JOURNAL *of the*

BOSTON SOCIETY OF CIVIL ENGINEERS

1848 BOSTON SOCIETY OF CIVIL ENGINEERS

104 YEARS

1848-1952

APRIL - 1952

VOLUME 39 NUMBER 2

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1952-1953

PRESIDENT

EMIL A. GRAMSTORFF

VICE-PRESIDENTS

J. STUART CRANDALL
(Term expires March 1953)

CHESTER J. GINDER
(Term expires March 1954)

SECRETARY

ROBERT W. MOIR

TREASURER

CHARLES O. BAIRD, Jr.

DIRECTORS

JOHN G. W. THOMAS
LESLIE J. HOOPER
(Term expires March 1953)

JAMES F. BRITTAIN
EDWIN B. COBB
(Term expires March 1954)

PAST PRESIDENTS

HARRISON P. EDDY, Jr.
THOMAS R. CAMP
JOHN B. WILBUR

SANITARY SECTION

FOZI M. CAHALY, Chairman
ARIEL A. THOMAS, Clerk

STRUCTURAL SECTION

EDWARD C. KEANE, Chairman
RUTH D. TERZAGHI, Clerk

TRANSPORTATION SECTION

H. LOWELL CROCKER, Chairman
HERMAN G. PROTZE, Clerk

HYDRAULICS SECTION

BYRON O. McCOY, Chairman
ARTHUR T. IPPEN, Clerk

SURVEYING & MAPPING SECTION

GEORGE W. HANKINSON, Chairman
C. FREDERICK JOY, Jr., Clerk

Editor—CHARLES E. KNOX

715 Tremont Temple, Boston, Mass.

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 39 APRIL, 1952 Number 2

PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

ANNUAL REPORTS

Journal of Boston Society of Civil Engineers is indexed regularly by Engineering Index, Inc.

Entered as second-class matter, January 15, 1914, at the Post Office at Boston, Mass., under Act of August 24, 1912

Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

Subscription Price $5.00 a Year (4 Copies)
$1.25 a Copy

EMIL A. GRAMSTORFF

PRESIDENT

Boston Society of Civil Engineers

1952-53

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 39 APRIL, 1952 Number 2

THE RELATIONSHIP BETWEEN CIVIL ENGINEERING AND ARCHITECTURE

PRESIDENTIAL ADDRESS BY JOHN B. WILBUR

(Boston Society of Civil Engineers, March 19, 1952.)

BY custom it is my duty, as retiring president, to deliver an address on this occasion; and by courtesy it is your obligation to listen to it. The advantage, I am afraid, lies on my side, for I—at least—can choose a subject in which I have current interest; I can only hope that it will be of interest to you as well. In any event, I am going to discuss the relationship between Civil Engineering and Architecture.

These two professions have much in common, since both, in the last analysis, are concerned with building to improve human environment—with construction that betters the conditions under which people live, work, and play. But building changes human environment in two quite different ways, depending on the degree to which direct and intensive human use is involved, and to which human measure thereby becomes the controlling parameter: it may be that the physical thing is of paramount importance—that safety, functional efficiency, and economy are the controlling factors; by contrast, the well-being of people in an environment may be the dominant characteristic—their intellectual and emotional satisfactions, as well as their physiological comforts—in short, their direct human responses.

These two manners of affecting human environment are not independent variables; neither can exist by itself, and either affects the other. To change physical environment in any way, inevitably affects direct human responses to some degree; and construction dedicated to changing the satisfactions involved in direct human responses, just as inevitably has some effect on physical environment. It is never a

matter of determining which exists to the exclusion of the other, but rather, of recognizing the existence of both—and of considering which predominates in a given situation.

For example, the primary effect of a highway bridge on human environment is physical; one who drives across it is interested mainly in its safety from both the structural and traffic viewpoints, and in its functional efficiency as an avenue of transportation. He is also interested in his emotional response to the structure—as are those who view it from up or down stream, or who live in its vicinity; these people will either accept or reject the bridge as a fitting part of its environment, but this—however important it may be—is secondary to physical considerations.

The primary effect of a house on human environment is, however, in the area of satisfying direct human responses. The occupant is interested, it is true, in structural and mechanical efficiency—for these contribute to comfort and convenience; yet in his prolonged and intimate association with the house, his emotional responses are tempered as much by the fitness of these physical features as they pertain to him as an individual, as by the physical features themselves. The home is more than a physical shelter; it shelters him psychologically as well; materials, colors, proportions, degrees of enclosure, and the like—all awake emotional responses that in a large measure determine whether or not the house is enjoyed by the occupant.

Perhaps, on the basis of the foregoing discussion, it is possible to redefine both Civil Engineering and Architecture in a manner that differentiates between the two professions: one might describe Civil Engineering as the science and art of building for the primary purpose of improving physical environment; on a comparable basis, Architecture would be defined as the art and science of building for the primary purpose of satisfying direct human responses.

With these definitions in mind, certain types of structures can be seen to lie rather clearly in the province of one profession or another: a power plant building, for example, is built primarily to provide suitable physical environment for the equipment it houses, and would come under Civil Engineering; an office building, on the other hand, is built primarily to satisfy the direct human responses of the people who work in it, and would come under Architecture. There is, however, an area that includes structures in which neither physical environment nor the satisfaction of direct human responses is the controlling

consideration; an assembly plant for automobiles might come under this category; there is room for honest differences of opinion as to whether such a structure is in the province of Civil Engineering or Architecture. The civil engineer might over-emphasize the importance of physical environment, and believe that it lay in his field; the architect might give undue weight to the importance of satisfying the direct human responses of the workers, and believe just as sincerely that it was in the province of Architecture; in such a case, I would suggest that it might properly come under either of the two professions. In classifying a structure as coming under one profession or the other, there is no thought of relegating total activity in any given instance to either profession; the point at issue is, rather, one of attempting to clarify the matter of which should be the principal, and which the subsidiary.

Yet to attempt to define either profession with brevity and precision is to invite criticism. What, for example, do we mean by the verb "build"? The ultimate objective of either Civil Engineering or Architecture is the actual construction that will improve environment, but this is the end result and not the total process. To build, in the broad sense, involves at least five phases, and these are: (1) Overall planning, in which the coordination of widely varying functions is involved, and which, in the last analysis, often resolves itself to the basic problem of land use; (2) Project planning, in which one function usually predominates, and which has as its objective the determination of the general solution that will lead to optimum results; (3) In some instances—as, for example, in some sanitary projects—the development of processes that will accomplish certain functions; (4) Design, which converts the general solution for a project into definitive plans and specifications; and (5) Actual construction, which translates plans and specifications into reality.

To what extent do civil engineers and architects participate in these various phases of building? Civil Engineers emphasize project planning, design, and construction; they sometimes develop processes. The extent to which they do or should participate in overall planning is a matter of importance and will be considered later; for the present, however, we will accept the premise that this is the province of the architect—or, more specifically, of his contemporary—the city or regional planner. Architects emphasize overall planning, project planning, and design; they do not develop processes; they inspect, but do not manage construction.

Perhaps then, Civil Engineering can be more accurately defined as the science and art of the planning, design and construction involved in the building of projects for the primary purpose of improving physical environment. Process development, because it is frequently not a factor, has been omitted in the interest of brevity. From a similar viewpoint, Architecture can be defined as the art and science of the planning and design involved in building for the primary purpose of satisfying direct human responses. Note that in this definition of architecture, building has not been restricted to individual projects, but encompasses the coordination of projects involving widely varying functions—an interpretation not implied in the definition of Civil Engineering. Whether or not this is appropriate, may well be challenged; I propose now to give further consideration to this point.

There is a rather fundamental aspect to this matter that has not, as yet, been stressed sufficiently; and I refer to the type of mental process in which each profession appears to excel. While the civil engineer improves physical environment for people, and is therefore interested in their behavior, yet he sometimes thinks of people as in groups, or in terms of statistical averages. He has an interest in such things as how much water per capita is consumed, or how many miles per year the average motorist drives; but he is even more concerned with the pipes that bring the water to the city dweller, and the highways over which the automobiles are driven. No one knows better than the civil engineer himself that his is far from an exact science; yet for the most part he deals with relatively tangible things—with matters that are susceptible to a certain degree of analysis, which in some areas is fairly rigorous. The architect, however, is primarily concerned with people and their responses. The things he builds are physical, and he must have physical knowledge; yet his greater interest is the people who use his structures and in their reactions to them. He is concerned with people as individuals as well as with groups of people; and because of this, he operates to an important degree in an area of intuitive action—since more often than not he deals with intangibles that are not susceptible to analysis.

To an engineer, the area of intuition may seem vague and unreliable, and so it may be; yet I sometimes wonder if engineers realize how much they themselves depend on it. If we consider the solution of a differential equation, for example, one might approach the matter by a systematic trial of all possible methods of solution, hoping to arrive ultimately at one that will be helpful in the given situation.

The successful mathematician, however, will in some manner sense the correct approach; he may arrive at it directly, or in any event he is likely to solve the equation without a conscious systematic search of all possible approaches.

We are creating a distinction between the formal, intellectual —with its logical, rational, systematized procedures—on the one hand; and the informal, intuitive, instinctive—working without any such system—on the other hand; perhaps as fundamental a difference as between analysis and synthesis. It would seem that the intuitive faculties appear in synthesis as a very important factor. It would seem, too, that the logical process, the analysis, the breaking down of things according to a system, is in itself a method of search—but that this systematizing does not in any way guarantee the ultimate solution of a problem that is unsystematized, and which, perhaps, has not been capable up to the present time of being systematized. The instinctive recognition of the correct procedure for doing something may be quite apart from that systematized analysis which corresponds to the search; the finding of the solution may be intuitive to a marked degree.

Planning—and overall planning in particular—involves synthesis as well as analysis. Many factors must be combined; and these include social, legal, and esthetic aspects as well as those that would be classified as functional, economic, or technical. Planning involves making decisions on the basis of what is best "on the whole." It involves relative judgments, and in this process one deals with intangible as well as tangible values. There are no satisfactory common denominators by which all factors can be compared on a uniform basis, and yet compromises must be reached which, taking all factors into consideration, represent the best total solutions that can be devised. Analysis is a most powerful weapon, and we could not do without it—yet, in my opinion, it is not the total key to creative engineering nor to the planning process; synthesis in thinking is an equally powerful and essential element. If we are to grasp new truths or envision new solutions, we must not belittle the intuitive faculty. This in no sense implies that the fruits of intuition should not be subjected to as rigorous an analysis as is possible, but simply recognizes the limitations of analysis by itself.

It is dangerous to generalize, but I think there is basis for believing that architects—though not as skilled in analysis as are civil engineers—are frequently more gifted as synthesists. This statement should not come as a surprise, for architects, because of their concern

with the response of people, operate to an important degree, as has already been pointed out, in an area of intuitive action—and synthesis appears to be largely intuitive. If it is true that architects excel in synthesis, we should recognize the implications of this in a discussion of the planning process. It would suggest that architects may be more talented planners than are civil engineers. In my opinion, this suggestion is worthy of careful note; yet it is in no way conclusive.

I would take the position that both analysis and synthesis are essential to the planning process, and that to excel in either at the expense of the other, does not lead to the highest level of planning potential. This suggests that planning can be done most effectively by: (1) a civil engineer who is gifted in synthesis as well as in analysis; or (2) an architect who is gifted in analysis as well as in synthesis; or (3) a true collaboration of the civil engineer and the architect. The decision as to which would be preferable in a given instance might well depend, in turn, on whether the planning is primarily concerned with physical environment, or primarily concerned with the satisfying of direct human responses, or whether it is more or less equally concerned with both. Again, there is no thought of relegating total planning activity in any instance to either profession, but, rather, of attempting to clarify the matter of which should be the principal, and which the subsidiary.

I do not deny that planning goes beyond the design of sewers and streets, that its ultimate objective is to improve human environment, that it involves social and economic as well as physical considerations. I do not claim that it is not necessary to protect the community against the specialist who approaches a problem from the point of view of only part of it. But, I do deny that because a man is a specialist in certain areas, he necessarily lacks the breadth of view to treat a problem as a whole; and I cannot see that because a man is not a specialist, he necessarily has the talents to coordinate activities. I do not think it makes very much difference whether the man is a specialist or not; I think it depends primarily on the man himself. I do not claim overall planning as the sole province of the civil engineer, but I am impelled to deny that it is the sole province of the architect.

Thus, in my definition of Civil Engineering, I would remove the implication that planning is restricted to specific projects, and to elaborate on the previous simpler statement, would say that "Civil Engineering is the science and art of: (1) the planning and design involved in building for the primary purposes of improving physical environ-

ment rather than that of satisfying direct human responses; and (2) the management of the heavy construction involved in building for both of these purposes." I would not change in principle my previous definition of Architecture, but to be more specific, would define it as "the art and science of the planning and design involved in building for the primary purpose of satisfying direct human responses rather than that of improving physical environment." In each of these definitions, I would interpret "direct human responses" to mean the emotional and physiological responses of the users of whatever is built.

In this discussion of the relationship between Civil Engineering and Architecture, it has been necessary—for purposes of clarification—to dwell on the differences between the two professions. That there are essential differences—not only in function, but also in temperament and motivation—is inevitable. Yet there is common ground as well; we must not lose sight of the all-important fact that both have the same overall objective—as, indeed, have all the fields of science and engineering—of improving human environment; and that between Civil Engineering and Architecture, the bond is particularly close, since each is concerned with building for this purpose. We should welcome areas of mutual endeavor—where we can work together toward a common goal—and the better each can understand the other, the more effectively can we collaborate.

As we work together, there is—essentially—only one acceptable basis for determining which group should assume leadership in a given instance; this is to be guided by public welfare, and involves a consideration of what each—on the basis of training and experience—is best prepared to do. There can be honest differences of opinion as to how the public can best be served—but this, at least, is the proper approach. No other basis is worthy of either of these two great professions.

ACKNOWLEDGEMENT:

During the preparation of this paper, the writer has had the benefit of discussions with Pietro Belluschi, Dean of Architecture and Planning, Lawrence B. Anderson, Head of the Department of Architecture, and Ernest N. Gelotte, Associate Professor of Construction in the Department of Architecture—all of the Massachusetts Institute of Technology. He wishes also to acknowledge with gratitude the help received, over a number of years, from Dr. A. R. Cullimore, President Emeritus of the Newark College of Engineering, in connection with the subject of synthesis vs. analysis in thinking.

METHODS AND RESULTS FROM M.I.T. STUDIES IN UNSTEADY FLOW

By Henry M. Paynter,* Member

(Presented at a meeting of the Hydraulic Section of the Boston Society of Civil Engineers, held on November 7, 1951.)

General Introduction

With the publication during the summer of 1951 of the first American treatise on hydraulic transients by George R. Rich [1],† and with the welcome visit to M. I. T. during the following winter of Professor Giuseppe Evangelisti, of Bologna, Italy, a renowned authority on unsteady flow, it has seemed fitting to honor these occasions by presenting a brief survey of recent and current work of practical interest in the field of unsteady flow and hydraulic transients, carried on by the Hydrodynamics Laboratory of the M. I. T. Department of Civil and Sanitary Engineering.

This paper is divided into three parts, covering three phases of the program: Part 1—Regulation and Governing of Hydroelectric Plants; Part 2—Graphical Solutions of Transient Problems; Part 3— Flood Routing by Admittance Methods. It is hoped that each part is sufficiently complete in itself to provide information of value and interest to a broad section of engineering fields. However, it must be stated in all fairness that most of the results presented here can be taken only as samples of the material that has been or will be published later in greater detail; numerous references are given to the pertinent literature.

Part 1—Regulation and Governing of Hydroelectric Plants

Introductory

The research program in this field at M. I. T. places particular emphasis on the transient behavior of hydro plants subjected to both large and small changes in operating conditions, taking into account actual performance characteristics of the plant components. The purpose of these studies has been three-fold:

*Assistant Professor of Hydraulic Engineering, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology.

†Bracketed numerals refer to entries in the bibliography at the end of this paper.

a) Development of an overall rational analysis of hydro plant transients with faithful attention to all important variables and characteristics;
b) Application and refinement of mathematical tools of analysis, including machine computation, with consideration of possible extension of these methods to related fields;
c) Execution of systematic general studies and presentation of results in readily available and generally useful form for application by engineers to design calculations.

As an example of the first objective, there is outlined in the paragraphs to follow the methods of analysis whereby general investigations of surge tank transients are made possible, with permanently useful results.

The second aim finds exemplification in Parts 2 and 3 of this paper, wherein concepts originating from the regulation studies have been applied to other unsteady flow problems.

As examples of the nature of the general results obtained and the form into which they have been put, a brief treatment is given of two topics of interest in hydro plant regulation:

a) A new surge formula for simple surge tank design.
b) Optimum governing of hydroelectric units.

Scope of Research on Regulation Problems

The ends of this program of study have required the consideration of the effects on transient behavior of the following plant and system components as indicated in Fig. 1:

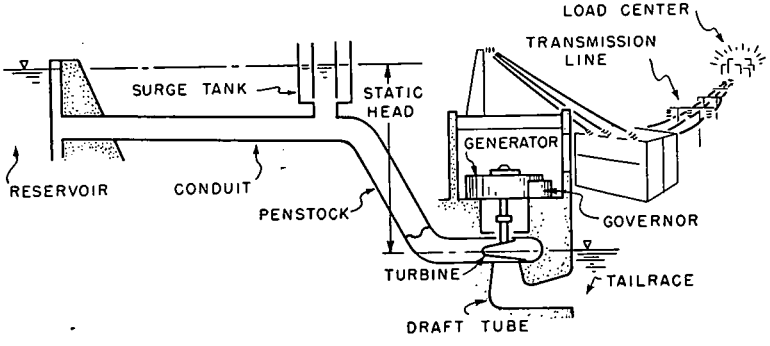


FIG. 1.—REPRESENTATIVE HYDROELECTRIC PLANT.

I— FLOWLINE	II— GENERATING UNIT	III— SYSTEM NETWORK
Including:	*Including:*	*Including:*
Reservoir	Turbine	Transformers
Conduit	Governor	Transmission lines
Surge tank	Generator	System regulation
Penstock	Voltage regulator	Load characteristics
Scroll case		
Draft tube		
Tailrace		

Morever, it has been necessary to treat not only the case of a hydro plant operating alone, but also to investigate the transient behavior of plants operating in parallel with steam or other hydro stations. These analyses have been primarily concerned with:

a) Hydraulic transients: surges and water hammer
b) Machine transients: speed regulation and governing stability
c) System governing: prime mover paralleling and load-frequency control.

A hydroelectric plant under transient conditions following appreciable disturbances manifests a highly *non-linear* response (in the sense that the behavior depends on the magnitude of the changes). This feature, in addition to the overall complexity, makes general solutions of these problems intractable by ordinary mathematical means, and has led to the use of a wide variety of tools, among them:

a) Mathematical analysis
b) Field measurements and laboratory experiments
c) Electrical analogies
d) Electronic computer and differential analyser studies
e) Graphical and numerical solutions.

Application of these techniques has led to the successful general treatment of a substantial part of the problem of transient performance and operating stability of hydroelectric plants, thus extending the pioneer work of other American and European investigators in the field [2] [3] [4] [5] [6] [7] [8] [9]. Chief among the topics treated, the following might be listed:

I—Surge Tank Studies:
a) Normalization of equations
b) Simple tank transients

c) Throttled tank transients
d) Differential tank transients
e) Surge tank stability
f) Pulsing and resonance

II—Water Hammer Studies:
a) Normalization of equations
b) Effects of friction and gate stroke on pressures
c) Efflux energies
d) Resonance and stability problems
e) Surge tank and governor effects.

III—Machine and Governor Studies:
a) Normalization of equations
b) Stability and damping of governors
c) Optimum governor settings
d) Effects of surges and water hammer
e) Effects of turbine characteristics
f) Effects of system and load characteristics
g) Improvements in governing.

IV—System Governing and Load-Frequency Control:
a) Normalization of equations
b) Parallel operation of prime movers
c) Electro-mechanical transients
d) Load-frequency control
e) Improvements in system regulation.

These topics are all treated in some detail in other works and papers of this writer [10] [11] [12] [13] [14] [15].

