidly growing airline business, including airline offices, ticket counter, 240 feet long and other facilities for the immediate future until the permanent building is erected.

Following these talks, the group, numbering about 85 persons, made an auto tour of the airport, driving out to Governor's Island where, from the top of the hill, an excellent view was obtained of the expanse of area reclaimed by the hydraulic fill, totalling approximately

30,000,000 cubic yards, to date, placed under contract with the Gahagan Construction Corporation of New York, four dredges being used, discharging at the rate of about 2,400,000 cubic yards per month.

Other contract work was visible, namely, the excavation of the clay for subgrade and the placing of gravel fill three and a half feet in depth over the clay, for certain runways, under a contract with B. Perini & Sons, Inc.

After lunch a small group was taken by launch to the Dredge "Nebraska" through the courtesy of Mr. Louis Newburg, Vice President and Treasurer of the Gahagan Construction Corp. The operation of the dredge and its equipment was explained by Mr. Vincent Hussin, Assistant Superintendent.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

OCTOBER 17, 1945.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the 20th Century Association, 3 Joy Street, Boston, Mass., and was called to order by President, Carroll A. Farwell. This was a Joint Meeting with the Designers Section, BSCE. Fifty-nine members and guests attended the meeting and dinner. Guests at the dinner included the speaker, Capt. Mack W. Angas, of New York, Capt. H. C. Fischer, Public Works Officer, U. S. Navy Yard, Boston, Capt. H. C. Griswold, Public Works Officer, U. S. Navy Yard, Boston, and Mr. John Ayer, representing the consulting engineers on the Dry Dock Project.

The President announced a change in the date of the November meeting from November 21 to Tuesday, November 20, which will be Student Night and a Joint Meeting with the American Society of Civil Engineers, Northeastern Section and Student Chapters, to be held at Northeastern University. The speaker to be Mr. Charles Froesch, Chief Engineer, Eastern Airlines, Inc.

President Farwell called upon Mr. Frank L. Lincoln, Chairman, Designers Section, to carry on any necessary business for that section.

President Farwell introduced the speaker of the evening, Capt. Mack W. Angas, CEC, USN, Superintending Civil Engineer, Area II, who gave a most interesting talk on "The Development of the New York Navy Yard During World War II. The talk was illustrated by slides and motion pictures and by a unique model of considerable size of the large complicated steel forms, showing framing and bracing which were erected and handled by cranes for floor and side walls of the two dry docks constructed of concrete by the tremie method.

A rising vote of thanks was given the speaker.

Adjourned at 9:15 P.M.

EVERETT N. HUTCHINS, *Secretary*

NOVEMBER 20, 1945. — A regular meeting of the Boston Society of Civil Engineers was held this evening at Richards Hall, Northeastern University and was called to order by President Carroll A. Farwell, at 7:00 P.M. One hundred seventy members and guests were present and one hundred fifty-five persons attended the dinner.

This meeting was the Annual Student Night with representatives from

the Student Chapters of the American Society of Civil Engineers at Harvard University, Massachusetts Institute of Technology, Tufts, Brown University, Dartmouth, Rhode Island State, Worcester Polytechnic Institute and of Northeastern University which is a section of the BSCE. This meeting was also a joint meeting with the Northeastern Section, American Society of Civil Engineers.

The dinner was held in Commons Hall, in the New Engineering Building of Northeastern University. The Student Engineering Society at the University arranged the details of the dinner and assisted in the enjoyment of the facilities of the school which had been made available by the faculty members.

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

President Farwell announced that the December meeting of the Society would be held at the 20th Century Association, 3 Joy Street, Boston. This to be a joint meeting with the Sanitary Section BSCE. Dr. Vlado A. Getting, Commissioner of Public Health of Massachusetts, will be the speaker.

Mr. Chester J. Ginder, acting Secretary, announced the following had been elected to membership:

Grade of Member: Louis H. Berger, William G. Stephenson*

President Farwell called upon Mr. William Bassett, President of ASCE to conduct any business matters required by that Society.

President Farwell called upon Prof. Albert Haertlein, President of ESNE who explained the activities and aims of the ESNE.