Surge Tank Equations

In terms of the nomenclature of Fig. 2, the three fundamental equations describing the behavior of a cylindrical *differential* surge tank may be written as follows:

Continuity: $$Q_c = A_t \frac{dY}{dt} + A_r \frac{dX}{dt} + Q_p \qquad (1.1)$$

Acceleration: $$\frac{L_c}{gA_c} \frac{dQ_c}{dt} + K_c \mid Q_c \mid Q_c + X = 0 \qquad (1.2)$$

Throttling: $$X = Y + K_t A_t^2 \left| \frac{dY}{dt} \right| \frac{dY}{dt} \qquad (1.3)$$

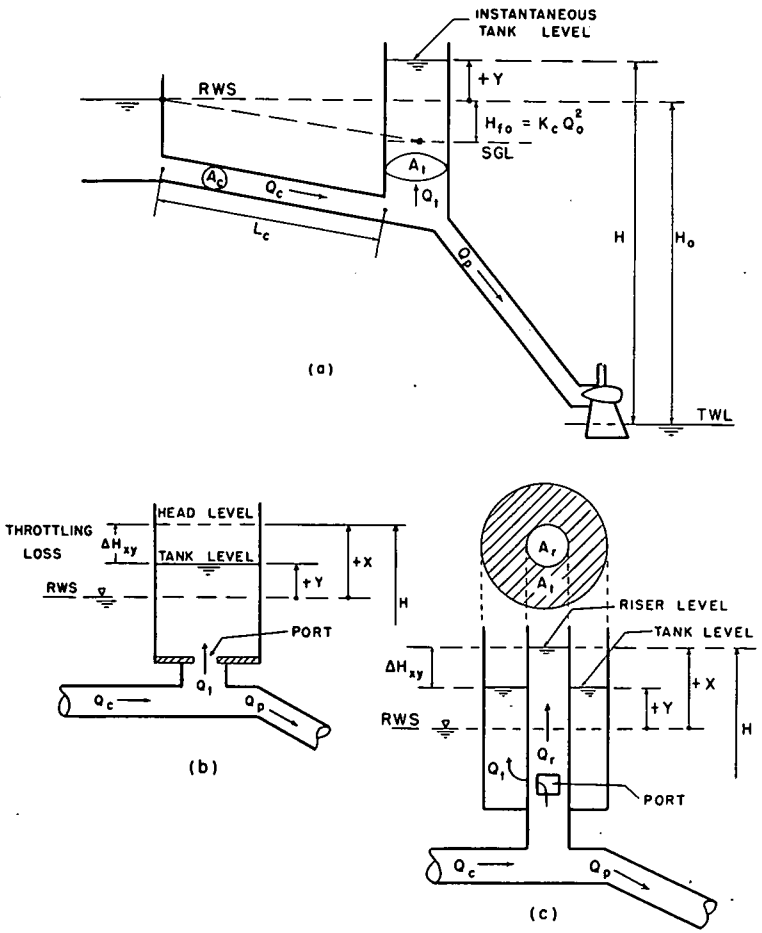


FIG. 2.—SURGE TANK NOMENCLATURE.

These equations are valid so long as neither tank nor riser spill, and so long as the port remains fully submerged. The restriction to "quadratic" conduit and throttling losses may be removed by considering the loss coefficients K_c and K_t as variables rather than constants. The purpose of the absolute value brackets on these loss terms is to indicate the necessary reversal of sign of the loss upon reversal of flow. The variables and constants might be listed as follows:

(A)—Variables

Flows:		Levels:	
Conduit Flow:	Q_c;	Tank Level:	Y
Penstock Flow:	Q_p;	Riser Level:	X

(B)—Constants

Areas:		Length:		Losses:	
Conduit Area:	A_c;	Conduit Length:	L_c;	Conduit Loss:	K_c
Tank Area:	A_t;			Tank Throttling:	K_t
Riser Area:	A_r;				

The simple and restricted-orifice tanks (the latter hereafter called a *throttled* tank) are merely special cases of the differential tank. This follows from the suppression of the appropriate constants in equations (1.1) (1.2) (1.3). In particular, if $A_r = 0$, then the tank would become a throttled tank since the internal riser would be absent; in this case the variable X becomes the *head level* at the base of the surge tank, and is above or below the tank level Y depending on the direction of flow through the port. If, in addition, the throttling coefficient K_t is made zero, then the tank becomes a *simple* tank, with the reduced equations:

$$Q_c = A_t \frac{dY}{dt} + Q_p \tag{1.4}$$

$$\left(\frac{L_c}{gA_c}\right) \frac{dQ_c}{dt} + K_c \,|\, Q_c \,|\, Q_c + Y = 0 \tag{1.5}$$

In all that follows the demand flow Q_p will be considered as the source of the disturbance to the equilibrium. If this flow is reduced suddenly from an initial value $Q_{p1} = Q_o$ to a final value $Q_{p2} = 0$, and if in equation (1.5) K_c is assumed for the moment to be zero, it is simple to show that Q_c and Y will oscillate in pure harmonic motion, with the following characteristics:

$$\text{Period} = 2\pi \sqrt{\frac{L_c}{g} \frac{A_t}{A_c}} = T_{ct} = \textit{Free Period} \tag{1.6}$$

$$Y_{max} = +Q_o \sqrt{\frac{L_c}{gA_cA_t}} = +Y_o = \textit{Free Surge} \qquad (1.7)$$

$$Y_{min} = -Y_o$$

$$Q_{max} = +Q_o$$

$$Q_{min} = -Q_o$$

The oscillations would, of course, continue indefinitely in the absence of friction. In practice, such a frictionless condition can only be approached, but never reached, and it might at first be thought that the foregoing solution has no physical significance. However, this is not true since the free period and free surge can serve as basic reference values. If one defines a new set of dimensionless variables, including the time variable, for equations (1.1), (1.2), and (1.3), the equations may be put into a completely dimensionless, or *normalized* form, suitable for general investigation.

Such a set of transformed variables are the following:

Flows:

Dimensionless Conduit Flow Ratio: $\bar{u} = Q_c/Q_o$ (1.8)

Dimensionless Demand Flow Ratio: $\bar{v} = Q_p/Q_o$ (1.9)

Levels:

Dimensionless Tank Level: $\bar{y} = Y/Y_o$ (1.10)

Dimensionless Riser (or Head) Level: $\bar{x} = X/Y_o$ (1.11)

Time:

Dimensionless Time Variable: $\tau = 2\pi\,(t/T_{ct})$ (1.12)

In terms of these variables, the equations (1.1), (1.2), and (1.3) become:

Continuity: $$\bar{u} = \frac{d\bar{y}}{d\tau} + B\,\frac{d\bar{x}}{d\tau} + \bar{v} \qquad (1.13)$$

Acceleration: $$\frac{d\bar{u}}{d\tau} + \frac{R}{2}\,|\,\bar{u}\,|\,\bar{u} + \bar{x} = 0 \qquad (1.14)$$

Throttling: $$\bar{x} = \bar{y} + F\left|\frac{d\bar{y}}{d\tau}\right|\frac{d\bar{y}}{d\tau} \qquad (1.15)$$

Thus the seven original constants (including the gravitational acceleration) are reduced to the three independent parameters:

Riser Area Parameter: $B = A_r/A_t$ (1.16)

Conduit Friction Parameter: $R = 2H_{fo}/Y_o$ (1.17)

Tank Throttling Parameter: $F = \Delta H_{xyo}/Y_o$ (1.18)

where H_{fo} and ΔH_{xyo} are the friction loss and throttling loss, respectively, corresponding to the index flow Q_o.

The principal benefits occuring from such normalization procedures are the following:

(a) All redundant constants have been eliminated;
(b) Similitude features are revealed;
(c) Greater speed and ease of solution is secured;
(d) Smaller numbers of solutions are required to obtain general conclusions.

The second feature permits the use of model studies and the subsequent extension of results to field installations. Moreover generalized model studies may be made, as at M. I. T., wherein the results are treated in the same way as numerical and analytical studies, ending with the construction of generalized charts obtained from actual water measurements. The third benefit is made evident in the studies of Part 2 of this paper. The last advantage suggests that more knowledge can be gained from a lesser number of solutions if the results are put in normalized form. The curves so obtained permit ready interpolation, and to some extent extrapolation, for particular instances.

It is finally of interest to note the following scheme of parameters:

I — Differential Tank
Three parameters: R, F, B

II — Throttled Tank ($B = 0$)
Two parameters: R, F

III — Simple Tank ($B = 0$) ($F = 0$)
One parameter: R

Surge Tank Formula

As an example of the benefit of normalizing techniques toward obtaining general solutions of surge tank problems, there is presented in the following paragraphs a useful formula that has been derived for the level changes in a simple tank following a sudden change in demand flow.

The equations for the simple tank in normalized form follow from equations (1.13) and (1.14), namely

$$\bar{u} = \bar{v} + \frac{d\bar{y}}{d\tau} \tag{1.19}$$

$$\frac{d\bar{u}}{d\tau} + \frac{R}{2} \mid \bar{u} \mid \bar{u} + \bar{y} = 0 \tag{1.20}$$

These may be still further reduced for the special case of sudden (step) changes in demand flow, by the introduction of the variables:

$$\begin{aligned} u &= R_o\bar{u} \\ y &= R_o\bar{y} \end{aligned} \tag{1.21}$$

where R_o is some index value of the friction parameter. In these terms, the equations become:

$$u = v + \frac{dy}{d\tau} \tag{1.22}$$

$$\frac{du}{d\tau} + \frac{1}{2} \mid u \mid u + y = 0 \tag{1.23}$$

Thus the parameter R has disappeared entirely from the equations. However, any particular demand flow change, from an initial value $Q_D = Q_1$ to a final value $Q_D = Q_2$ is now represented by corresponding initial and final values of the demand flow variable v. Thus:

$$\begin{aligned} v_1 &= u_1 = R_1 \\ v_2 &= u_2 = R_2 \end{aligned} \tag{1.24}$$

where, as before, R is the ratio of twice the friction loss to the free surge for the particular flow. Thus, the values

$$R_1 = \frac{2H_{f1}}{Y_1} = \frac{2K_cQ_1^2}{Q_1\sqrt{\dfrac{L_c}{gA_cA_t}}} = \left[2K_c\sqrt{\frac{gA_cA_t}{L_c}}\right] Q_1 \tag{1.25}$$

$$R_2 = \frac{2H_{f2}}{Y_2} = \frac{2K_cQ_2^2}{Q_2\sqrt{\dfrac{L_c}{gA_cA_t}}} = \left[2K_c\sqrt{\frac{gA_cA_t}{L_c}}\right] Q_2 \tag{1.26}$$

completely determine the initial and final steady states, and therefore the nature of the solutions of equations (1.22) and (1.23). This is

the identical system of normalization used by Milne in his valuable studies of these equations [16] [17] [18]. It is hoped that this exposition will serve to make his work more useful to engineers.

A result of this generalization has been the ability to represent the level change in a simple tank under the foregoing conditions, by the following formula, derived from energy considerations:

$$z_m = -\Delta R \left[\sqrt{1 + \left(\frac{K\Sigma R}{2}\right)^2} + \left(\frac{K\Sigma R}{2}\right) \right] \qquad (1.27)$$

where z_m is the level change in these "R" units; $\Sigma R = R_2 + R_1$; $\Delta R = R_2 - R_1$; and the surge factor K depends on the initial and final steady states. Thus, symbolically,

$$K = K\ (R_1, R_2) \qquad (1.28)$$

A curve of the values of K corresponding to these values of the friction parameters R_1 and R_2 is given in Fig. 3. Tables A and B give the

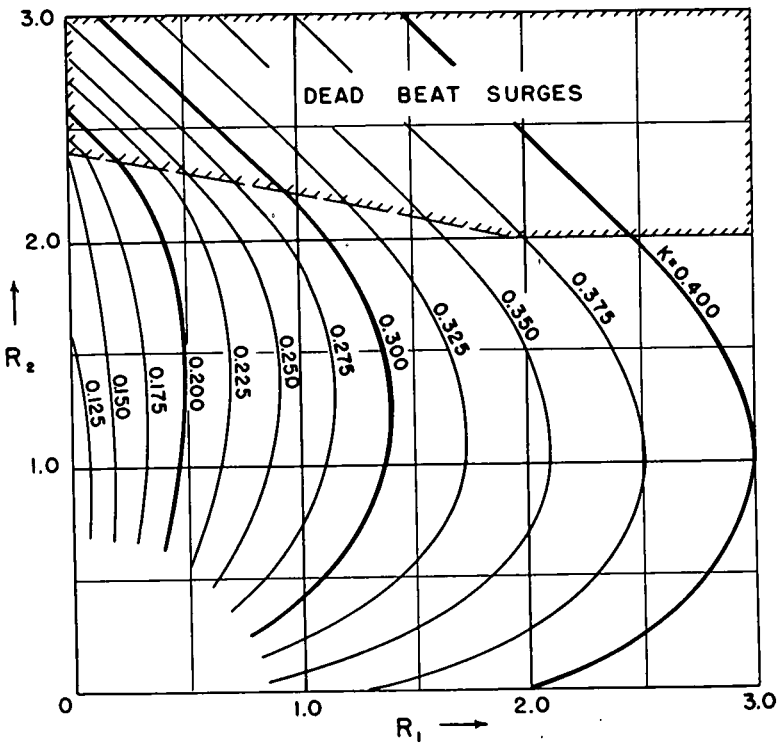


FIG. 3.—VALUES OF SURGE FACTOR—K.

values for small R and large R, respectively. A good approximate formula for values of R_1 and R_2 less than unity may be written:

$$K = \left(\frac{0.10R_2 + 0.33R_1}{\Sigma R}\right)(1 + 0.1\Sigma R) \qquad (1.29)$$

The surge formula, equation (1.27), may also be written directly in physical terms, probably more useful for engineering calculations, as:

$$\text{Level Change} = \sqrt{(\Delta Y_o)^2 + (K\Delta H_f)^2} + (K\Delta H_f) \qquad (1.30)$$

Here ΔH_f and ΔY_o measure the friction gradient change and surge increment, respectively, where

$$\Delta H_f = |H_{f2} - H_{f1}| = K_c\,|Q_2^2 - Q_1^2| \qquad (1.31)$$

$$\Delta Y_o = |Y_2 - Y_1| = |Q_2 - Q_1|\sqrt{\frac{L_c}{gA_cA_t}} \qquad (1.32)$$

The following example serves to show the application of this formula to a practical case.

Example 1

I — Plant Data: See Example 3 of Part 2.

II — Tank Parameters:

Free Period: $T_{ct} = 251$ seconds
Free Surge: $Y_o = 25.00$ feet
Tank Parameter: $R_o = 0.50$

III — Transients:

(1) *Full Load Rejection*: $R_1 = 0.50$; $R_2 = 0$

Value of K: (a) From Table A: $K = 0.348$
(b) From Eq. (1.29) $K = 0.35$

Surge Increment $\Delta Y_o = 25.00$ ft.; Gradient Change $\Delta H_{f_0} = 6.25$ ft.
$K\Delta H_{f_0} = 2.19$ ft.

Level Change $= \sqrt{(25.0)^2 + (2.19)^2} + (2.19) = 27.29$ feet

(2) *Full Load Acceptance*: $R_1 = 0$; $R_2 = 0.50$

Value of K: (a) From Table A: $K = 0.101$
(b) From Eq. (1.29): $K = 0.10$

Surge Increment $\Delta Y_o = 25.00$ ft.; Gradient Change $\Delta H_{f_0} = 6.25$ ft.
$K\Delta H_{f_0} = 0.63$ ft.

Level Change $= \sqrt{(25.0)^2 + (0.63)^2} + (0.63) = 25.71$ feet

(3) *Half to Full Load*: $R_1 = 0.25$; $R_2 = 0.50$

Value of K: (a) From Table A: $K = 0.187$
(b) From Eq. (1.29): $K = 0.19$

Surge Increment $\Delta Y_0 = 12.50$ ft; Gradient Change $\Delta H_{f_0} = 4.69$ ft.
$K\Delta H_{f_0} = 0.89$ ft.

$$\text{Level Change} = \sqrt{(12.5)^2 + (0.89)^2} + (0.89) = 13.42 \text{ feet}$$

TABLE A
Values of the Surge Factor K
Small Values of R

	Values of R_1										
R_2	0	0.1	0.2	.03	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.0	*	.336	.339	.342	.345	.348	.352	.355	.358	.361	.364
0.1	.097	.219	.256	.280	.296	.307	.318	.325	.333	.340	.345
0.2	.098	.178	.223	.244	.265	.280	.293	.303	.312	.320	.327
0.3	.099	.158	.199	.227	.246	.260	.274	.286	.296	.305	.313
0.4	.100	.147	.184	.210	.232	.248	.260	.272	.283	.293	.300
0.5	.101	.141	.173	.200	.220	.237	.250	.263	.274	.283	.293
0.6	.102	.137	.166	.192	.211	.227	.242	.255	.266	.275	.285
0.7	.103	.135	.161	.185	.204	.220	.235	.248	.259	.269	.278
0.8	.105	.133	.157	.180	.198	.214	.229	.242	.253	.263	.273
0.9	.107	.132	.155	.176	.193	.209	.224	.237	.248	.258	.267
1.0	.110	.132	.154	.173	.189	.205	.220	.232	.243	.254	.263

TABLE B
Values of the Surge Factor K
Large Values of R

	Values of R_1												
R_2	0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
0	*	0.34	0.35	0.36	0.36	0.37	0.38	0.39	0.40	0.40	0.41	0.41	0.42
0.25	0.10	0.23	0.27	0.30	0.32	0.34	0.35	0.36	0.38	0.39	0.40	0.41	0.42
0.50	0.10	0.19	0.24	0.27	0.29	0.31	0.33	0.34	0.36	0.37	0.38	0.40	0.41
0.75	0.10	0.17	0.22	0.25	0.28	0.30	0.32	0.33	0.35	0.36	0.38	0.39	0.40
1.00	0.11	0.16	0.21	0.24	0.26	0.29	0.31	0.33	0.34	0.36	0.37	0.39	0.40
1.25	0.12	0.16	0.20	0.23	0.26	0.29	0.31	0.33	0.34	0.36	0.38	0.39	0.40
1.50	0.12	0.17	0.20	0.23	0.26	0.29	0.31	0.33	0.35	0.37	0.38	0.39	0.41
1.75	0.13	0.17	0.20	0.24	0.27	0.30	0.32	0.34	0.36	0.38	0.38	0.40	0.41
2.00	0.14	0.18	0.22	0.25	0.28	0.31	0.34	0.36	0.375	0.39	0.39	0.41	0.42
2.25	0.14	0.20	0.24	0.28	0.31	0.34	0.36	0.38					
2.50	0.18	0.23	0.28	0.31									
2.75													
3.00													

Dead Beat Surges
(below solid line)

Optimum Governing of Hydroelectric Units

Another representative study undertaken at M. I. T. concerns the proper adjustment of the settings of hydro governors to provide optimum regulation in an interconnected system. In such studies the use of the electronic analog computer has been invaluable [12] [32].

The principles of stabilization of the governor of a hydroelectric unit are indicated in Fig. 4. The left diagram illustrates the behavior

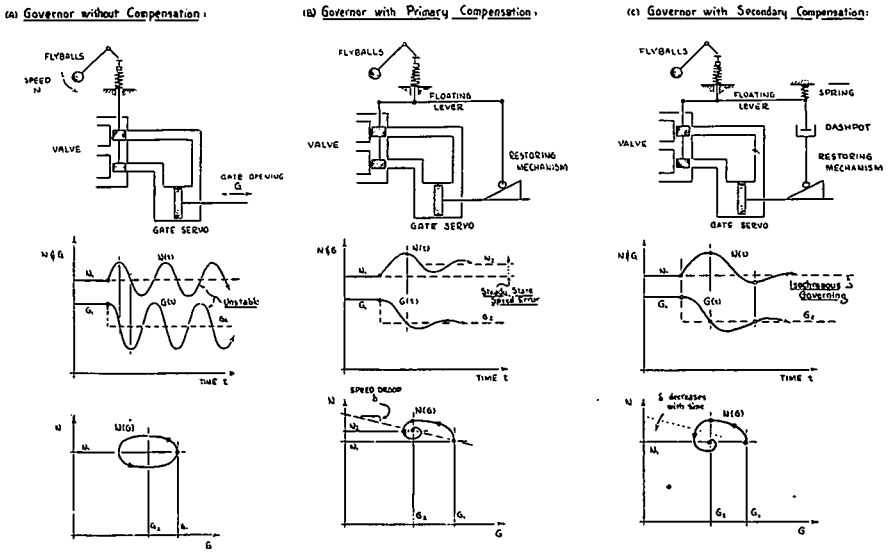


FIG. 4.—STABILIZATION OF GOVERNORS.

of the governed unit if no compensating mechanism is provided: a steady hunting would take place. The center panel, Fig. 4(b), demonstrates that provision of a restoring effect from gate servo back to the floating lever will stabilize the unit but will result in a *droop* between gate opening and speed such that the unit is left with a steady state speed error. This error may be removed and the governor made *isochronous* by putting a spring-dashpot mechanism in the restoring linkage as indicated in Fig. 4(c); in this case the droop may be said to "run out" in time at a rate depending on the dashpot needle setting. These two elements of stabilization are called first or *primary* compensation (restoring effect) and second or *secondary* compensation (dashpot effect). These settings for any particular unit, may be

measured by the speed droop (δ) that would exist if the dashpot needle were fully closed (making the normally temporary droop fixed), and by the "relaxing" or recovery time of the dashpot (T_r) that could be observed if the dashpot pistons were given an initial displacement with normal needle setting. These constants and their adjustments may thus be related directly to the governor mechanism as shown in Fig. 5. It is important to note that the temporary droop (δ) is not the same as the adjustable but permanent droop (σ) which is required for paralleling prime movers in an interconnected system.

Also in Fig. 5, the effects of the adjustments of primary and secondary compensation are indicated by curves which were taken directly from the oscilloscope screen of the electronic computer and which have been amply confirmed by field tests. The speed transients indicated in the figure are those following sudden small load increments ΔP, with the instantaneous speed N plotted against time t. The origin of coordinates for each trace indicates the corresponding relative values of primary and secondary compensation. For best governing in hydro plants both settings will vary directly with the *water inertia or acceleration time* T_w, where:

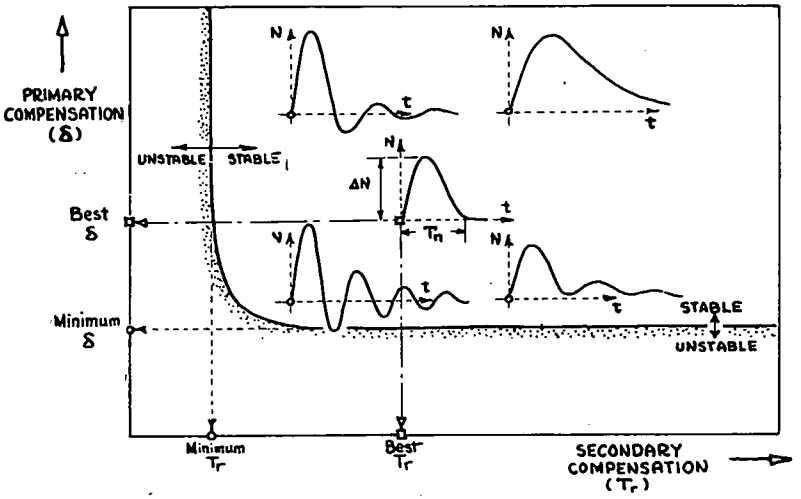
$$T_w = \frac{\Sigma(LV)}{gH_o} \quad \text{(in } \textit{seconds}\text{)} \tag{1.33}$$

and where $\Sigma\,(LV) = Q_o \int dL/A$ is the effective length-velocity (*square feet/second*) measuring the momentum of the entire water column from intake (or surge tank) to *draft tube exit*; and H_o is the rated head (*feet*) of the turbine. The best restoring effect (primary compensation) setting will depend also on the magnitude of the *machine inertia* or *acceleration time* T_m, which is defined as:

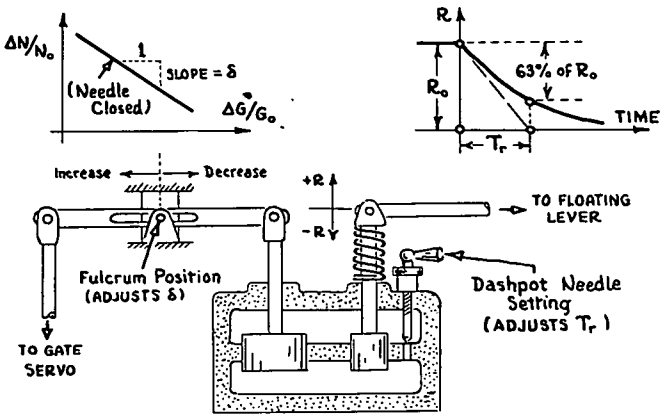
$$T_m = \frac{N_o^2\, WR^2}{1.6 \times 10^6\, P_o} \quad \text{(in } \textit{seconds}\text{)} \tag{1.34}$$

where N_o is the prime mover synchronous speed (*rpm*); WR^2, the flywheel effect of runner and rotor (*lb-ft²*); and P_o, the rated capacity (*hp*).

As a single example of the numerical values of these settings and the consequent transient behavior, one may consider the extreme case of an isolated hydro unit carrying a purely resistive load. In this case, for best governing with a sensitive governor and small increments:



(a) EFFECT of GOVERNOR SETTINGS



(b) RELATION of SETTINGS to DASHPOT

FIG. 5.—GOVERNOR SETTINGS.

Temporary Droop: (Needle closed) $\delta = 2 \cdot \left(\frac{T_w}{T_m}\right)$ [$\times$100 for percent]

Dashpot Time: (Needle normal) $T_r = 5 \cdot (T_w)$ [in *seconds*]

The characteristics of the step-load-change transient indicated in Fig. 5 that corresponds to these settings (again for small load changes ΔP) can be expressed as follows:

Relative Speed Change: $$\left(\frac{\Delta N}{N_o}\right) = 2.5 \cdot \left(\frac{T_w}{T_m}\right) \cdot \left(\frac{\Delta P}{P_o}\right)$$

Transient Time: $$T_n = 8 \cdot (T_w)$$

The water acceleration time (T_w) varies from less than 0.5 seconds for medium and high head plants with pressure regulation to more than 3 seconds for some large, low head plants. Thus the important need for sufficient machine WR^2 becomes apparent even with the best governing possible. Moreover, plants with high water inertia will necessarily be poorer regulators of system frequency than those with low water inertia, due to length of their transient intervals, which in extreme cases may be more than 20 seconds.

Part 2—Graphical Solutions of Transient Problems

Introductory

As an example of a method of analysis which was developed primarily to fill a definite need in the hydraulic transient studies, but which has obvious applications to broad fields of engineering, there are presented in the paragraphs to follow the principles of a graphical procedure called the *slopeline* method.

No claim is made as to the originality of this method, in as much as the basic notions have been in use for some time. However, certain new applications are briefly outlined here which are felt to be of considerable usefulness in engineering calculations.

Principles of the Slopeline Method

The slopeline method is a technique of graphical integration useful for solving a wide variety of engineering problems represented by differential equations. As a preliminary example, let it be desired to determine the solution of the equation:

$$\frac{dy}{dx} = f(x), \qquad \text{or,} \qquad dy = f(x)dx \tag{2.1}$$

in the form $y = y(x)$ (i.e. y the running integral of x), for continuous values of x, and where $f(x)$ is given in (or can be put into) graphical form, as shown in Fig. 6(a).

The basic assumption of the slopeline method lies in the use of the relation:

$$\Delta y = [f]_{ave} \cdot \Delta x \tag{2.2}$$

to give the change in y, namely Δy, corresponding to the change Δx, assuming that the average value of f over the interval Δx is given by the arithmetic mean, thus:

$$[f]_{ave} = \frac{1}{2}(f_1 + f_2) \tag{2.3}$$

where f_1 and f_2 are the values at beginning and end of the interval respectively. Thus the increment Δy becomes:

$$y = \frac{1}{2}(f_1 + f_2)\,\Delta x = \frac{\Delta x}{2}(f_1 + f_2) \tag{2.4}$$

This means that the trapezoidal method of integration is employed. The basis of the graphical construction is to evaluate Δy in two parts so that:

$$\Delta y = \left(\frac{\Delta x}{2}\right) f_1 + \left(\frac{\Delta x}{2}\right) f_2 \tag{2.5}$$

This is performed graphically as shown in Fig. 6(b). From the diagram it may be seen that:

$$\Delta y_1 = f_1 \tan \alpha = f_1 \left(\frac{\Delta x}{2}\right)$$

where $\tan \alpha = (\Delta x/2)$ is the tangent of the slopeline drawn *downwards* from f_1 to the base and the slopeline drawn *upwards* from that point to the top of f_2. The *physical* slope of the slopelines is determined by the scales of f and y, since $(\Delta x/2) = \tan \alpha = (\Delta y_1/f_1) = (\Delta y_2/f_2)$.

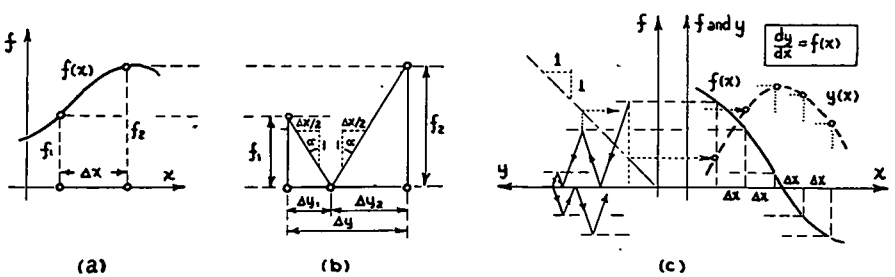


FIG. 6.—PRINCIPLES OF SLOPELINE METHOD.

Finally, then, it is clear that the total increment Δy is determined as the sum of the two parts or:

$$\Delta y = \Delta y_1 + \Delta y_2 = \left(\frac{\Delta x}{2}\right) f_1 + \left(\frac{\Delta x}{2}\right) f_2$$

which was the desired result from Eq. (2.5). A very convenient arrangement for carrying out this process is indicated in Fig. 6(c).

In this way given any initial value of f, say f_o, and the corresponding value y_o, a running integral or continuous function y (x) may be found merely by drawing a series of slopelines starting from the initial values f_o and y_o, going to the base, then back and forth to each subsequent ordinate. The 45° line indicated serves merely to permit the graphical transfer of the y—values from the horizontal to the vertical scale for the final plot of y (x).

The validity of this method corresponds to the precision of the trapezoidal approximation, and errors may generally be reduced merely by decreasing the interval Δx, (thus "steepening" the slopelines). This method has a marked superiority over all other known graphical methods of comparable simplicity, giving a maximum precision of results for any fixed interval, thus permitting the use of fairly large intervals without causing serious error, provided that the interval points are reasonably distributed with respect to the given function f(x).

A very similar method in all essentials forms the basis of a recent paper by K. E. Sorensen [19]. However, the germ ideas were derived by the present writer from certain obvious analogies to the graphical method for water hammer and elastic waves as initially proposed by Schnyder [20] in 1929, and later refined by Angus [21], Bergeron [22], and other writers. The techniques of Bergeron are outlined in superb detail in his last book [23]; in this work the suggestiveness of a more general application of this slopeline technique is apparent. It was after study of this last mentioned treatise in 1950 that the methods outlined here were developed by the writer to rapidly check results obtained by the electronic computer. This method was first used in practice in January, 1951, for the design of a restricted orifice surge tank in connection with the redevelopment of the Ripogenus Plant of the Great Northern Paper Company, under the engineering supervision of the Stone and Webster Engineering Corporation.

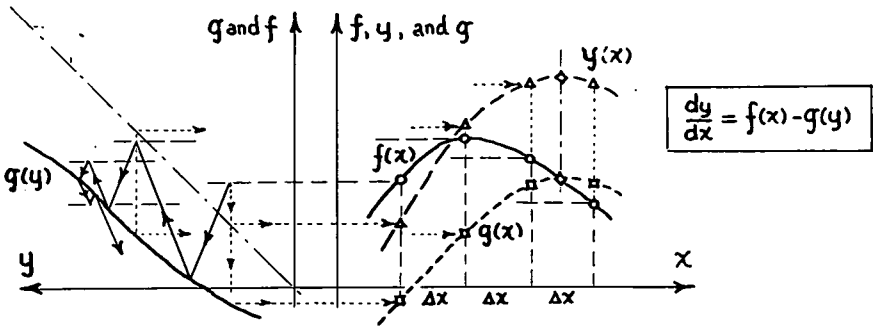


FIG. 7.—SOLUTION OF FIRST ORDER EQUATIONS.

Solution of First Order Equations

The application of the slopeline construction to the solution of first order differential equations of the form

$$\frac{dy}{dx} + g(y) = f(x) \qquad (2.6)$$

follows in a straightforward way. In the left hand plot of Fig. 7 the function g(y) is now plotted. Then the slopelines, instead of going all the way to the base, "reflect" from this function g(y) in order to represent the mean value of the integrand, $[f - g]_{ave}$, over the interval. Thus, since the change Δy is given by:

$$\Delta y = [f - g]_{ave} \cdot \Delta x \qquad (2.7)$$

the variation in y may be determined interval by interval. The behavior of the slopelines upon change in sign of [f — g] in this figure corresponds to that of [f] alone in Fig. 6. When f = g it is clear from Eq. (2.7) that the change in y is zero, and y has an extreme value.

It should be particularly noted that the same y — g diagram may be used for many different "disturbance" functions f(x), without additional plotting other than the appropriate slopelines; this implies a substantial economy of time when many solutions are desired for a particular system.

Application to Flood Routing Through Reservoirs

The governing equation for reservoir flood routing may be written:

$$\frac{dS}{dt} = Q_i - Q_o \qquad (2.8)$$

where S = storage volume (prismatic only); t = time; Q_i = inflow; Q_o = outflow. The surcharge storage volume is related to the depth H above the spillway crest through the relationship

$$S(H) = \int_0^H A(H) \cdot dH \qquad (2.9)$$

where A (H) is the reservoir surface area corresponding to any depth H. Thus a curve or tabular relationship can be obtained between S and H as shown in Example 2, Table III.

The outflow Q_o may generally be assumed to vary with the head H above the spillway crest according to the expression:

$$Q_o(H) = MLH^{3/2} \qquad (2.10)$$

where M (H) is the spillway coefficient, depending on the depth H, and L is the spillway length. Such a relationship is shown in Table II of the example. If the operation of outlet sluices were to be considered in addition to spillway outflow, additional families of curves could be obtained in the form $Q_o = Q_o(n,H)$ where n represents the particular configuration of outlets and control works.

Combining equations (2.9) and (2.10), one may write, symbolically,

$$Q_o = Q_o(S) \qquad (2.11)$$

which signifies that for any value of storage volume S, there is a corresponding outflow over the spillway Q_o. The points for this curve or table may be obtained by evaluating the expressions for Q_o and S according to equations (2.9) and (2.10) for the same values of H. Such a *storage-outflow* curve is indicated in Table IV and plotted in the left hand side of Figs. 8 and 9.

The basic differential equation may then be rewritten in the form:

$$\frac{dS}{dt} + Q_o(S) = Q_i(t) \qquad (2.12)$$

which is of the same form as equation (2.6), with t corresponding to x, S to y, Q_i to f, and Q_o to g. The solution procedure becomes identical.

The steps in one cycle of the slopeline method as applied to reservoir routing are outlined below Fig. 8. A numerical example follows.

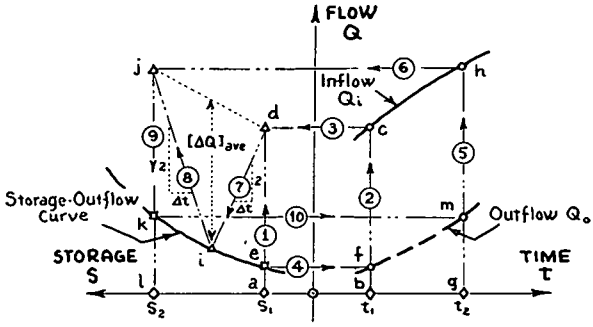


FIG. 8.—APPLICATION TO RESERVOIR ROUTING.

Steps in One Cycle of the Slopeline Method for Reservoir Routing

DESCRIPTION OF LINE

(1) Beginning at point (a) with the initial value of storage S_1 corresponding to time t_1, a vertical line is drawn upwards.

(2) From point (b) at time t_1, a vertical is drawn upwards to the intersection with the inflow curve at point (c), which is Q_{i1}.

(3) From point (c) a horizontal line is drawn to the intersection with Line (1) at point (d).

(4) From point (e) at the intersection of Line (1) with the storage-outflow curve, a horizontal is drawn to the right to point (f), the interesection with Line (2), giving the outflow Q_{o1} at the beginning of the interval.

(5) From point (g) at time t_2 a vertical is drawn upwards to the intersection with the inflow curve at point (h), giving Q_{i2}, the inflow at the end of the interval.

(6) From point (h) a horizontal is drawn toward the left.

(7) From point (d), a DOWNWARD SLOPELINE is drawn to the intersection with the storage-outflow curve at point (i).

(8) From point (i), an UPWARD SLOPELINE is drawn to the intersection with Line (6) at point (j).

(9) From point (j), a vertical is drawn downward to intersect the storage curve at point (k) and the axis at point (l), giving the final value of the storage, S_2.

(10) From point (k), a horizontal is drawn to the intersection with Line (6) at point (m), giving the outflow Q_{o2}, at the end of the interval, thus completing the cycle.

EXAMPLE 2: RESERVOIR AND FLOW DATA AS GIVEN IN TABLES I, II, AND III

I — Inflow Hydrograph:

Plotted in Figure 9

t HOUR	Q_i AF/HR	t HOUR	Q_i AF/HR
0	0	12	850
2	200	14	700
4	400	16	550
6	600	18	400
8	800	20	250
10	1000	22	100

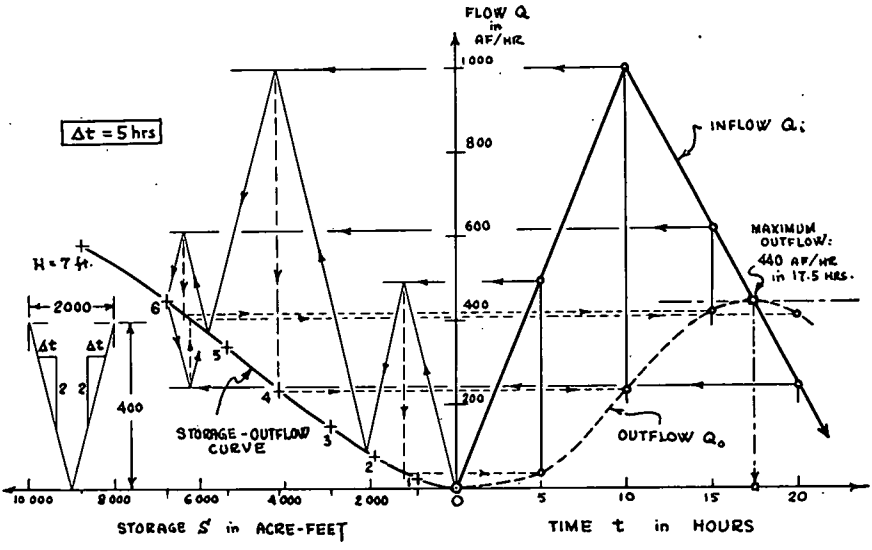


FIG. 9.—SLOPELINE METHOD FOR RESERVOIR ROUTING.

II — Spillway & Outflow Data

Spillway Equation: $Q_o = MLH^{3/2}$

Spillway Crest Length $L = 100$ feet

H	M	$H^{3/2}$	Q_o	Q_o
FEET	—	—	CFS	AF/HR
0	(3.0)	0	0	0
1	3.10	1.00	310	26
2	3.20	2.83	905	75
3	3.35	5.20	1742	145
4	3.50	8.00	2800	233
5	3.60	11.18	4025	336
6	3.70	14.70	5440	453
7	3.75	18.52	6950	580

III — Reservoir Areas and Storage Volumes

H	A	$A_m = \frac{A_1 + A_2}{2}$	$\Delta S = A_m \Delta H$	S
FEET	ACRES	ACRES	AF	AF
0	910			0
		930	930	
1	950			930
		975	975	
2	1000			1905
		1040	1040	
3	1080			2945
		1140	1140	
4	1200			4085
		1270	1270	
5	1340			5355
		1420	1420	
6	1500			6775
		1600	1600	
7	1700			8375

IV — Computed Storage — Outflow Curve
Plotted in Figure 9

H	S	Q_o
FEET	AF	AF/HR
0	0	0
1	930	26
2	1905	75
3	2945	145
4	4085	233
5	5355	336
6	6775	453
7	8375	580

V — Computation of Slope Lines

Taking $\Delta t = 5$ hours, then a slopeline triangle may be found for plotting purposes from the proportion:

$$\frac{\Delta t}{2} = \frac{5}{2} = \frac{\text{Storage Increment } (\Delta S)}{\text{Flow Increment } (\Delta Q)}$$

If one takes, say, $\Delta Q = 400$ AF/HR, then:

$$\Delta S = \frac{5}{2} \times 400 = 1000 \text{ AF}$$

Accordingly a pair of opposing sloping triangles are constructed in Figure 9, with:

$$\text{BASE} = 1000\text{AF}$$
$$\text{ALTITUDE} = 400\ \text{AF/HR}$$

using the scales of the coordinates on the drawing.