*Transfer from Grade of Junior.

President Farwell then introduced the speaker of the evening, Mr. Charles Froesch, Chief Engineer, Eastern Airlines, Inc., who gave an extremely interesting talk on "Influence of Air Power on the Future of America". The talk was illustrated by slides and also moving pictures.

A rising vote of thanks was given the speaker.

Adjourned at 9.55 P.M.

CHESTER J. GINDER,
Secretary, pro tem

DECEMBER 19, 1945.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the 20th Century Association, 3 Joy Street, Boston, Mass., and was called to order by President, Carroll A. Farwell, at 7:00 P.M. This was a joint meeting with the Sanitary Section, BSCE. Twenty-eight members and guests were present and twenty-eight persons attended the dinner.

The President announced that the January meeting would be held at the 20th Century Association, 3 Joy Street, Boston. The speaker to be Mr. Bryant L. Hopkins, Hydraulic Engineer and Manager, Kennebec Water Power Company, Bangs Station, Waterville, Maine.

President Farwell called upon Mr. Murray H. Mellish, Chairman, Sanitary Section, to conduct any business matters necessary for that Section.

President Farwell introduced the speaker of the evening, Dr. Vlado A. Getting, Commissioner of Public Health of Massachusetts, who gave a very interesting talk on "Flouride in the Treatment of Water Supplies and Its Effect on Dental Health". The talk was illustrated with slides. Numerous questions of interest were presented by members.

A rising vote of thanks was given the speaker.

Adjourned at 8:15 P.M.

EVERETT N. HUTCHINS, *Secretary*

SANITARY SECTION

OCTOBER 3, 1945.—A meeting of the Sanitary Section was held at the Society Rooms at 7:10 P.M. on the above date following an informal dinner gathering at the Ambassador Restaurant. Twenty-nine persons attended the meeting.

The Chairman announced that a joint meeting with the Hydraulics Section would be held on November 7, partially to replace the customary June meeting of the Section, which was not held this year.

The speaker of the evening was Allen J. Burdoin, Senior Engineer, Metcalf & Eddy, who presented a very interesting, illustrated paper, entitled "Gas Engine Power for Sewage Treatment Plants."

After a short general discussion, the meeting adjourned about 9 P.M.

GEORGE C. HOUSER, *Clerk*

DESIGNERS' SECTION

OCTOBER 17, 1945.—A joint meeting of the Designers' Section with the Boston Society of Civil Engineers was held at the Twentieth Century Association at 6.00 P.M. President Carroll A. Farwell presided and introduced the speaker of the evening, Captain W. Mack Angas, CEC, USN, whose subject was "The Development of the New York Navy Yard During World War II."

Captain Angas described the additional ship-repair, ship-service and other public works facilities installed in the Yard, speaking in detail about the construction of the new battleship-size shipbuilding and repair docks. His description, slides and moving pictures covered the dry dock project completely, including the underwater pile driving, the installation of tremie forms for underwater concrete, the handling of concrete by pumpcrete methods and the placing of concrete under water. A large wooden model of the tremie trusses and forms used for both floor and wall pours was displayed and explained.

The paper was followed by a general discussion.

There were sixty members and guests present. The meeting closed at 9.00 P.M.

HENRY I. WYNER, *Clerk*

NOVEMBER 14, 1945.—Following an informal dinner at the Ambassador Restaurant the Designers' Section of the Boston Society of Civil Engineers met at the Society Rooms, 715 Tremont Temple, Boston. Chairman Frank L. Lincoln called the meeting to order at 6.50 P.M.

The Clerk's report of the previous meeting was read and accepted.

Mr. Lincoln introduced Joseph S. Newell, Professor of Aeronautical Structural Engineering, Massachusetts Institute of Technology, the speaker of the evening, whose subject was "Some Problems in the Analysis of Aircraft Structures."

Professor Newell described the design technique currently used in the airplane industry for wing and fuselage structures utilizing thin structural elements subject to local buckling. Application of the procedure was illustrated by a typical design problem.

The talk was followed by an open discussion period. The meeting was attended by 19 members and guests who indicated great interest in the application of airplane design technique to the design of bridge and building structures.