Solution of Second Order Equations

A general class of second order equations and vibration problems may be reduced through scale changes and transformations of variables to the simultaneous solution of the two first-order equations of the form:

$$\frac{dz}{dx} + f(z) + y = 0 \qquad (2.13)$$

$$-\frac{dy}{dx} - g(x, y) + z = 0 \qquad (2.14)$$

where x is the independent variable, y and z, the dependent variables, and f and g, functions of the arguments indicated. As an example of such a system, one might consider the equations for a simple surge tank in the normalized form developed in Part 1, as follows:

$$\frac{du}{d\tau} + f(u) + y = 0 \qquad (2.15)$$

$$-\frac{dy}{d\tau} - v(\tau, y) + u = 0 \qquad (2.16)$$

In this case $\tau = 2\pi\ t/T_{ct}$ is the normalized time variable; u and y are the normalized flow and level variables respectively, and f and v are the friction and demand characteristics, respectively. While all vibration and surge tank problems may be treated by the slopeline method *without normalization,* a substantial gain in time and effort is secured by this transformation procedure. Accordingly, only problems in this form will be here considered.

The key to the application of the methods of the previous paragraphs to the solution of such equations lies in the notion of *simultaneous solution.* If, as in Fig. 10 (a), one considers the values of y in equation (2.13) as fixed and specified as sketched, and similarly, in Fig. 10 (b), the values of z in equation (2.14) to be known, then the solution of each equation, *independently,* would follow in the direct

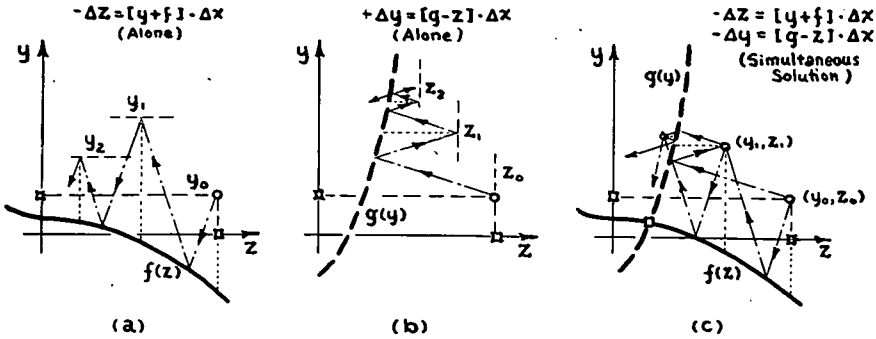


FIG. 10.—SOLUTION OF SECOND ORDER EQUATIONS.

fashion indicated. However, actually all that is known about the values of y and z is that their changes must satisfy both equations (2.13) and (2.14), so that the change Δy is given by:

$$-\Delta y = [g - z]_{ave} \cdot \Delta x \tag{2.17}$$

and the change Δz from

$$-\Delta z = [y + f]_{ave} \cdot \Delta x \tag{2.18}$$

The slopeline method permits the simultaneous application of these conditions interval by interval, giving rise to the graphical construction on Fig. 10 (c).

Slopeline Solutions of Surge Tank Transients

The normalized (completely dimensionless) simple tank equations may be written again in the form:

Acceleration: $$-\frac{du}{d\tau} = y + f(u) \tag{2.19}$$

Continuity: $$-\frac{dy}{d\tau} = v(\tau, y) - u \tag{2.20}$$

where the notation follows from the previous sections.

As an example of the application of the slopeline method, consider the transient resulting from sudden rejection of full load in a plant with the following data:

EXAMPLE 3

Plant Data: Conduit Diameter: $D_c = 10$ feet

Conduit Length: $L_c = 3220$ feet

Conduit Velocity: (Full Load) $V_o = 10$ ft/second

Conduit Friction Loss: (Full Load) $H_{fo} = 6.25$ feet

Tank Diameter: $D_t = 40$ feet

Tank Parameters:

Free Period: $$T_{ct} = 2\pi \frac{D_t}{D_c}\sqrt{\frac{L_c}{g}} = 251 \text{ seconds}$$

Free Surge: $$Y_o = V_o \frac{D_c}{D_t}\sqrt{\frac{L_c}{g}} = 25.0 \text{ feet}$$

Tank Parameter: $$R_o = 2H_{fo}/Y_o = 0.50$$

Transient Conditions:

Full Load Rejection Study:

$$\text{Initially,} \quad v_1 = u_1 = 0.50; \quad y_1 = -\frac{R_1^2}{2} = -0.125$$

$$\text{Finally,} \quad v_2 = u_2 = 0; \quad y_2 = 0$$

The slopeline solution for this case is shown in Fig. 11, where the additional assumption of square-flow friction loss has been made in this instance. Both the horizontal and vertical systems of slopelines have been drawn with the same slope; this can be done whenever the equations are in normalized form, with obvious advantages. However, instead of using an angle α of slopelines, given by the expression

$$\tan \alpha = \frac{\Delta\tau}{2} \quad \text{or} \quad \alpha = \tan^{-1}\left(\frac{\Delta\tau}{2}\right) \tag{2.21}$$

which would follow logically from the slopeline theory just developed, it may be shown that greater precision is possible if α is taken as:

$$\alpha = \Delta\tau/2 \tag{2.22}$$

In this way, as damping effects become less important, the graphical solution becomes *exact* even with appreciable increments $\Delta\tau$. Moreover, it had been found that an approximate optimum balance of speed and precision occurs if the angle and increment be taken so as to divide the time into one-sixteenth parts of the free period T_{ct}; for the friction-

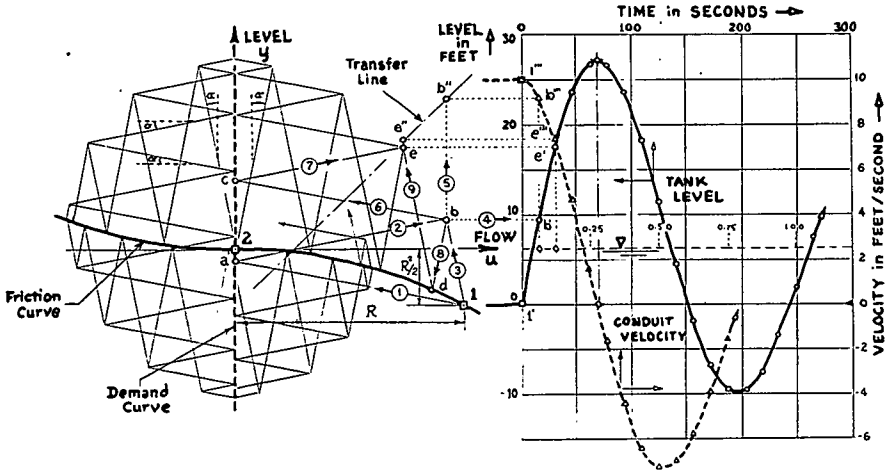


FIG. 11.—APPLICATION TO SURGE TANKS.

Steps in Typical Cycles of the Slopeline Solution of Surge Tank Transients

DESCRIPTION OF LINE

(1) From point (1), the initial steady state, a "LEVEL" SLOPELINE (LSL) is drawn to the left to the intersection with the *demand curve* at point (a).
(2) From point (a), a (LSL) is drawn to the right.
(3) From point (1), a "FLOW" SLOPELINE (FSL) is drawn upwards to the intersection with Line (2) at point (b).
(4) From point (b), a horizontal is drawn to the right to plot point (b′), the *tank level* at the end of the first interval Δt.
(5) From point (b), a vertical is drawn upwards to the transfer line at point (b″), thence horizontally to plot point (b‴), the *conduit velocity* at the end of the first interval.
(6) Repeating now, from point (b) a (LSL) is drawn left to point (c).
(7) From point (c), another (LSL) is drawn to the right.
(8) From point (b), a (FSL) is drawn downwards to the intersection with the friction curve at point (d).
(9) From point (d), a (FSL) is drawn upwards to the intersection with Line (7) at point (e). The level and flow points are plotted as before.

less case (R approaching zero), there would be determined then, four points in each quadrant, or sixteen points per cycle, of the corresponding free vibration. This corresponds to an angular time increment $\Delta\tau = (1/16) \times 360° = 22°30'$ or $\pi/8$ radians. Then the slopeline angle α is one-half this value or:

$$\alpha = \Delta\tau/2 = 11°15' \qquad (2.23)$$

A smaller time interval could be used, but it would result in increased labor generally disproportionate to the small gain in precision, while a larger increment proves impractical due to a rapid loss in precision,

together with the difficulty of accurately tracing a smooth curve with an insufficient number of points.

The steps for one cycle of the slopeline process applied to simple tanks are indicated below Fig. 11. Also shown are auxiliary scales which convert graphically from normalized measure to physical units of length, velocity, and time.

Sample solutions (without detailed explanation) are depicted in Fig. 12 for three other cases of interest:

a) Simple tank: slow closure;
b) Simple tank: part load acceptance with constant power;
c) Throttled tank: part load acceptance.

The slopeline method has been refined to treat rapidly all normally encountered transient disturbances in these tanks, as well as in the true differential tank. Tanks of variable cross-section and systems of surge tanks may also be handled readily by this approach. The utility of the procedure for studies of mechanical and electrical vibrations is also appreciable. Full particulars will soon be available in other works by the writer [14] [24].

Part 3—Flood Routing by Admittance Methods

Introductory

The use of impedance and admittance functions to characterize the behavior of transients and oscillations in a linear system is an old, "tried and true" method; for example, it forms the basis of all electrical network theory. However, the direct and conscious application of these concepts to hydraulic transient problems has been little appreciated to date, although such methods seem promising. Outlined in the following paragraphs is a preliminary accounting of some current studies underway at M. I. T. to explore the possibilities of improved flood routing procedures making use of these concepts.

Admittances and Flood Routing

In the absence of simple physical laws describing the behavior of changing flows in a reach of a river, it is not reasonable to abandon too hastily the useful concepts of linearity and superposition. While all real phenomena are non-linear to a greater or lesser degree (in the sense that the governing relationships are not just simple proportionalities involving the variables and their rates of change), nevertheless, the use of linear tools with appropriate caution and modification

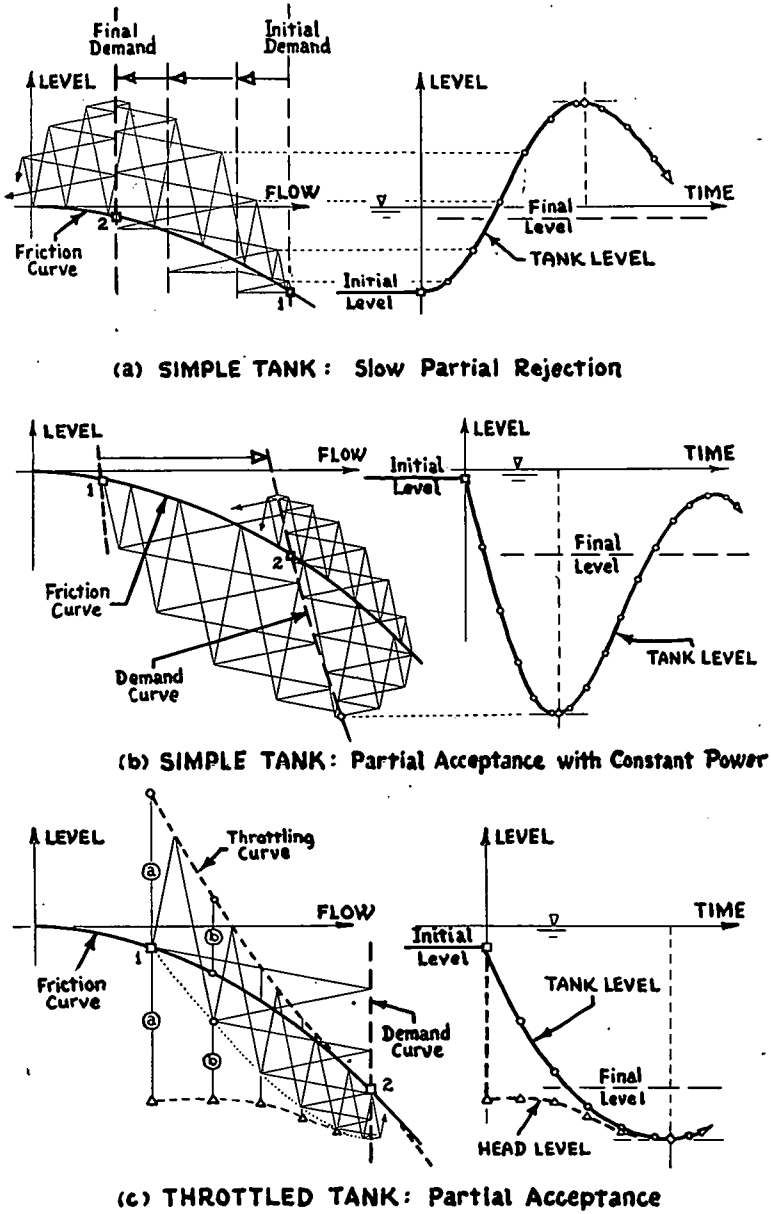


FIG. 12.—SURGE TANK SOLUTIONS.

can lead to many valuable methods and results. This concept has been at the heart of all rational physical and engineering analysis.

Accordingly, it might be asked: what is the most general assumption of linear response that one could make? The answer is simple: subject to the single assumption that superposition is valid, the response of any system can always be represented by the Duhamel superposition theorem:

$$y(t) = x_0 A(t) + \int_0^t A(t-\tau) \cdot d[x(\tau)] \qquad (3.1)$$

where x (t) is the INPUT or DISTURBANCE; y (t) the OUTPUT or RESPONSE; and A (t), called the INDICIAL RESPONSE or ADMITTANCE, represents the relation between input x and output y, and is the response of the system to a unit step input as shown in Fig. 13. This relationship is capable of describing very complex systems either with lumped elements and a finite number of degrees of freedom (such as vibratory systems and electrical networks) or with continuous elements and an infinite number of degrees of freedom (such as flood flows in rivers, elastic waves, long transmission lines, and heat flow problems).

An excellent introductory treatment of admittances and the superposition theorem is given in the book of von Karman and Biot [25]. However, a brief derivation of the basic formula is in order.

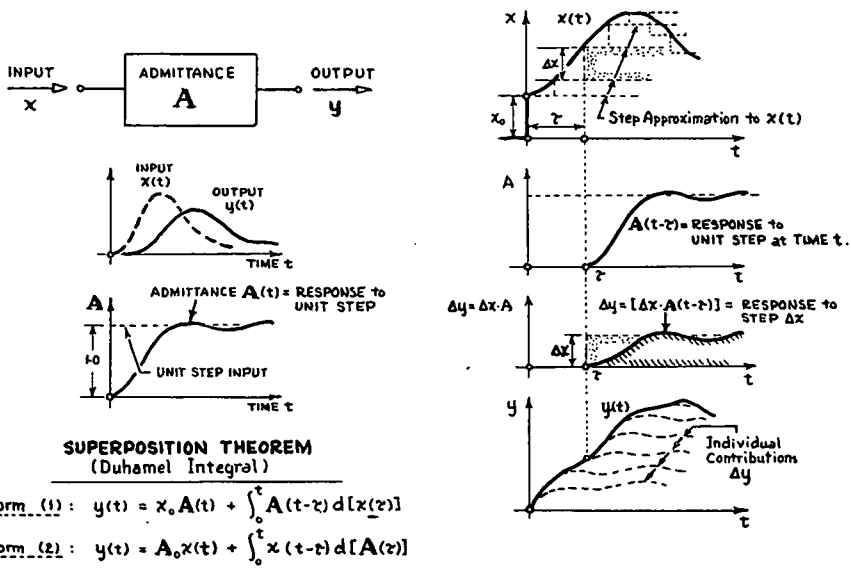


FIG. 13.—SIGNIFICANCE OF ADMITTANCE FUNCTION.

Referring to Fig. 13, A (t) is shown to be the output resulting from a unit step input. The general input x(t) may be considered to be the sum of a series of small steps. The height of the step at a certain time τ is shown as Δx. The result of applying such an input, Δx, to the system is to give rise to an output which has the same shape as A(t), but which begins at time τ and has a magnitude proportional to the input Δx, thus:

	INPUT	OUTPUT
Unit Step:	1	$A(t-\tau)$
Step:	Δx	$\Delta x \cdot A(t-\tau)$

If the results of all such steps are added together, with the height of each step becoming infinitesimally small, the result is the Duhamel integral.

There are a number of marked advantages in considering flood routing problems in this light; among them one might list the following:

(1) It is the most general and universally applicable linear method.

(2) All existing linear techniques of routing may be considered as special instances of this method.

(3) No additional assumptions or equations are required.

(4) The relationship between x (t) and y (t), as characterized by A (t), is always possible to find and is unique.

(5) The admittance function A (t) can be determined from past flood records.

(6) On the other hand, A (t) can also be related to the characteristics and real properties of the stream.

(7) The method is amenable to rapid manual computation, as well as machine solutions and analog representations.

(8) It is possible to work with gage heights directly and use stage-routing methods.

(9) Floods may be routed successfully and simply through long channel reaches.

(10) It is not absolutely necessary to correct initially for ground-water increases, tributary or local inflows; these may always be investigated and corrections made after the analysis.

Such methods are not new to the hydraulic field; the unit hydrograph method is an outstanding example of this approach applied to the relation between rainfall and runoff in a stream. However, the conventional practice in this case of using only a "unit storm" is not a necessary part of the linear treatment. Rather, the above formulation suggests that considerable knowledge is derivable from every past storm, regardless of the space-wise or time-wise distribution of rainfall over the area.

Significant Properties of Admittances

Just as in any AC or DC electrical network one may write:

$$e_{input} = \tilde{Z} \cdot i_{output}$$
$$i_{output} = \tilde{A} \cdot e_{input} \qquad (3.2)$$

where $\tilde{Z}$ = Impedance, and $\tilde{A} = 1/\tilde{Z}$ = Admittance, so one can write for any linear system:

$$\underbrace{\begin{bmatrix} \text{output} \\ \text{or} \\ \text{effect} \\ \text{or} \\ \text{response} \end{bmatrix}}_{y} = \underbrace{\begin{bmatrix} \text{Admittance} \end{bmatrix}}_{\tilde{A}} \cdot \underbrace{\begin{bmatrix} \text{input} \\ \text{or} \\ \text{cause} \\ \text{or} \\ \text{disturbance} \end{bmatrix}}_{x} \qquad (3.3)$$

However, one must consider the admittance $\tilde{A}$ as an *operator*, or operation, which when applied to the input x gives the output y. Every linear system may be characterized by a unique operator $\tilde{A}$. If a series of such systems $\tilde{A}_1$, $\tilde{A}_2$, etc., are arranged in tandem such that the output from the first becomes the input to the second, and so forth, then the results may be expressed as follows:

For three such systems, there would result:

$$x \rightarrow [\tilde{A}_1] \rightarrow [\tilde{A}_2] \rightarrow [\tilde{A}_3] \rightarrow y$$
$$y = [\tilde{A}_1 \cdot \tilde{A}_2 \cdot \tilde{A}_3]\, x$$
$$y = \tilde{A}_s : x$$

and for n systems:

$$x \rightarrow [\tilde{A}_1] \rightarrow [\tilde{A}_2] \rightarrow \ldots \rightarrow [\tilde{A}_n] \rightarrow y$$
$$y = [\tilde{A}_1 \cdot \tilde{A}_2 \cdot \ldots \cdot \tilde{A}_n]\, x$$
$$y = [\Pi \tilde{A}_k]\, x$$
$$y = \tilde{A}_s \cdot x$$

In short, any sequence, finite or infinite, of linear admittances will result in an effective resultant admittance $\tilde{A}_s$, which is the product of all the admittances in the chain.