The meeting adjourned at 8:30 P.M.

HENRY I. WYNER, *Clerk*

DECEMBER 12, 1945.—A meeting of the Designers' Section of the Boston Society of Civil Engineers was held at the Society Rooms, 715 Tremont Temple, Boston, following an informal dinner at the Ambassador Restaurant. Chairman Frank L. Lincoln presided and called the meeting to order at 6:55 P.M.

The Clerk's report of the previous meeting was read and accepted.

Mr. Lincoln introduced the speaker of the evening, Mr. Oscar S. Bray, Structural Engineer with Jackson & Moreland, Engineers, who presented a paper entitled: "Design and Construction of Mystic River Tunnel."

Mr. Bray described the design analysis and construction procedure utilized for the electric service tunnel driven under the Mystic River for the Boston Edison Company. Requirements for the crossing, various solutions studied and the final conclusions which determined the type of tunnel selected were discussed in detail.

The reading of the paper was followed by an open discussion period.

The meeting, which was attended by 54 members and guests, was adjourned at 8:05 P.M.

HENRY I. WYNER, *Clerk*

HYDRAULICS SECTION

NOVEMBER 7, 1945.—A joint meeting of the Hydraulic Section and the Sanitary Section was held this evening at the Society rooms at 7 p.m., following an informal dinner at the Ambassador Restaurant. 35 persons attended the meeting.

In the absence of Chairman McDonald, Harold A. Thomas, Jr. introduced the speaker of the evening, Mr. R. M. Leggette, Consulting Ground Water Geologist, from New York City. He gave a very interesting and informative talk on "Artificial Re-charge of Ground Water".

After a considerable discussion the meeting adjourned at 9.15 p.m.

J. G. W. THOMAS, *Clerk*

ADDITIONS

Members

LOUIS H. BERGER, 37 Williams Street, Roxbury 19, Mass.

VINCENT R. CAPUTO, 26 Eliot Street, Medford 55, Mass.

SANFORD S. MITCHELL, 180 Commonwealth Avenue, Boston, Mass.

EDWARD F. WILCOX, R.F.D. #3, Torringford Avenue, Torrington, Conn.

DANIEL W. MILES, 7259 Constance Avenue, Chicago, Illinois.

Juniors

JOSEPH J. BULBA, 368 Franklin Avenue, Hartford, Conn.

ROBERT N. KUEHN, 436 Mamouth Road, Manchester, N. H.

ROBERT J. MARKELL, 595 Fulton Avenue, Hempstead, Long Island, N. Y.

EDWARD MURPHY, 647 E Third Street, South Boston, Mass.

SAMUEL J. PATTISON, JR., 122 East Side Parkway, Newton 58, Mass.

FRANCIS SATTIN, 119 Harvard Street, Malden 48, Mass.

DEATHS

PROF. J. DONALD MITSCH Jan. 18, 1946

APPLICATIONS FOR MEMBERSHIP

[January 16, 1946]

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 applicants until the expiration of fifteen (15) days from the date given.

For Admission

HOWARD B. ALLEN, West Roxbury, Mass. (b. May 2, 1910, Sedgwick, Maine). Graduated from Northeastern University in 1930, B.C.E. Experience; City of Newton Engineering Department, on the cooperative plan while attending Northeastern University, working as rodman and transitman in a field party, June to December, 1930, Chief of Survey Party for City of Newton; April-August, 1931, State of Maine Highway Commission, as construction inspector, assistant resident engineer and survey party chief on highway construction. August, 1931-January, 1942, Commonwealth of Massachusetts Department of Public Works, Highway Division as Senior Engineering Aide. Drafting room work on preliminary plans, sections, profiles. (1931-1934) Project Department on location, design, estimating of quantities and preparation of estimates of cost of major projects. (1934-42) January to August, 1942, Shreve, Lamb & Harmon-Fay, Spofford & Thorndike, Junior designer on Newfoundland and Arctic bases. August-October, 1942, Metcalf & Eddy, structural details and miscellaneous design and layout for sewage and water works. October, 1942-May, 1943, Jackson & Moreland, structural detail and layout. May, 1943, to present with Metcalf & Eddy on design and layout work for sewage treatment, water works and refuse disposal plants. Refers to *T. C. Coleman, A. J. Burdoin, F. L. Flood, F. A. Marston, A. L. Shaw.*