This resultant admittance $\tilde{A}_s$, relating output y to input x depends

on the particular nature of the system and completely determines the response of the system to any particular disturbance. As just mentioned $\tilde{A}$ is not generally a simple constant but is rather an operator. By means of various integral transformations, such as the Laplace or Fourier transformations, this function may be expressed as an *algebraic function,* such as:

Laplace Representation: $\tilde{A} = \tilde{A}(p)$ where $p =$ Complex number

Fourier Representation: $\tilde{A} = \tilde{A}(\omega)$ where $\omega =$ Angular frequency

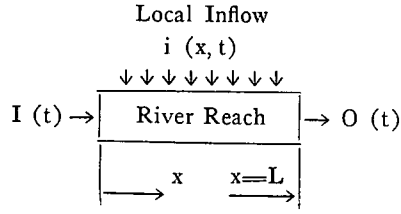
For details of such representations, see, for example, the works of Gardner and Barnes [26], Churchill [27], Jaeger [28], as well as the Karman-Biot text previously cited.

However, as a much more practical expedient, for the present purpose, it is of interest to consider the representation of such operators in the *time domain*; in this case A becomes A (t), the INDICIAL RESPONSE OR ADMITTANCE FUNCTION, which is the response of the particular system to a *unit step input* as indicated in Fig. 13. Application of A to the input x becomes *convolution* as represented by the Duhamel integral.

As applied to the present problem, one may consider the input x to be the *inflow I,* and the output y, as the *outflow O,* of a particular river reach. The admittance function A(t) will be considered as characterizing the behavior of the reach.

While the generality of the admittance concepts includes the possibilities of propagation and diffusion as the flood wave moves downstream, together with continuous variation of the stream properties along the length of the reach, yet it is important to mention at the outset a serious difficulty to be overcome in the practical application of this method, as well as all other methods: the problem of continuous local inflow along the length of the reach, which is in the last analysis at least partly unknown and unknowable. Moreover, this "inflow" may even in extreme cases become *negative* (outflow) due to evaporation and to leakage *into* the groundwater table.

As a model for this process, one may consider the following scheme:



At least conceptually, this situation may be considered "lumped" into the following admittance scheme:

$$I \rightarrow [A_1] \xrightarrow{i_1} [A_2] \xrightarrow{i_2} \ldots \xrightarrow{i_{n-1}} [A_n] \rightarrow O$$

Even so, the errors in determining or estimating the local inflows i_1, i_2, etc., will in turn cause corresponding errors in the determination of the admittances A_1, A_2, etc.

A possibly fruitful model for a drainage basin is shown in Fig. 14.

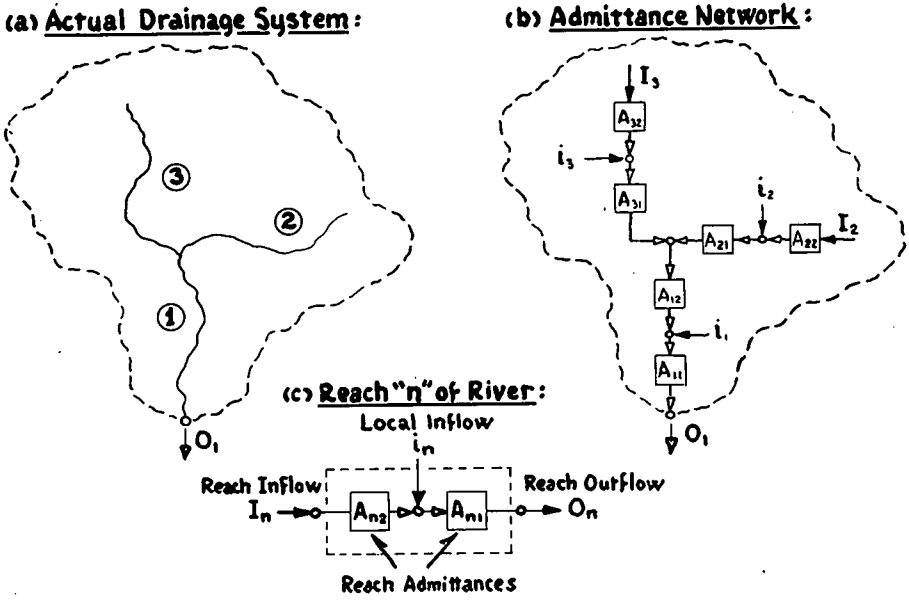


FIG. 14.—FLOOD ROUTING WITH ADMITTANCE NETWORK.

The physical streams are replaced by a network of admittances which are here shown to have but one intermediate junction for concentrating all local inflow.

Admittances of Existing Methods

In Fig. 15 are shown the admittance functions of several of the more common routing procedures. These will now be discussed.

The upper part of the figure, Fig. 15(a), shows the admittance corresponding to a pure travel delay of a flood wave, with no distor-

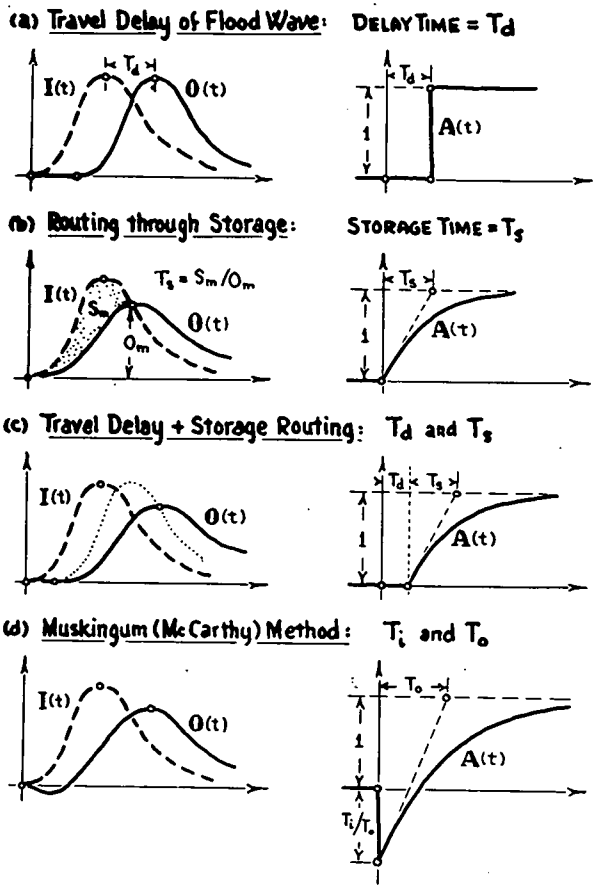


FIG. 15.—ADMITTANCE FUNCTIONS OF EXISTING ROUTING METHODS.

tion. It is clear that A(t) is merely another unit step displaced in time by the delay T_d.

A common source of routing techniques has been the lumped storage or continuity equation:

$$\frac{dS}{dt} = I - O \tag{3.4}$$

previously employed in Part 2 under reservoir routing. If only prismatic storage is considered, and the relationships to depth are con-

sidered *linear*, then storage S may be considered as related to outflow O by the relation:

$$S = T_s \cdot O \tag{3.5}$$

where T_s is the storage time constant and is related to both the general flood and to the admittance as indicated in Fig. 15(b).

The resulting differential equation is first order, and has the form:

$$T_s \frac{dO}{dt} + O = I \tag{3.6}$$

The admittance function in this case is given by:

$$A(t) = 1 - e^{-t/T_s} \tag{3.7}$$

The combination of travel delay with prismatic storage is shown in Fig. 15(c). The result follows directly from (a) and (b) above.

The widely and successfully used "Muskingum Method" outlined by G. T. McCarthy in 1938 [29] is based upon an assumed storage function taking into account both prismatic and wedge storage in the form:

$$S = T_i I + T_o O \tag{3.8}$$

for use with equation (3.4). In conventional notation the storage time constants T_i and T_o are given by:

$$\begin{aligned} T_i &= Kx \\ T_o &= K(1-x) \end{aligned} \tag{3.9}$$

The resulting first order differential equation has the form:

$$T_o \frac{dO}{dt} + O = I - T_i \frac{dI}{dt} \tag{3.10}$$

A general flood and corresponding admittance for this equation are shown in Fig. 15(d). In this case the admittance A(t) is given by:

$$A(t) = 1 - \left(\frac{T_i + T_o}{T_o}\right) e^{-t/T_o} \tag{3.11}$$

It is found that for most reaches T_i approximately equals T_o. The physical interpretation of the initial negative values resulting from this method is difficult to find; any investigation of the actual continuous mechanisms involved in the passage of waves will suggest that such a phenomenon is a characteristic of the method and not of the

natural stream. This subject will be mentioned again in a later paragraph.

Note that in all the preceding cases, as with all idealized instances where there is no net local inflow (or outflow), the volumetric continuity condition

$$\int_0^\infty I(t)\ dt = \int_0^\infty O(t)\ dt \tag{3.12}$$

results in the *necessary* condition on admittances that

$$A(t) \rightarrow 1.00 \text{ as } t \rightarrow \infty \tag{3.13}$$

Experimentally determined admittances which fail this condition may be considered to be in error through the effects of local inflow.

Routing Procedure with Given Admittance

In order to appreciate the significance of the application of admittance concepts to flood routing, one may consider the simplest case of routing a known inflow I (t) through a reach with known admittance A(t), under the assumption of no local inflow, to obtain the final outflow O(t).

The superposition theorem can best be used numerically in its finite form, assuming an initially zero value of inflow (i.e. $I_o = 0$):

$$O_n = \sum_{k=1}^{n} a_{n+1-k} \cdot \bar{I}_k \tag{3.14}$$

which becomes, on expansion:

$$O_1 = a_1 \cdot \bar{I}_1$$

$$O_2 = a_2 \cdot \bar{I}_1 + a_1 \cdot \bar{I}_2$$

$$\vdots$$

$$O_n = a_n \cdot \bar{I}_1 + a_{n-1} \cdot \bar{I}_2 + \ldots + a_1 \cdot \bar{I}_n$$

The distribution coefficients a_k are merely the changes in admittance ΔA_k corresponding to the interval used, as shown in Fig. 16. The values of inflow $\bar{I}$ are taken as the mean values of inflow I over the interval of integration.

It may be seen from Fig. 16 that the continuity condition of

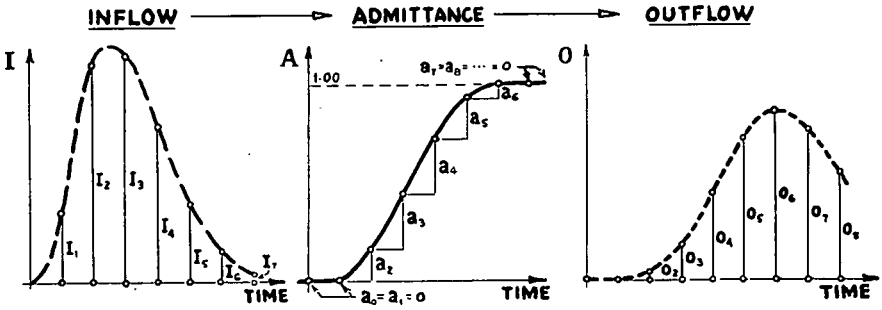


FIG. 16.—NUMERICAL ROUTING WITH GIVEN ADMITTANCE.

equation (3.13) imposes a condition on the distribution coefficients a_k which represent the admittance in this finite form, namely:

$$\sum_k a_k \equiv 1.0 \tag{3.14}$$

In the case of oscillating admittance functions (not shown) the coefficients a_k might be both positive and negative. It should be noted in passing that the distribution coefficients here are analogous to the distribution coefficients of the unit hydrograph method.

TABLE C

Example of Admittance Routing

		k	0	1	2	3	4	5	6	7	8	...
		a_k	0	0	0.16	0.29	0.28	0.20	0.07	0	Σ=1.00	---
k	I_k	I_k \ O_k	0	0	14	84	221	362	428	385	271	..
0	0	90	0	0	14	26	25	18	6	0	0	...
1	180	365	— →	0	0	58	106	102	73	26	0	...
2	550	565	— —	→	0	0	90	164	159	113	40	...
3	580	490	— —	— —	→	0	0	78	142	137	98	...
4	400	300	— —	— —	— —	→	0	0	48	87	84	...
5	200	140	— —	— —	— —	— —	→	0	0	22	41	...
6	80	50	— —	— —	— —	— —	— —	→	0	0	8	...
7	20	...	— —	— —	— —	— —	— —	— —	→	0	0	...

Table C details the calculation procedure corresponding to Fig. 16. The resemblance to the unit hydrograph procedure becomes clear at this point. Indeed, in this form, the admittance method becomes the generalization of the method proposed by Tatum [30], and the so-called "non-storage" routing procedures discussed in the book of Linsley, Kohler, and Paulhus [31].

It is altogether possible to perform the routing procedure using automatic computing equipment, either in step-form using equation (3.14) or in continuous form using equation (3.1), which may be rewritten for flood applications in the form:

$$O(t) = \int_0^t A(t-\tau)\, dI(\tau) = \int_0^t I(t-\tau)\, dA(\tau) \qquad (3.16)$$

There now exists high-speed electronic computing equipment of commercial make [32], which will permit the continuous solution of both the direct problem (Given I and A to find O), as well as the inverse problem (Given I and O to find A), which will be discussed in the next article. Fig. 17 shows schematically the nature of such computer components for a single river reach.

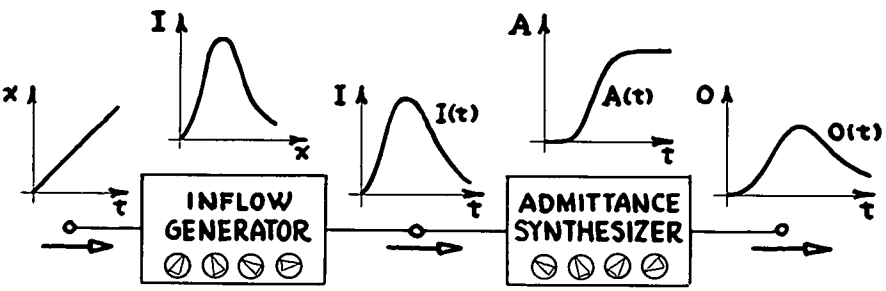


FIG. 17.—AUTOMATIC ADMITTANCE ROUTER.

Determination of Flood Admittances

As mentioned at the outset of this presentation, the flood admittances of natural channels may be determined both on the basis of past flood records and from rational considerations.

The determination of A(t) where I(t) and O(t) are known functions is essentially a so-called "integral equation" problem. For manual computation, using the finite difference form of the integral, equation (3.14), it might at first be supposed that a simple inversion of the chain of linear equations:

$$
\begin{aligned}
O_1 &= a_1 \bar{I}_1 \\
O_2 &= a_2 \bar{I}_1 + a_1 \bar{I}_2 \\
O_3 &= a_3 \bar{I}_1 + a_2 \bar{I}_2 + a_1 \bar{I}_3 \\
&\text{(etc.)}
\end{aligned}
\tag{3.17}
$$

in the form:

$$
\begin{aligned}
a_1 &= O_1/\bar{I}_1 \\
a_2 &= (O_2 - a_1 \bar{I}_2)/\bar{I}_1 \\
a_3 &= (O_3 - a_2 \bar{I}_2 - a_1 \bar{I}_3)/\bar{I}_1 \\
&\text{(etc.)}
\end{aligned}
\tag{3.18}
$$

would solve this problem. However, in practice, the application of equations (3.18) gives rise to unstable calculations, since both numerators and denominators on the right side are generally small. As more workable alternatives, one may set up a chain of linear regression equations for the a_k and use successive approximation techniques for solutions, or one may use a series of convergent approximations to the distribution coefficients a_k directly.

Two items of existing automatic computing equipment offer possibilities of considerable assistance for this inversion problem. The first is an integral equation analyser at M. I. T., upon which solutions of admittance functions have been successfully obtained. The second machine is the linear synthesizer previously referred to and sketched in Fig. 17, which is capable of solving rapidly both the direct and inverse problem. As pictured, a first element is capable of generating an arbitrary wave form representing $I(t)$, while the second component, capable of either direct or trial adjustment, synthesizes the admittance $A(t)$. The output from this component therefore represents the outflow $O(t)$. It would be possible to set up a network of these components to simulate the drainage system of Fig. 14. With such a computer assemblage, one would be able to route and predict floods continuously, and continually revise the estimates of reach inflows and local inflows, as well as the admittance functions themselves. A flood analyser in this form would be more versatile and flexible than the existing machines developed for this task [33] [34] [35].

As a result of preliminary studies using the manual computation and machine techniques outlined above, it has been found that in many cases, the admittances of river reaches may be represented with

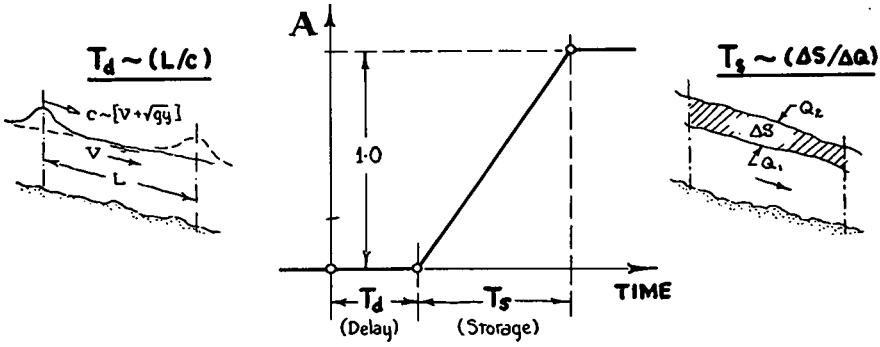


FIG. 18. APPROXIMATE FORM OF ADMITTANCE FUNCTION FOR MOST NATURAL STREAMS.

surprising accuracy by the simple function shown in Fig. 18. This function can be described by just two constants: the travel delay T_d and the storage-distortion time T_s. As indicated, the travel delay may be associated with the time of wave propagation down the reach, while the storage time is related to the ratio of the valley storage increment ΔS to the equilibrium flow increment $\Delta Q = \Delta I = \Delta O$. Since this admittance resembles that of Fig. 15(c), it might be stated that the effect of routing through a normal river reach is roughly equivalent to the combined effect of a pure delay and a single storage lag; this feature has been suspected for some time, being mentioned, for example, in the paper of Clark [36].

Another rich source of qualitive and quantitive information in flood admittances comes through an analogy existing between the equations for unsteady flow in a uniform channel and those describing transients in an electrical transmission line; this analogy is indicated in Table D. For relatively small flow changes (small floods) the analogy is quantitatively valid, while for major floods the parallelism becomes merely qualitative. The advantage of such an analogy lies in the fact that there is a large body of electrical experience which may be applied, with appropriate limitations, to the flood routing problem. For example, one significant conclusion that has been drawn from theoretical and experimental studies of the transmission line problem, and which is directly applicable to flood admittances, might be stated as follows (with the hydraulic interpretation given parenthetically) If the transmission line (or *natural channel*) is of sufficient length, regardless of the original frequencies or intensities of the input signal

TABLE D

Flood Wave — Electrical Analogy

	Uniform Water Channel		Uniform Electrical Line	
A — Equations:	$-\frac{\partial y}{\partial x} = \frac{1}{g}\frac{\partial v}{\partial t} + mv$		$-\frac{\partial e}{\partial x} = L\frac{\partial i}{\partial t} + Ri$	
	$-\frac{\partial v}{\partial x} = \frac{1}{d}\frac{\partial y}{\partial t}$		$-\frac{\partial i}{\partial x} = C\frac{\partial e}{\partial t}$	
B — Variables:	Depth Change	y	Voltage	e
	Velocity Change	v	Current	i
	Time	t	Time	t
	Distance	x	Distance	x
C — Constants	Unit Inertia	(1/g)	Unit Inductance	(L)
	Unit Storage	(1/d)	Unit Capacitance	(C)
	Unit Resistance	$(m = V_o/C_o^2 R_o)$	Unit Resistance	(R)
D — Properties:	Wave Celerity:	$c = \sqrt{gd}$	Wave Celerity:	$c = 1/\sqrt{LC}$
	Surge Impedance:	$Z_o = \sqrt{d/g}$	Surge Impedance:	$Z_o = \sqrt{L/C}$
	Travel Delay:	$T_d = l/c$	Travel Delay:	$T_d = l/c$
	Diffusion Time:	$T_s = (m/d)l^2$	Diffusion Time:	$T_s = (RC)l^2$

to the line (or *flood inflow*), the resistance term Ri (or mV) will eventually become predominant compared to the inductive term $L \frac{\partial i}{\partial t}$ (or *inertia* $\frac{1}{g} \frac{\partial v}{\partial t}$). This means that at a point sufficiently removed from the input or sending end of the transmission line, the admittance function relating the signal at this point to the input signal closely resembles Fig. 18 and is of a pure delay plus diffusion form. Knowledge of this sort is very helpful when applied to the problem of determining flood admittances from past records, making suitable corrections and allowances for unknown local inflows.