JOHN F. FLAHERTY, South Boston, Mass. (b. October 10, 1907, Boston, Mass.) Graduated from Mechanics Arts High School in 1925; attended Lincoln Institute 1931-1933; Graduated Lowell Institute at M.I.T., in 1939. Employed by City of Boston Transit Commission as rodman in field party on construction of Sumner Traffic Tunnel 1932-1932; Boston Public Works Department as Junior Engineering Aide 1932-1935; as rodman-transitman, 1935-1937; as senior engineering aide 1937-1941, in field party on surveys for and construction of sewerage works and performing related office work such as calculations, drafting, etc; as Junior Civil Engineer 1941-1944, Chief of party on surveys for, and resident engineer on construction of sewerage works and performing the related office work, estimates, computations, drafting, etc; Represented the City of Boston in establishing the ratios for Metropolitan sewerage assessments for 1942-1943 and determining the amount for the pumping contract between the State and City for the years 1942-1946; as Assistant Engineer 1944 to date, making studies of sewage pumping and flow records, compiling data for post-war construction program, and assisting in preparing a preliminary report on a sewerage treatment plant for the Main Drainage District. At present Acting Engineer of Design in charge of the pumping stations and disposal works of the Main Drainage system and draughting room of the Sewer Division. Registered Professional Engineer, Massachusetts Certificate No. 511. Refers to *G. Haskell H. Schiano, W. T. Morrissey, E. Wright.*

CLIFFORD E. MOFFET, Waltham, Mass. (b. October 4, 1917, Los Angeles, California). Pasadena Junior College in Pasadena, California 1935-1937; June, 1937—September, 1938, worked for two general contractors on general construction work. Entered Mass. Institute of Technology, September, 1938, received S. B. degree in Civil Engineering in June, 1941. 1939-1941 was employed part time in the C. E. Department of M.I.T. During the summer of

1940 was employed by the Inland Steel Company at their Wheelwright, Kentucky, mines, and during the winter of 1940-1941, was employed part time by Chas. T. Main, Inc., of Boston, on design at Camp Edwards; July, 1941-June, 1942, was with Standard Oil Company of California as Junior Engineer; June, 1942-February, 1943, with Leeds, Hill, Barnard and Jewett, Consulting Engineers in Los Angeles, as engineer on army camp and hospital design; Feb. 1943, entered the U. S. Navy as Ensign Civil Engineer Corps, spent two years overseas at Tarawa, Pearl Harbor and Tinian, released from active duty as Lt. CEC, USNR in December, 1945. For sixteen months I was a staff officer for Sixth Naval Construction Brigade in charge of designing and construction of camps and hospitals. At present employed as structural engineer by Chas. T. Main, Inc., Boston. Refers to *J. B. Babcock, D. W. Taylor, J. B. Wilbur.*

CHARLES A. PERKINS, Harvard, Mass. (b. March 15, 1913, Westminster, Mass.) Graduated from Northeastern University June 1936, with B. S. degree in Civil Engineering. Experience, July, 1936, with Parker, Bateman & Chase, Civil Engineering, Clinton, Mass., as transitman and stayed there until May, 1937; associated with the New England Structural Steel Company, Everett, Mass., until March 1939. At this time I took a job as transitman with the Metropolitan District Water Supply Commission, giving line and grade with a field party on the Quabbin Dam until it was completed in August 1940. Then took a position as a civil engineering draftsman in the Navy Design Section at Quonset Point, R. I. Duties were to assist in making plans and calculations for roads, walks, runways, bulkheads, drainage and other items necessary for a new air base. During this time I took a refresher course in Structural Design at Rhode Island State College. In January, 1942, transferred from the Design Section to Civil Service in the Public Works Department of the Naval Air Station. As Associate Civil Engineer my duties were to supervise the drawing of plans, to prepare specifications and cost estimates for all work coming under the jurisdiction of the Public Works Department. In November, 1945, I relinquished my position at Quonset to purchase the firm of Parkman, Bateman & Chase, Civil Engineers, Clinton, Mass., which I now operate under the same name. Refers to *C. O. Baird, F. W. Bateman, E. D. Mortenson, C. A. Richardson.*