Another clue to the general significance of the admittance function shown in Fig. 18 comes from the fact that if the Muskingum equation (3.10) is applied to a channel in such a way that the reach is subdivided into many small parts, all with the same time constant $T_i = T_o$, the resulting admittance resembles Fig. 18 if the number of subdivisions is very large. However, even with many such sections, the admittance still has an initial sudden jump to —1 (for an *odd* number) or +1 (for an *even* number). These jumps cannot be physically meaningful, due to the finite time for a disturbance to traverse a reach of a natural stream.

Non-Linear Effects

Attempts have been made in recent years to introduce non-linear storage equations, such as:

$$S = aI^m + bO^n \tag{3.19}$$

into the "lumped" continuity equation

$$\frac{dS}{dt} = I - O \tag{3.20}$$

in order to obtain a better fit with past flood hydrographs. While the four constants a, b, m, n, may be considered to have a rational origin, nevertheless it is found that for no two floods will the coefficients for best fit be quite the same. Moreover, for any given storage function of the form of equation (3.19), it is rarely possible to fit past hydrographs exactly. The presently proposed linear admittance technique, on the other hand, can always exactly fit any given past record of I(t) and O(t) (at least in theory) even though no two admittances, derived after the manner of the preceding paragraphs, will be quite the same. However, in many cases, by using appropriate values of

only the two time constants T_d and T_s, flood outflows may be fitted or predicted with ease and with adequate precision; in addition, these constants may always be estimated (lacking past records) on the basis of simple physical reasoning as indicated on Fig. 18.

Real non-linear effects do exist, however, just as in the unit hydrograph method. The two most significant of these seem to be: (1) a seasonal effect, and (2) an order of magnitude or flood-size effect which depends on both inflow and outflow magnitudes. These effects may be handled in this procedure, as with the unit hydrograph, by constructing a family of admittance functions, all of which resemble one another qualitatively, but which are quantitively slightly different (i.e. the values of T_d and T_s are different). Sufficient experience with this method is not yet available to estimate the number of such admittances necessary to characterize a channel reach adequately and precisely; however, with an automatic computer technique after the fashion of Figs. 14 and 17, it would always be possible to vary the admittances by trial for best fit with a flood *during its occurrence.*

Acknowledgements

Much credit should be given to the many people whose inspiration and assistance have made this research program possible. Particularly appreciated is the cooperation of Professor A. T. Gifford, colleague of the writer, Dr. A. T. Ippen, Director of the M. I. T. Hydrodynamics Laboratory, and Dr. J. B. Wilbur, Head, Department of Civil and Sanitary Engineering, M. I. T. Grateful acknowledgement is also given to the Division of Scientific Grants-in-Aid of the Research Corporation of New York for providing funds to acquire the electronic analog computer which has become a valued item of equipment in the Hydrodynamics Laboratory.

References and Bibliography

1. Rich, G. R. *Hydraulic Transients,* Engineering Societies Monographs, McGraw-Hill, New York, 1951.
2. Strowger, E. B. and Kerr, S. L. Speed changes of hydraulic turbines for sudden changes of load. *ASME Transactions,* pp. 209-232, v. 48, 1926.
3. Concordia, Crary, and Parker. Effect of prime-mover speed governor characteristics on power system frequency variations and tie-line power swings. *AIEE Transactions,* v. 60, 1941.
4. Rudenberg, R. The frequencies of natural power oscillations in interconnected generating and distribution systems. *AIEE Transactions,* v. 52, 1943.

5. Davis, A. The field of system governing. *ASME Transactions*, pp. 207-220, v. 62, 1940.
6. Evangelisti, G. *La Regolazione delle Turbine Idrauliche*. Zanichelli, Bologna, 1947.
7. Gaden, D. *Contribution a l'etude des regulateurs de vitesse. Considerations sur le probleme de la stabilite*. Editions la Concorde, Lausanne, 1945.
8. Almeras, P. Influence de l'inertie de l'eau sur la stabilite d'un groupe hydroelectrique. *La Houille Blanche*, 1945/46, p. 81, 131, 189, 407.
9. Cuenod, M. Influence des phenomenes de coup de belier sur le reglage de la vitesse des turbines hydrauliques. *La Houille Blanche*, Mars/Avril, 1949.
10. Paynter, H. M. *Stability of Surge Tanks*, S. M. Thesis, M. I. T., 1949.
11. Paynter, H. M. *Transient Analysis of Certain Nonlinear Systems in Hydroelectric Plants*, Sc. D. Thesis, M. I. T., 1951.
12. Paynter, H. M. Electrical analogies and electronic computers for surge and water hammer problems. (ASCE paper in process) (Presented at the Jackson Meeting of the ASCE Hydraulics Division, November, 1950.
13. Paynter, H. M. Optimum governor settings in hydro-steam interconnections. (Paper in preparation for ASME).
14. Gifford, A. T. and Paynter, H. M. *Regulation of Hydroelectric Plants*. (Mimeograph notes) 1952.
15. Gifford, A. T. and Paynter, H. M. New relationships for the analysis of surge tank transients. *Proceedings of the M. I. T. Hydrodynamics Symposium*, June, 1951.
16. Milne, W. E. *Damped Vibrations*. University of Oregon Pubs., v. 2, n. 2, August, 1923.
17. Milne, W. E. *Tables of Damped Vibrations*. University of Oregon Pubs., Math. Series, v. 1, n. 1, March, 1929.
18. Milne, W. E. *Tables of Derivatives for Damped Vibrations*. Oregon State Monographs, Math. and Statistics Series, n. 1, December, 1935.
19. Sorensen, K. E. Graphical solution of hydraulic problems. *ASCE Proceedings*, Separate No. 116, v. 78, 1952.
20. Schnyder, O. Druckstösse in Pumpensteigleitungen. *Schweizerische Bauzeitung*, v. 94, n 22 and 23, 1929.
21. Angus, R. W. Waterhammer in pipes; Graphical treatment. *Inst. of Mech. Eng. Proceedings*, v. 136, 1937.
22. Bergeron, L. Methode graphique generale de calcul des propagations d'ondes planes. *Memoires de la Soc. des Ing. Civils*, Juli/Aug., 1937.
23. Bergeron, L. *Du Coup de Belier en Hydraulique au Coup de Foudre en Electricité*. Dunod, Paris, 1950.
24. Paynter, H. M. *Graphical Solutions of Engineering Transients*. (Book in process).
25. von Karman, T. and Biot, M. *Mathematical Methods in Engineering*. McGraw-Hill, New York, 1940.
26. Gardner, M. F. and Barnes, J. L. *Transients in Linear Systems*. Wiley, New York, 1942.
27. Churchill, R. V. *Modern Operational Methods in Engineering*. McGraw-Hill, New York, 1944.

28. Jaeger, J. C. *An Introduction to the Laplace Transform.* Methuen, London, 1949.
29. McCarthy, G. T. *The Unit Hydrograph and Flood Routing.* U. S. Engineer Office, Providence, R. I., 1938.
30. Tatum, F. E. *A Simplified Method of Routing Flood Flows Through Natural Valley Storage.* U. S. Engineers Office, Rock Island, Ill., 1940.
31. Linsley, Kohler, and Paulhus. *Applied Hydrology.* McGraw-Hill, New York. 1949.
32. Philbrick Researches, Inc. *Catalog and Manual.* 230 Congress Street, Boston, Mass.
33. U. S. Engineers. *Engineering Construction-Flood Control.* Engineer's School, Ft. Belvoir, Va., 1940.
34. Linsley, Foskett, and Kohler. Electronic device speeds flood routing. *Engineering News Record,* pp. 64-66, v. 141, n 26, 1948.
35. Glover, Hebert, and Daum. Application of an analog computer to the hydraulic problems of the Sacramento-San Joaquin Delta in California. (Paper presented at the ASCE Hydraulics Division meeting, Jackson, Mississippi, Nov., 1-3, 1950).
36. Clark, C. O. Storage and the unit hydrograph (p. 1429). *ASCE Transactions,* v. 110, 1945.

IMPORTANCE OF THE NET LOAD TO THE SETTLEMENT OF BUILDINGS IN BOSTON

By Harl P. Aldrich, Jr.*

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on November 14, 1951.)

Soil engineers have realized for many years that the net building load, defined herein as the sum of dead and live load minus the weight of excavated soil, and the history of a clay stratum are important factors affecting the magnitude and rate of settlement due to consolidation of the clay. It is the purpose of this paper to describe a unique investigation (1)** of the foundation clay at the site of the Charles Hayden Memorial Library at the Massachusetts Institute of Technology and to emphasize once again the importance of these factors. Data and results from the Library investigation will be used to correlate settlement curves of a group of buildings in the greater Boston area with the net stress change below the buildings in a zone, which will be shown to be particularly critical, near the bottom of Boston blue clay.

General

Boston and its outlying suburbs are well known for their difficult foundation problems. The sedimentary blue clay which underlies most of the region averages 70 or 80 feet in thickness and is partly responsible for Boston's relatively low skyline. Fortunately, the blue clay usually has a stiff upper crust and sometimes is overlain by a layer of dense sand and gravel. A majority of the buildings in Boston are founded on these good bearing materials.

Various types of foundations and construction practices have been used successfully throughout the area to limit the settlement and, especially, the differential settlement. These designs depend, of course, on the exact soil conditions and the type of structure. Many of the older buildings are founded on wood piles, generally piles bearing on the dense sand and gravel or stiff upper clay but occasionally friction piles driven into the soft clay. Where foundation conditions permit,

*Assistant Professor of Soil Mechanics, Massachusetts Institute of Technology, Cambridge, Mass.

**Numbers in parentheses refer to the Bibliography.

spread footings are used. Concrete caisson piles, of the type originated by the late Colonel Charles R. Gow for the Boston area, have met with a great deal of success for all but the heaviest buildings. When building loads are exceptionally large one alternative is to bypass all compressible soil with long piles to ledge or hardpan. The new 26 story John Hancock Mutual Life Insurance Company building, the tallest structure in Boston, furnishes one of the most notable examples of this approach with its steel H-piles that are over 100 feet long (2).

Settlement of a building founded above a thick compressible soil may be attributed to at least three sources. Settlement may occur as a result of shear strains, especially in the soil immediately below individual caissons, piles or spread footings. The second source involves a volumetric strain, resulting from consolidation, in these same areas of high pressures. Finally, a large part of the settlement may occur because of consolidation throughout the remaining deeper portion of the compressible stratum. While settlement from the first source and to a somewhat lesser degree from the second source occurs soon after building loads are applied, that from the third source may require many years or even decades to take place. Settlement from the first two sources depends on concentrations of load from individual footings, caissons or piles while deep consolidation is more a function of the average net building load. Nearly all structures with foundations that rest above Boston blue clay, regardless of the foundation type, are subject to some settlement as a result of consolidation deep in the clay. Although caissons, spread footings or piles may have been designed for adequately safe bearing intensities as given by the building codes, settlement that may be excessive still can occur.

An outstanding construction procedure used to limit settlement from deep consolidation is reported by A. Casagrande and R. E. Fadum (3) for the foundations of the New England Mutual and Liberty Mutual Life Insurance Company buildings. The basements of these two buildings have been designed so that net loads are nearly zero. This procedure demonstrates the principle that if an excavation is made and if a building with a weight equal to that of the soil removed is immediately placed in the hole, the underlying compressible soil does not recognize any appreciable stress change. Actually, the soil swells a small amount during and after the excavation period but recompression usually occurs early during the construction period. Even though

these heavy buildings are founded above 70 feet of Boston blue clay, their settlements have been relatively small.

In order to predict the ultimate settlement and time-settlement curve for a building founded above a thick stratum of compressible soil, data from three rather distinct sources are required.

1. Exploratory borings are required to determine the thickness of the compressible strata and their depths relative to ground surface. They also disclose soil conditions above and below the clay which are important in that they affect the speed at which settlement takes place.
2. Laboratory consolidation tests on undisturbed samples of the clay are needed to determine the stress-strain and strain-time relationships of the soil. In addition, the consolidation test is useful for estimating the maximum pressure to which the clay has been loaded during its history.
3. Knowledge of the building loads and their distribution is required along with an estimate of the excavation load in order that net stresses transferred to the clay may be evaluated.

From these three types of data the soil engineer is able to estimate by means of well known theories the settlement characteristics of the building and the magnitude of the *excess* porewater pressure caused in the clay by the building loads.

Long before the advent of modern soil mechanics, many foundation engineers had recognized the value of checking predicted settlements by observing actual settlements. Settlement observation points have occasionally been placed in basements of buildings for this purpose. Not until recently, however, have water pressure measuring devices, called piezometers, been installed in the foundation soil of a building for the purpose of checking theoretical concepts.

Research at the Charles Hayden Memorial Library

Before construction was started at the Hayden Library on April 7, 1948, the opportunity for a study of foundation stresses and settlements was recognized. At the recommendation of D. W. Taylor of M. I. T. and with the cooperation of W. H. Mueser of the firm Moran, Proctor, Freeman and Mueser, 32 settlement observation points were placed in the basement and 10 piezometers were installed in and just

above and below the 90 foot layer of Boston blue clay which underlies the building. These installations as well as the settlement observations and piezometer readings will be described following a discussion of other data used for this investigation. These brief descriptions will be followed by a summary of the more important analyses which pertain specifically to the subject of this paper.

Exploratory Borings: Numerous "dry sample" borings were made at the Hayden Library site. The average soil conditions disclosed by these borings are shown in Figure 1. This soil profile is typical of that found in many parts of the Boston area. The coarse sand and gravel stratum below the Hayden Library varied in thickness

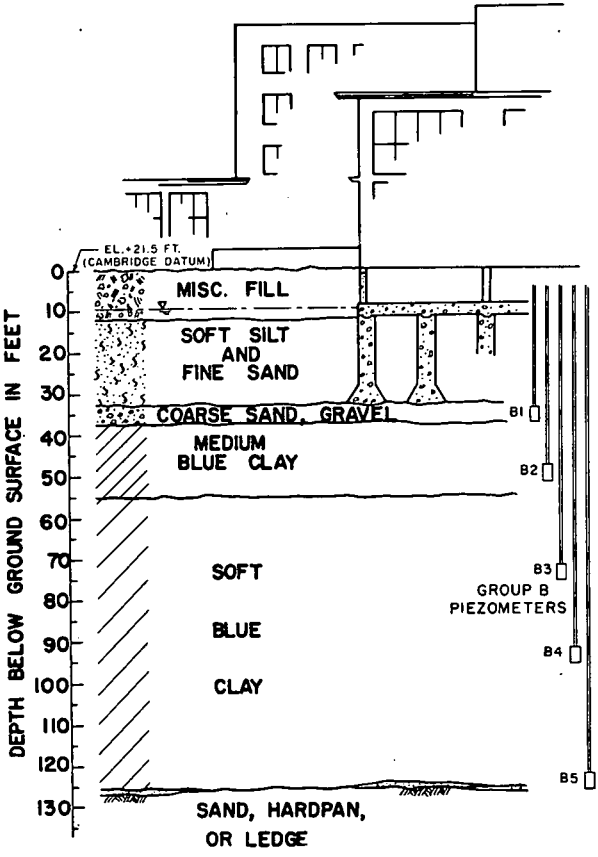


FIG. 1.—AVERAGE SOIL CONDITIONS AT CHARLES HAYDEN MEMORIAL LIBRARY.

from zero to over 6 feet by actual measurement just before each caisson was poured. The average thickness was only about 2 feet.

Laboratory Consolidation Tests: In one special boring, called Boring No. 11, a 3-inch fixed-piston sampler was used. Twelve laboratory consolidation tests, run in the standard manner of doubling the applied load at 24 hour intervals, were performed by W. Enkeboll (4) on undisturbed samples from this boring. These tests clearly indicate that the soft blue clay shown in Figure 1 is considerably more compressible than the overlying medium clay. The average compression index, C_c,* of the medium clay is 0.06 while that of the soft clay is 0.20. Unconfined compression tests showed that while the upper clay had an unconfined compressive strength of about 1.5 tons per sq ft, this strength decreased with depth to a value of 1.0 at the bottom of the clay. Values of the coefficient of consolidation and the maximum pressure to which the clay had been consolidated in its history are discussed in later sections.

Building Loads: The Hayden Library is a steel frame building designed by Voorhees, Walker, Foley and Smith of New York. The firm of Moran, Proctor, Freeman and Mueser acted as consulting foundation engineers and Thompson-Starrett Co., Inc. was awarded the construction contract.

The building is supported for the most part by Gow Caissons which transmit the building loads to the coarse sand and gravel above the clay. Superstructure loads are carried by steel columns directly to the tops of caissons except in one particularly heavily loaded area where a 3½ foot reinforced concrete slab distributes each column load to several caissons. Figure 2, which is a plan view of the Library, shows the location of this slab.

After an average excavation of 9.5 feet, the site was dewatered by wellpoints to facilitate work on the caissons. As the building was constructed actual loads were computed and recorded continuously by H. deR. Gibbons (5). The diameters of the 152 caisson bells were designed so as not to exceed 2 tons per sq ft on the medium blue clay. Actually, the intensity of pressure on the medium clay below the caisson bells does not exceed 1.5 while the average is about 1.1 tons per sq ft.

---

*The compression index is equal to $\frac{de}{d(\log_1 p)}$ where e is the void ratio of the clay and p the intergranular pressure. Cc is simply the slope of the commonly used void ratio vs pressure (log scale) curve. The value quoted above for the medium clay, for example, is the average slope at the mid-stratum overburden intergranular pressure of 1.4 tons per sq ft.

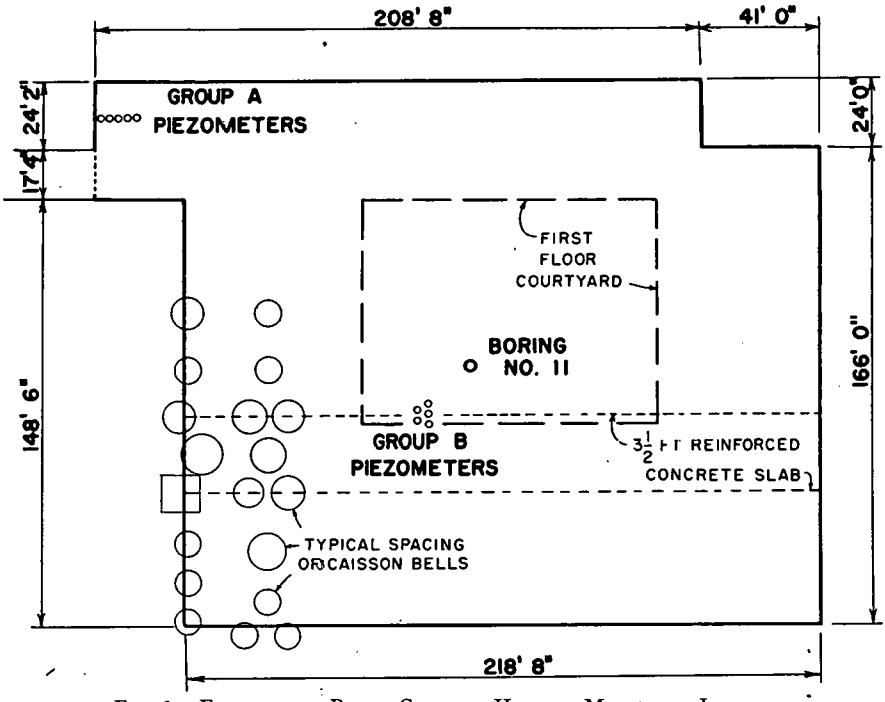


FIG. 2.—FOUNDATION PLAN, CHARLES HAYDEN MEMORIAL LIBRARY.

The average net stress over the entire building area is plotted against construction time in Figure 3. As of early 1951, this stress was only 0.14 tons per sq ft at the surface of the clay. While the weight of the Library was found to be nearly equal to the design dead loads, the live load including book stacks is less than 20 per cent of the design value. In spite of the heavy book stacks the actual live floor load averages only 15 lb per sq ft. The fact that the total live load is only 2 or 3 per cent of the dead weight of the Library may indicate that live loads are hardly worth including in the settlement analysis for many buildings.