JOHN J. SCHEUREN, JR., Swampscott, Mass. (b. August 2, 1919, Boston, Mass.) Graduated from Massachusetts Institute of Technology in 1931, S. B. degree. Worked while on leave of absence in 1930 for the Gow Company as rodman and for J. H. McNamara, Inc., as concrete plant engineer. 1931, 1932, B. Perini & Sons, as efficiency engineer on concrete production (transit mix) as assistant superintendent, charge concrete plans, paving, etc., on Boston-Worcester Turnpike; 1933, engineer at Quabbin Reservoir Enfield Dike for Perini, later assistant superintendent, in charge of access roads, Grand Central Parkway, Jamaica, Long Island; 1933-1934, superintendent Highway Construction at Haverhill, N. H.; 1934-1935 superintendent on Road and Bridge Construction on Concord Turnpike; 1935, engineer and night superintendent on Barkhampsted Dam in Connecticut; 1936, superintendent construction, canal (Cape Cod) revetment and roads; 1937, superintendent construction Barbers Crossing Grade Elimination at Worcester, Mass.; 1938, engineer, New Jersey Approaches—Lincoln Tunnel; 1938-1939, superintendent construction, Pittsburg Reservoir Dam, N. H., Water Resources Board; 1939-

1940, engineer on Tuscarora Tunnel, Penn., Turnpike; 1941-1942, General Superintendent L. S. Foreman Company, Kansas City, Mo., 26" pipe lines, Kansas to Chicago; 1942, superintendent Watter Brothers Roads Company, Syracuse Air Base Paving; 1942, superintendent West Shore Construction Company, Suffern, N. Y., general superintendent, Roads, Railways and Utilities at Seneca Ordnance Depot, Geneva, N. Y.; 1943, general superintendent for L. S. Foreman, Kansas City, on construction of "Big Inch" pipeline; 1943-1945, Corps of Engineers, A.U.S., construction work, Africa and Italy. Refers to *F. Dittami, J. L. Doherty, R. S. Holmgren, W. T. Morrissey, W. F. Uhl.*

Transfer from Grade of Junior

CHARLES M. KELLEY, Dorchester, Mass. (b. April 8, 1912, Boston, Mass.) Graduated from Northeastern University in 1935, with Honor. B. S. degree in Civil Engineering. Have taken review courses in Hydraulics and Concrete Design at Brown University, Providence, R. I., in 1942, 1943. Also took special course in Prospecting and Mining given by the University of Alaska at Anchorage, Alaska in 1944. Presently engaged in the study of law at Boston College Law School, third year of the regular course leading to LL.B. degree. Experience, 1935-1936, Massachusetts Department Public Works, surveyor in the Highway Division. Subsequently employed in the Waterways Division, as draftsman; 1936, Merritt, Chapman, Scott Corp., timekeeper on reconstruction of platforms at Commonwealth Pier, So. Boston, Mass.; 1936-1937, Revere School Department, instructor in mathematics in the Evening High School; 1936-1937, U. S. Engineer Office, Boston, Mass., Engineer draftsman; 1938-1939, U. S. Engineer Office, Memphis, Tenn., Civil Engr. draftsman; 1939-1940, U. S. Engineer Office, Concord, N. H., Junior Civil Engineer on real estate surveys and appraisals for the Franklin Falls Dam and the Blackwater Dam; and on the subsequent construction of the Blackwater Dam as an inspector of general construction; 1940-1941, U. S. Engineer Office, Port-of-Spain, Trinidad, British West Indies, Junior Engineer on original surveys for U. S. Army Air Base at Cumuto; 1941-1942, U. S. Engineer Office, Antigua, British West Indies, Junior, Associate Engineer, Chief of control surveys, then Office Engineer and finally inspector in charge of construction of concrete structures and of Air Corps buildings and hangars; 1942-1944, U. S. Engineer Office, Providence, R. I., Assoc. Engr. Assistant to Chief of Operations, supervising administration of certain military construction contracts, including work of resident engineer, negotiating modifications to contracts, expediting procurement of materials, reviewing procurement of materials, reviewing payment estimates against completed work, etc. In the field, directed the layout and subsequent changes in construction; 1944-1945, Headquarters Alaskan Department, Office of the Engineer, Anchorage, Alaska, Associate Engineer, project engineer for one of the secret Aleutian posts reviewing requests for construction, suggesting changes in some or recommending approval thereof to the Comm. General. Was also head of the Real Property sub-section. 1945 to present, New England Division Engineer Office, Boston, Associate Engineer in Real Estate Division performing necessary field work in connection with the acquisition of real estate interests for both military and civil projects. Refers to *C. O. Baird, E. A. Gramstorff, H. I. Wyner, W. W. Zapolski.*