Settlement Observations: Thirty-two settlement observation points were placed throughout the basement of the Library soon after erection of the steel was started. These observation points are steel bolts which have been welded to the interior columns or cast into the exterior concrete walls. Elevations of the top of a brass pin, inserted in a drilled hole in the center of the bolt, are determined by means of

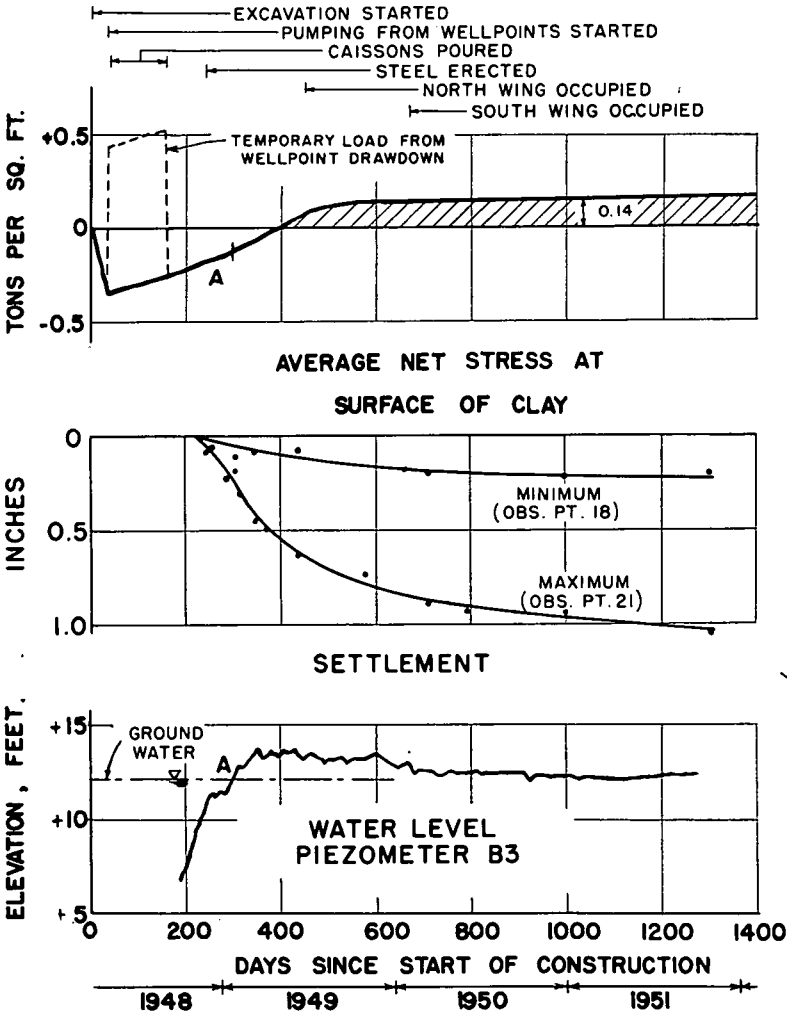


FIG. 3.—TIME CURVES, CHARLES HAYDEN MEMORIAL LIBRARY.

a precision water leveling device. The instrument, built for this project, can be used to close a circuit of ten observation points to within 0.02 inches.

Settlement of the Hayden Library, from the time the observation points were installed to November 3, 1951, is shown in Figure 4. Usually, dish shaped or concave patterns of settlement are associated

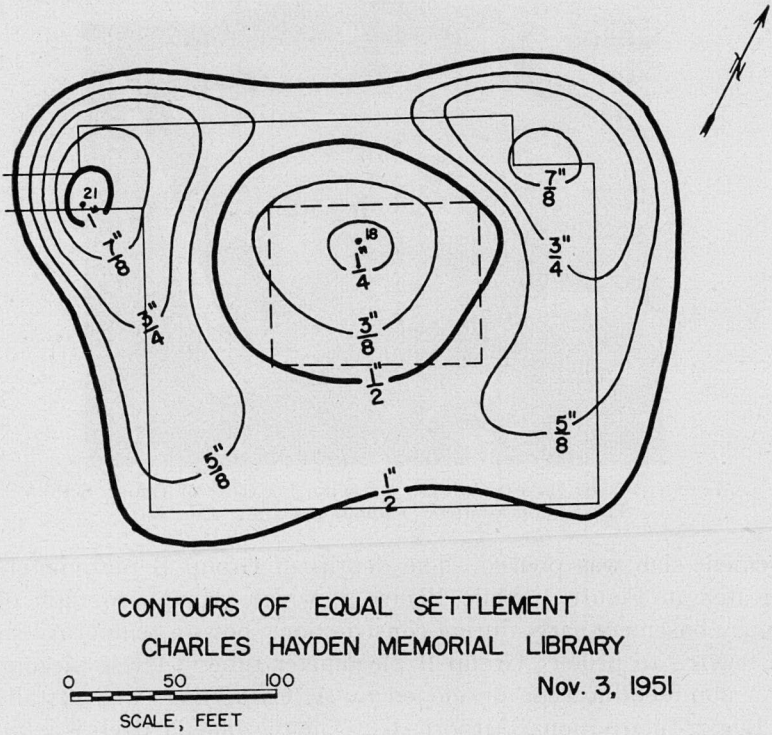


FIG. 4.

with structures founded on thick deposits of compressible soil. The opposite or convex pattern at this building may be explained in part by the fact that unit pressures below exterior caissons are considerably larger than below interior bells and, as will be explained later, most of the settlement may be attributed to volumetric and shearing strains in zones directly below individual caissons rather than to consolidation deep in the clay.

Differential settlements are not as severe as they first appear from Figure 4 since two-thirds of the settlement occurred during construction when some differential settlement can be tolerated without excessive damage. Maximum and minimum time-settlement curves shown in Figure 3 illustrate the speed at which the settlement took place.

Piezometer Installation: Ten piezometers were installed in two groups of five each, as shown in plan in Figure 2, shortly after the

FIG. 5.—FOUNDATION OF HAYDEN LIBRARY SHOWING LOCATION OF GROUP B PIEZOMETERS. (Photograph courtesy Thompson-Starrett Co., Inc.)

basement slab was poured. The depths of Group B piezometers are indicated in Figure 1 while Figure 5, which is a photograph of the Library basement early during construction, shows a temporary shelter constructed to protect Group B piezometer tubes. These piezometers are a non-metallic type developed by A. Casagrande and installed at the Logan International Airport (6). The piezometer itself is a Norton porous tube surrounded by a pocket of sand. Water pressures are determined by measuring the water level in a small diameter plastic tube which extends from the piezometer to basement level. This type of piezometer has proved to be entirely satisfactory for this investigation.

A summary of the water level readings in Group B piezometers is presented in Figure 6. When the first readings were taken on November 23, 1948, 230 days after the start of construction, the excess porewater pressures in the clay were negative as a result of pressure release from excavation. A negative excess pressure at any depth is shown in Figure 6 by a distance to the left of the dashed curve which represents an assumed zero excess pressure. Positive excess pressures plot to the right of the dashed curve and indicate that the clay is consolidating. These curves clearly point out two important items relative to the clay at the Hayden Library site:

1. The total head (pressure head plus head due to elevation) is

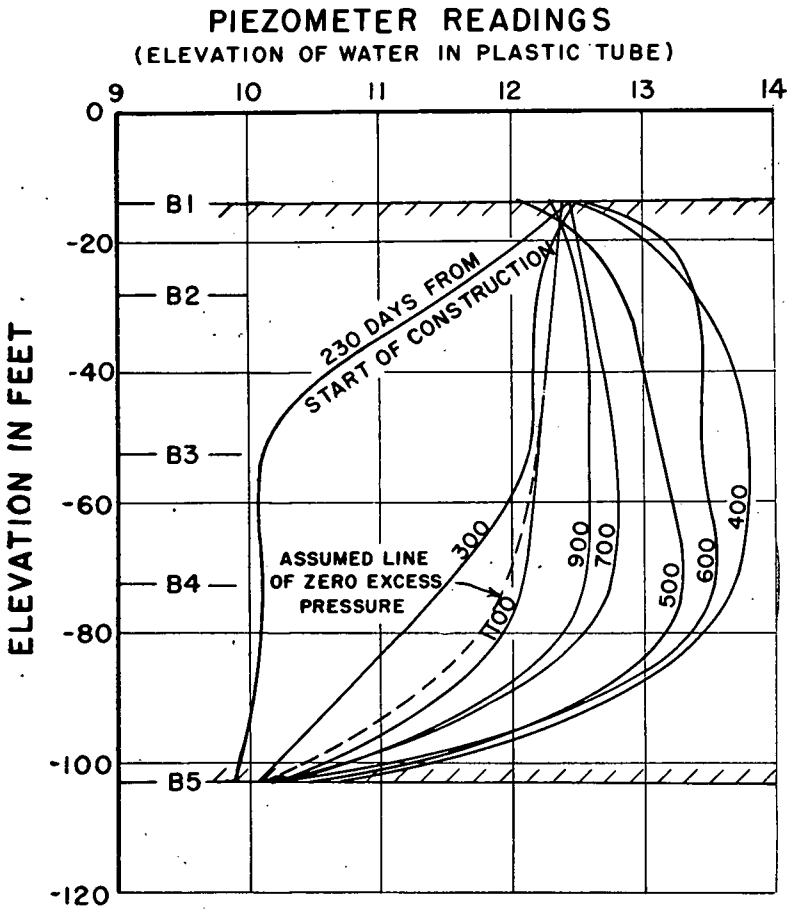


FIG. 6.—SUMMARY OF GROUP B PIEZOMETER READINGS.

approximately 2 feet smaller at the bottom of the clay than it is at the top. This may be thought of as a negative artesian condition.

2. Drainage is somewhat restricted at the bottom of the clay. This is shown by the small but definite response of piezometer B5 to building loads as they were applied.

A plot of water level vs construction time for piezometer B3 is shown in Figure 3.

Analysis of the Data: Many complex factors enter into the analysis and correlation of the data obtained at the Charles Hayden Memorial Library. Insofar as possible, a summary of analyses pertaining to the magnitude of the settlement will be discussed first, followed by considerations of the rate at which the subsidence occurred.

It is difficult if not impossible to ascertain the true source of the small settlement which has occurred at the Hayden Library. Unfortunately no settlement observation points could be placed within the clay stratum. Therefore, actual settlement as a result of consolidation deep in the clay cannot be separated from settlement due to compression of the sand and gravel stratum and shear strains in the clay immediately below the caissons. It is the author's opinion, after making analyses of various types, that a major part of the settlement is a result of compression in a zone within 10 feet of the bottom of the caissons.

An early settlement analysis (7) of the Hayden Library indicated that the building would settle a maximum of 2½ inches from consolidation throughout the clay only. This estimate was based on anticipated net foundation stresses, which were considerably larger than the actual, and compression indices of the clay given by slopes of the laboratory consolidation curves. Later analyses (1), based on laboratory compression indices and the actual net foundation stresses, indicated that settlements would be of the order of ½ to 1 inch from consolidation of the clay only. While the latter range of predicted settlement is exactly that which has occurred, the predicted settlement contours over the foundation are quite different from the actual contours shown in Figure 4. The maximum settlement was predicted for the heavy slab area whereas the figure shows a maximum settlement at the northwest corner. This discrepancy between the predicted and the actual settlement pattern is perhaps the most obvious reason for suspecting that the settlement is *not* primarily due to consolidation throughout the deep clay stratum.

An investigation of the relationship between caisson pressures and corresponding settlements demonstrates that a majority of the Library settlement resulted from strains directly below individual caissons. The settlement of an isolated footing on a highly cohesive soil is, according to theory, proportional to the intensity of pressure if that intensity is small compared to the ultimate bearing capacity of the clay. Similarly, other things being equal, the settlement is

approximately proportional to the diameter of the loaded area. In general then

$$\rho \sim qd$$

Notations and units used in this investigation are as follows:

- ρ settlement of caisson in inches
- q intensity of pressure at surface of clay based on loads added since the start of settlement observations, in kips per sq ft
- d diameter of loaded area at surface of clay based on diameter of caisson bells and an assumed load spread of 30 degrees to the vertical, in feet

Figure 7 gives a plot of qd vs ρ for caissons at the Library for which settlements have been observed. The points in this figure are reasonably well represented by the expression

$$\rho = 0.030\ qd$$

However, the figure shows that the line best representing the data passes below the origin and has the equation

$$\rho = 0.1 + 0.025\ qd$$

The deviation of the actual settlement from that given by this equation is not more than 35 per cent except for three caissons which are located near fill placed for a terrace west of the building. In all probability the fill has contributed to the settlement of these caissons.

It was anticipated that the in-situ compression index of the clay stratum would be considerably smaller than the values obtained from laboratory consolidation tests. Meager data are available from the Library installations to compute the average compression index in situ by a method, called the "pore pressure-area" method by J. P. Gould (8), which makes use of piezometer readings and settlement observations. The distance between any two piezometer curves at a given depth in Figure 6 represents a transfer of stress from water to the soil skeleton if no loads are applied during the period. The area between the curves divided by the height of the plot can be converted, then, to an average increase in intergranular pressure, p, for the period. Settlement which takes place during the period represents an average void ratio decrease, Δe, which may be readily computed if the thickness and void ratio of the clay are known. The average in-situ compression index may then be determined from the formula

$$C_c = \frac{\Delta e}{\Delta(\log_{10} p)}$$

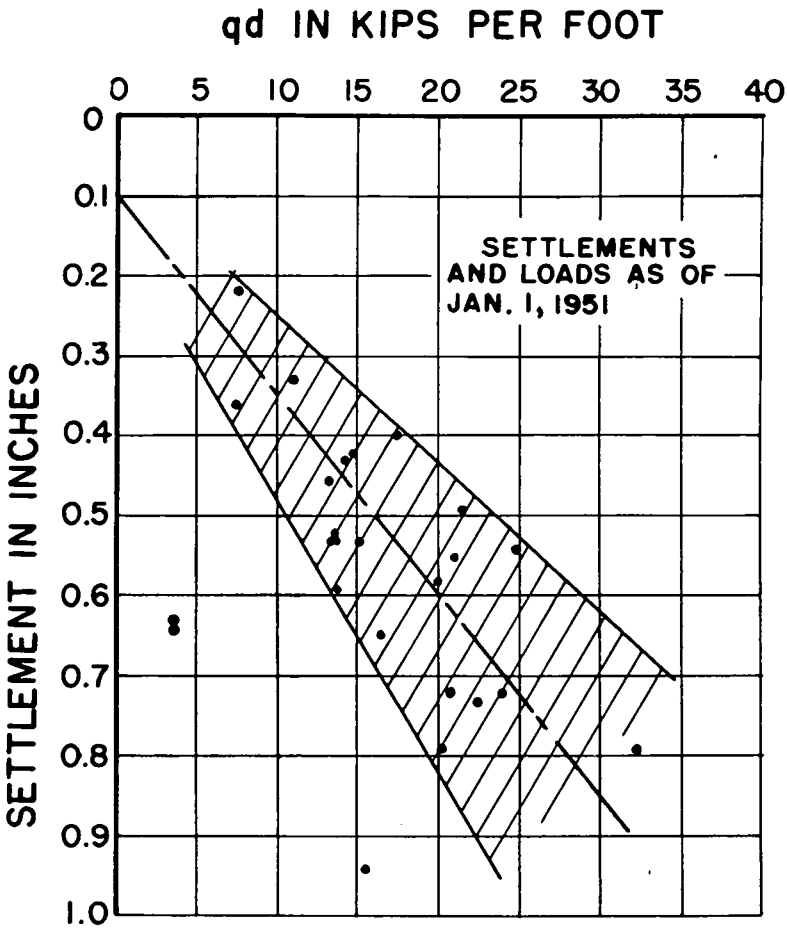


FIG. 7.—RELATIONSHIP OF THE SETTLEMENT AND THE PRODUCT OF THE PRESSURE INTENSITY AND DIAMETER OF THE LOADED AREA FOR INDIVIDUAL CAISSONS, CHARLES HAYDEN MEMORIAL LIBRARY.

Computations of this type for the period following construction give compression indices of the order of 0.02 for the medium clay and 0.06 for the deeper soft clay.* These values are only one-third of those given by the consolidation tests. Nevertheless, they were determined on the assumption that the small settlement which occurred during

*It was assumed for this analysis that the ratio of the compression index of the medium clay to that of the soft clay in situ was the same as the ratio determined from laboratory tests of about 1:3.

this period is due entirely to consolidation throughout the clay. The author's studies lead him to believe that these values of C_c are perhaps twice as large as the in-situ values.

Predictions of the speed at which consolidation and subsequent settlement will occur are perhaps more questionable than predictions of the magnitude of the settlement when net building loads are small. The speed of swell and consolidation at the Hayden Library may be shown qualitatively by two examples: (1) At a time of 300 days after the start of construction the *excess* porewater pressure in the clay was nearly zero after having been negative as a result of pressure release due to the excavation. At this time, represented by point A in Figure 3, the average net stress at the surface of the clay was still negative and equal to —0.13 tons per sq ft, whereas a net stress of zero would be associated with zero excess pressure if swell and consolidation were infinitely slow. In spite of the tendency of the wellpoint drawdown to lower the piezometer readings, then, the piezometers recovered quickly to their steady state readings long before the net load was zero; (2) From 300 days, point A, to the end of the construction an average load of 0.27 tons per sq ft was applied to the surface of the clay. According to the Westergaard stress transmission theory this load would cause an increase in pressure of 0.20 tons per sq ft at the location of piezometer B3. If the load were applied suddenly, or if consolidation were infinitely slow, the porewater pressure in the clay at B3 would increase by about 0.20 tons per sq ft. The water level in the piezometer tube would respond to this pressure increase by rising 6.5 feet. The actual water level rise in piezometer B3 from point A to the end of construction was only one quarter of this or less than 2 feet. It would appear then that dissipation of the excess porewater pressure in the clay occurred just about as rapidly as the load was applied.

The coefficient of consolidation, c_v, as determined from laboratory tests, is a measure of the rate at which consolidation takes place if the flow of water is one dimensional. While consolidation at the Hayden Library was undoubtedlly three dimensional, radial flow was not studied in detail because of complex loading patterns and the fact that excess porewater pressures were nearly zero at the end of construction. Nevertheless, in order to obtain a measure of the average rate at which pore pressure dissipation occurred, a one dimensional capillary flow consolidation analogy model was built by A. C. Rigas

(9) and used by the author. Let c_m be defined as the coefficient of consolidation which, according to the model, is required to give the same speed of *average* pore pressure dissipation as occurred at the Library. The average c_v from consolidation tests was 0.0045 sq cm per sec, whereas c_m from model studies was between 15 and 18 times this value. In other words, pore pressure dissipation and consequently settlement from consolidation occurred 15 to 18 times as fast as would be predicted from the results of consolidation tests. A considerable portion of this discrepancy, without question, may be accounted for by lateral flow of water in the foundation clay.

In summary, this phase of investigation at the Hayden Library has shown that consolidation tests are not likely to give reliable data which can be used to predict time-settlement curves for buildings with small net loads. Furthermore, settlement from consolidation is likely to be very small when net loads are small and whatever settlement does occur often takes place during construction. These conclusions should not surprise soil engineers, but should challenge them to provide answers to at least two important questions: (1) Why cannot consolidation tests be used to predict the settlement of structures with small net loads?; and (2) How large can the net building load be before settlement from consolidation becomes excessive? Answers to these questions for buildings on Boston blue clay can best be approached by first considering the relationships between the net building load and the pressure history of the clay.

Relationship of the Net Building Load and the Pressure History of the Clay

The relationship of the net building load and the pressure history of Boston blue clay and the importance of this relationship to building settlements will be illustrated with the aid of Figures 8 and 9.