JOHN H. MANNING, Newton, Mass. (b. February 27, 1916, Boston, Mass.) Graduated from Northeastern Univer-

sity in 1939, with B. S. degree in Civil Engineering. As co-operative work student, employed by City of Newton Engineering Department from 1936 to 1939, as rodman, and transitman in survey party on municipal engineering work. June, 1939, to July, 1941, as transitman in survey party with City of Newton Engineering Department on municipal engineering work; July, 1941, to December, 1943, with U. S. Engineer Department, as Chief Draftsman and Office Engineer and Assistant Area Engineer at U. S. Engineer Sub-Office, Bangor, Maine, on airport and housing construction; February, 1944, to date with Civil Engineer Corps of U. S. Navy, as Lieutenant (junior grade) in the 28th U. S. Naval Construction Battalion, engaged in construction work in the European and Pacific Theatres. Refers to *A. Q. Robinson, A. E. Everett, C. O. Baird, E. A. Gramstorff.*

ARTHUR J. YARDLEY, Edgewood, Rhode Island. (b. April 17, 1914, Warwick, Rhode Island). Graduated from Rhode Island State College, 1936, with B.S. degree in Civil Engineering. Experiences, July, 1936, to December, 1938, worked for Taco Heaters, Inc., Providence, R. I., Assembly Man and Machine Operator. Mechanical experiences in construction of hot water heaters. In charge of ordering materials, machining and assembling eight different hot water heaters. Used lathe, radial frill, gana drill and grinder; December, 1938 to July, 1940, Student Engineer, Hydraulic & Report Section, worked on rainfall studies, flood frequencies, unit graph studies, flood routing, making area capacity curves for reservoir sites, spillway design flood gate operation of flood control reservoirs, flood control economic studies; July 1940, to January, 1941, Inspector of General Construction, inspected placing and compacting of earth rock fill in dam embrankment, peizometer tubes, settlement gages, sand and gravel filters, drilling and blasting of ledge, construction of forms for concrete and collected cost data on plant and labor used by contractor; January, 1941, to August, 1941, Inspector and Senior Inspector, inspected excavation and backfill of building foundation, also inspected forms, steel reinforcing and placing of concrete. From March, 1941, until August, 1941, was Senior Inspector of all sewer, water and electrical construction; August, 1941 to October, 1941, Chief Inspector of construction of buildings, utilities, roads, hangers, taxiways and gasoline distribution systems at Grenier Field Air Base, Manchester, N. H.; October, 1941 to January, 1944, Area Engineer, in charge of all military operations at Grenier Field Air Base, Manchester, N. H., project involving an expenditure of approximately $8,000,000 and providing housing and operational facilities for 5,000 troops. January, 1944 to October, 1945, on duty with U. S. Navy as Ensign. Four months at Navy Schools, sixteen months as Executive Officer on LC1 (L) 607, June, 1945, promoted to Lieutenant (J.G.); November, 1945, to present date, Engineer, (Civil) with U. S. Engineer Office, Park Square Building, Boston, Mass. Refers to *I. Chase, Jr., E. F. Childs, H. B. Shumway, W. H. Fowler.*

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Vol. XXXIII JANUARY, 1946 No. 1

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