The first objective in this discussion will be to compare the slope of the laboratory compression curve with the slope of a curve which represents in-situ compression from building loads. Figure 8 shows the results of a laboratory consolidation test on an undisturbed sample of blue clay from a depth of 72.6 feet below the Hayden Library. This curve is simply a stress-strain curve with stress plotted to a logarithmic scale and with strain, measured by changes in void ratio, plotted on a linear scale. The maximum pressure to which this sample has been consolidated in its history may be estimated by an empirical graphical

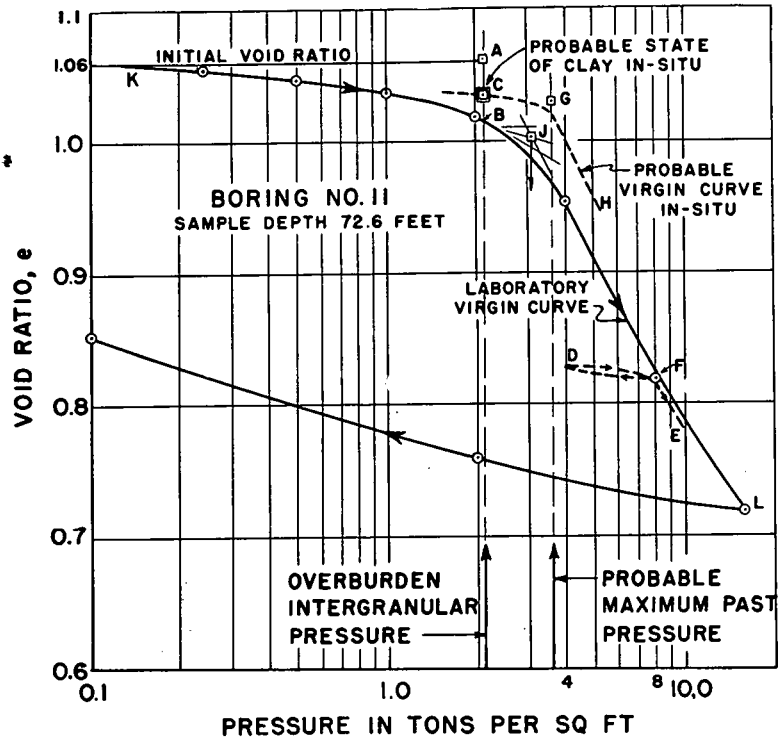


FIG. 8.—RELATIONSHIP OF LABORATORY COMPRESSION AND COMPRESSION IN SITU, CHARLES HAYDEN MEMORIAL LIBRARY.

construction proposed by A. Casagrande (10). This construction is based on the similarity in shapes between the laboratory loading curve KL and the curve DE. Curve DE represents the reloading curve which would have occurred if the sample in the laboratory, after having been consolidated to point F, had been allowed to swell to point D and then reloaded. Point F would represent the maximum pressure to which the sample was consolidated before this hypothetical rebound and reloading. Its relationship to curve DE is similar to the relationship of point J to the actual laboratory curve KL. A vertical line through point J, therefore, is a reasonable approximation, based on the laboratory curve, of the maximum pressure to which the clay was consolidated before it was tested in the laboratory. Points of maximum past pressure, determined by the graphical construction from consolidation tests on samples from Boring No. 11 at the Hayden Library, have

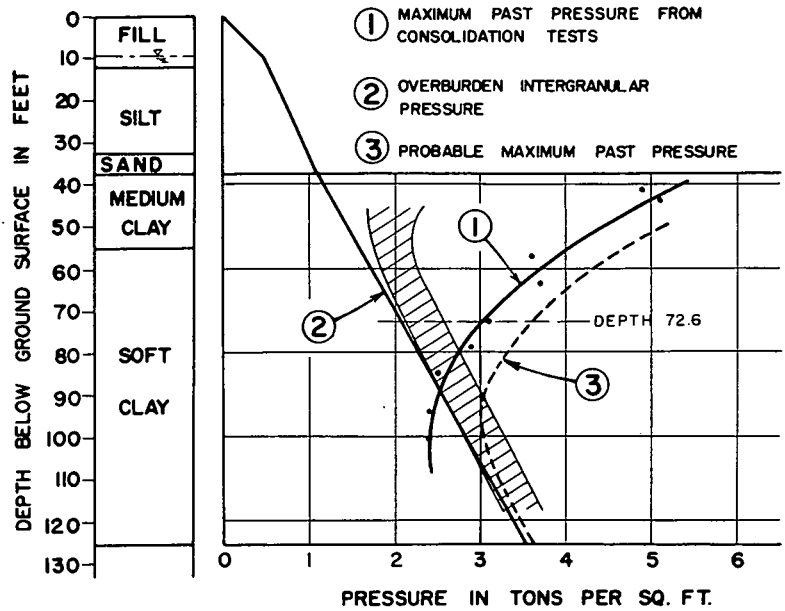


FIG. 9.—PRESSURE VS DEPTH CURVES, CHARLES HAYDEN MEMORIAL LIBRARY.

been plotted in Figure 9 as a function of depth. Curve 1 has been drawn through these points.

Curve 2 in Figure 9 is a plot of present overburden intergranular pressure determined from unit weights of the various soil strata—true unit weight for that portion of the fill above ground water level and submerged weight for all soil below the water table. It may be seen from the relationship of curves 1 and 2 that the upper portion of the clay is highly precompressed. This precompression is generally believed to be a result of surface drying thousands of years ago when the clay was above water.* Furthermore, it appears from the relationship of curves 1 and 2 that the maximum past pressure is smaller than the overburden pressure below a depth of about 90 feet. This relationship is not reasonable unless artesian water pressure exists below this

* I. B. Crosby (11) states that "at the time of the ice retreat the sea stood a little higher than at present, and then the land rose until the sea level was at least 70 feet lower than at present. During this period of lower sea level the clay was eroded and weathered and peat was formed in places. Then the sea level rose, and silt was deposited upon the clay and peat." (The author interprets this quotation to mean that the elevation of the sea remained essentially constant but that the land rose, then subsided after eroding.) Evidence to support the erosion and surface drying as described by Crosby are gullies, some of which are 50 feet deep, and the fact that the medium upper clay is often yellow which is evidence of oxidation.

depth, a condition which many engineers in the past have believed to exist. Piezometers at the Hayden Library have shown, however, that the water pressure is actually slightly smaller than static at the bottom of the clay (see Figure 6). This absence of an artesian condition below Boston blue clay is verified by piezometers at the Logan International Airport (8) and deep borings in other locations. The maximum past pressure at the bottom of the soft clay must therefore be at least as large as the overburden intergranular pressure. It must be concluded then that the maximum past pressures, given by the graphical construction, are smaller than the actual. A possible explanation for this discrepancy is the inevitable disturbance to the natural structure of the clay during and after sampling. At the Hayden Library site the discrepancy was of the order of 0.6 or 0.7 tons per sq ft at the bottom of the clay.

With the knowledge that the maximum past pressure must be greater than the overburden pressure, the author has drawn curve 3, Figure 9, to represent the probable relationship. Curve 3 has been drawn slightly to the right of curve 1 at the bottom of the clay to allow both for the small negative artesian condition and the fact that the author believes Boston blue clay as well as most clay deposits are at least slightly precompressed throughout their entire depth. This small precompression would result at the bottom of the clay, for example, if at any time there had been water pressure fluctuations below the clay or temporary surface loads. In any event, the sample at a depth of 72.6 feet below the Hayden Library has a maximum past pressure of more nearly 3.6 tons per sq ft than 3.1. This probable pressure has been represented by a vertical line at 3.6 tons per sq ft in Figure 8. The present overburden intergranular pressure, 2.1 tons per sq ft at this depth, is also shown.

Point C in Figure 8 has been selected to represent the probable state of the clay at a depth of 72.6 feet below the site of the Library before the boring was made. It has been located below point A and above point B for the following reasons. If the clay sample used for the consolidation test had undergone no volume change during sampling, storage and preparation for testing, the void ratio of the clay in situ would be equal to 1.06, the initial void ratio of the sample in the consolidometer. In this event, point A would represent the initial state of the clay in situ. It is the author's belief, however, that while the net volume change of a sample of Boston blue clay may be nearly

zero, the central portion of the sample swells following sampling and after extrusion from the sample tube as water migrates inward from the remolded outside cylindrical surface. In this event, the initial void ratio of the clay in the laboratory is somewhat greater than the void ratio in situ and point C is therefore *below* point A. In all probability when the sample is loaded in the laboratory to its in-situ pressure it will compress to a slightly smaller void ratio than it had in situ.* Therefore, the void ratio in situ is *above* point B. If point C represents the state of the clay in situ, point G would be the probable state when the clay was precompressed by drying.

At a depth of 72.6 feet swell and compression of the clay, caused by excavation and construction of a building, take place along the relatively flat dashed curve through point C. The laboratory consolidation test, however, shows a considerably steeper slope, that at point B. Therefore, if the compression index represented by this slope was used in a settlement estimate, predicted settlements would be several times larger than the actual as long as the net stress did not exceed the precompression represented by the distance CG. K. Terzaghi and R. B. Peck (12) state the opinion that, for the best of samples, the slope of the laboratory curve is two to five times the field compression curve if the net stress is smaller than about one half the amount the clay is precompressed. In general, the discrepancy in slopes for a given net stress is greatest when the precompression is smallest. The precompression near the bottom of Boston blue clay is at most very small. Therefore, these discrepancies, again probably a result of disturbance to the sample structure, explain why consolidation tests are not likely to give reliable data which can be used to predict the settlement of buildings with small net loads on Boston blue clay.

As long as net stresses transferred to the clay are smaller than the precompression throughout the depth, settlement as a result of consolidation will probably be negligible. However, if the net stress is greater than the precompression CG, Figure 8, the clay will pass by point G during consolidation and enter the steep virgin portion of the curve. Whether or not the curve breaks exactly at point G and is as steep as shown may be open to question. Nevertheless, a situation of this type which is most likely to occur near the bottom of Boston blue

*It is possible that some highly precompressed expansive soils will swell sufficiently following sampling that, after having been recompressed to in-situ pressure, their void ratio will actually be higher than the value in situ. Nevertheless, the author believes that sample disturbance is sufficient to outweigh this possibility for the example drawn.

clay where the precompression is small, may result in large settlements from consolidation. There is no definite answer to the question of how large the net building load, as originally defined, can be before settlement from consolidation becomes excessive. It is the magnitude of the net stress increase near the bottom of the blue clay which probably is most important. This stress depends not only on the net building load but also, among other things, on the size of the building and the depth to the bottom of the clay. Some indication of the critical net stress can be given by comparing the settlement curves of a number of buildings in Boston with their corresponding net stresses at points near the bottom of Boston blue clay.

Settlement Records for Buildings Around Boston

Settlement records of eight buildings in the greater Boston area have been selected for this study. The depth of blue clay below these buildings varies from 55 to 95 feet. Foundation types include piles, Gow Caissons, spread footings and mats and the structures vary in size from 80 feet square to 280 by 460 feet.

It can be shown by a very simple but admittedly approximate demonstration that the net stresses below each of the eight buildings are likely to exceed the precompression in the underlying clay stratum only in the lower 30 or 40 feet. The first step in this demonstration was to compute the net stresses in the clay below points of maximum or near maximum settlement for each of these buildings. These stresses, computed by the Westergaard stress transmission theory, were then plotted to the right (positive net stress) or left (negative) of curve 2 in Figure 9. Before plotting these points, which fall within the shaded area, the mid-depth of the clay stratum below each building was placed at a common depth of 81 feet in Figure 9. While the absolute values of depth and pressure in Figure 9 obviously do not apply except for the Hayden Library site, the *relative* positions of the shaded area and curves 2 and 3 generally hold for the eight building sites. The shaded area passes to the right of curve 3 only in the lower part of the clay. In other words, only in this zone are net stresses likely to exceed the precompression represented by the distance between curves 2 and 3. Above this zone the net stresses are small compared to the amount the clay is precompressed.

Figure 10 shows curves of maximum or near maximum settlement for each of the eight buildings. The number in parenthesis is the net

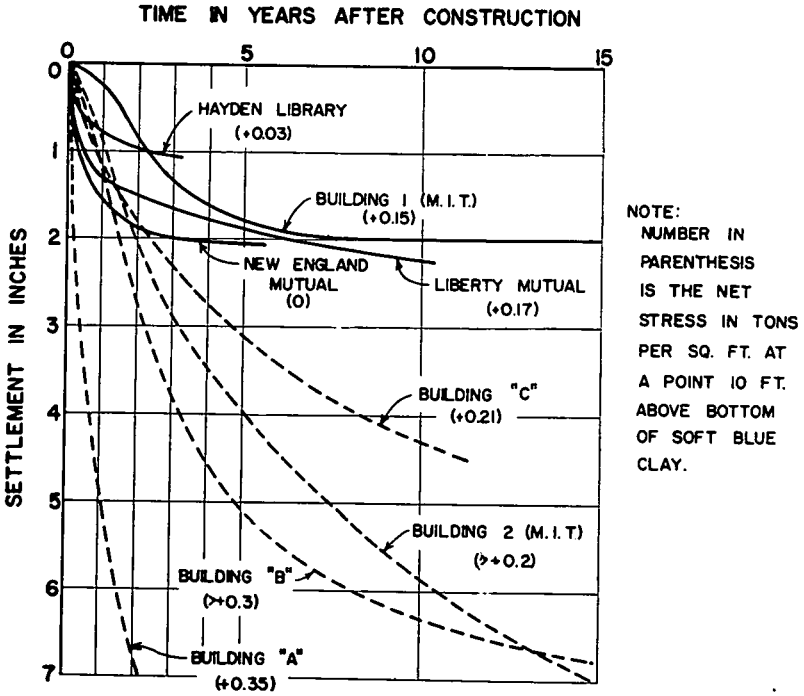


FIG. 10.—TIME-SETTLEMENT CURVES FOR BUILDINGS AROUND BOSTON.

stress transferred to a point arbitrarily taken 10 feet from the bottom of the soft blue clay and directly below that part of the building which has the settlement shown. Data for buildings "A", "C", New England Mutual and Liberty Mutual are from A. Casagrande (2), A. Casagrande and R. E. Fadum (3), and R. E. Fadum (13). Data for building "B" and Buildings 1 and 2 at M.I.T. are from unpublished records kept in the Soil Mechanics Laboratory at M.I.T.

Building "A" is a large structure supported by Gow Caissons and spread footings above a 90 foot stratum of blue clay. The settlement shown in Figure 10 occurred at a heavy tower section. The maximum differential settlement of this structure at the end of 2 years was over 5 inches.

Building "C", a small structure, is founded on Gow Caissons above a 55 foot stratum of blue clay. Settlement observations were not made until 2 years after construction started and the time-settlement curve

is one which Fadum (13) has estimated would apply if movements had been recorded from the start. It is believed that settlement of this building has been affected by a larger structure nearby which was built not long before building "C" was constructed.

The Liberty Mutual building is founded on Gow Caissons which bear on the hard upper crust of a 70 foot blue clay stratum. The New England Mutual building is supported by footings and mats above a similar clay stratum. Both of these structures were purposely designed with deep basements in order to reduce net loads. Excavation for the Liberty Mutual varied from 12.5 to 25 feet while that for the New England Mutual averaged 35 feet. The 2 inch settlement at the New England Mutual site, most of which took place during construction, is largely a result of rapid recompression following swell associated with the deep excavation.

Building "B" is a monumental structure supported by wood piles some of which were driven into and others through a layer of dense sand and gravel overlying an 80 foot layer of blue clay. This building is still settling at a rate of 0.04 inches per year, 35 years after construction.

Building 1, a section of the main M.I.T. building constructed in 1916, is founded on wood piles bearing on a stratum of dense sand and gravel averaging 12 feet in thickness. Building 2, another section, is also supported by wood piles, some of which penetrate through a somewhat thinner stratum of sand and gravel into the underlying clay. While the building weights are similar, the net stresses transferred to the clay are smaller at Building 1 partly because of a thicker sand and gravel stratum located nearer ground surface.* Building 2, which has settled 8.5 inches since 1916, is subsiding at a rate of 0.05 inches per year. The author has not been able to explain satisfactorily why Building 1, on the other hand, has not settled a measurable amount in the last 25 years.

The four buildings which have settlements shown by solid lines in Figure 10, have net stresses of between zero and +0.17 tons per sq ft near the bottom of the soft clay. Settlement of these structures is believed to be mainly due to recompression of the clay after swell

*Prior to 1890 a majority of the M.I.T. site was a tidal flat with ground surface at about El. +7 based on the Cambridge datum (mean sea level is about El. +11). The tip of a gravel point, called Whittimore's Point, projected east across the present Massachusetts Avenue and just into the area now occupied by Building 1. Between 1898 and 1912 a total of 14 feet of fill was placed at the site except for the area near Building 1 where little filling was required. There can be little doubt that this fill, as well as the difference in net stresses, has contributed to the differential settlement between Buildings 1 and 2.

following excavation and to local consolidation and shear strains in areas directly below caissons, pile groups or spread footings. The remaining four buildings, with rather serious settlement magnitudes, have net stresses of between +0.21 and +0.35 tons per sq ft. Settlement of these buildings may be attributed in part, of course, to the sources mentioned above but perhaps far more to the slow consolidation deep in the clay where the precompression has been exceeded. It would appear then, that if the net building stress near the bottom of Boston blue clay exceeds perhaps +0.2 tons per sq ft, settlement of the building will be excessive. This statement should be regarded only as a rough general guide for preliminary studies since each soil profile and building in the Boston area poses its special problems which must be carefully studied before a final foundation design can be adopted.

Summary and Conclusions

1. The advantage of excavating a quantity of soil nearly equal in weight to that of a building has again been demonstrated by the Charles Hayden Memorial Library building at M.I.T. Largely because of its small net load, settlement of the structure has been small and a major part of it occurred during the construction period.

2. Settlement observation points and piezometers which were installed at the Hayden Library site have given data which indicate that consolidation tests are not likely to give reliable information for use in predicting time-settlement curves for buildings with small net loads on Boston blue clay. Consolidation at the Library site occurred along an average compression curve approximately one-fifth to one-tenth as steep as that given by laboratory tests and consolidation took place 15 to 18 times as fast as would be predicted if results of these tests were used.

3. The author has concluded that while the upper part of the 90 foot clay stratum underlying the Hayden Library is highly precompressed, the bottom 20 or 30 feet is, at most, only slightly preconsolidated. This state is believed to be generally true for the blue clay in a large part of the greater Boston area. It has led the author to the conclusion that the net stress increase near the bottom of the clay is perhaps the most important factor affecting the magnitude of the building settlement caused by consolidation. From a preliminary study of the settlement characteristic of eight buildings in the greater Boston area, it appears that if the net stresses in this deep zone exceed

perhaps 0.2 tons per sq ft, settlement of the building is likely to be excessive.

In conclusion, the author wishes to point out that while the settlement point and piezometer installations at the Charles Hayden Memorial Library have given valuable field data relative to building settlements, more and better installations of this type are needed. Only with carefully planned full scale investigations will soil engineers be able to check existing hypotheses and theories used to predict the settlement of buildings founded above a thick compressible soil.

Acknowledgements

The author wishes to acknowledge gratefully the guidance given by Professor D. W. Taylor throughout this investigation. Thompson-Starrett Co., Inc. cooperated by allowing the author free access to the Library premises during construction and by furnishing valuable construction records. Many members of the Soil Mechanics staff at M.I.T., particularly Mr. W. R. Sutherland, assisted the author in taking settlement observations and piezometer readings. Finally, especial thanks to M.I.T. for making this investigation possible.

Bibliography

1. Aldrich, Harl P. Jr., "Analysis of Foundation Stresses and Settlements at the Hayden Library," Mass. Inst. of Technology, Sc.D. Thesis, 1951. Unpublished.
2. Casagrande, A., "The Pile Foundation for the New John Hancock Building in Boston," Journal of the Boston Society of Civil Engineers, Vol. 34, 1947, p. 297.
3. Casagrande, A. and Fadum, R. E., "Application of Soil Mechanics in Designing Building Foundations," Trans. Am. Soc. C. E., 1944, p. 383.
4. Enkeboll, William, "Report on Soil Investigations for the Proposed New Library Building and Nuclear Physics Laboratory at M. I. T." November, 1946. Unpublished.
5. Gibbons, H. deR., "The Loads Effective in Causing Consolidation at the Hayden Library," Mass Inst. of Technology, S.M. Thesis, 1949. Unpublished.
6. Casagrande, A., "Soil Mechanics in the Design and Construction of the Logan Airport." Journal of the Boston Society of Civil Engineers, April, 1949.
7. Grand, B. A., "A Settlement Analysis of the Hayden Library," Mass Inst. of Technology, S.M. Thesis, 1949. Unpublished.
8. Gould, J. P., "Analysis of Pore Pressure and Settlement Observations at Logan International Airport," Harvard Soil Mechanics Series, No. 34, December, 1949.

9. Rigas, A. C., "The Consolidation Analogy Model," Mass. Inst. of Technology, S.B. Thesis, 1951. Unpublished.
10. Casagrande, A., "The Determination of the Pre-Consolidation Load and its Practical Significance," Proc. Int. Conf. Soil Mechanics, Vol. III, Harvard University, 1936.
11. Crosby, I. B., "Evidence From Drumlins Concerning the Glacial History of the Boston Basin," Bulletin of the Geological Society of America, Vol. 45, 1934, p. 135.
12. Terzaghi, K. and Peck, R. B., "Soil Mechanics in Engineering Practice," Wiley, 1948, p. 70.
13. Fadum, R. E., "Observations and Analysis of Building Settlements in Boston," Sc.D. Thesis, Graduate School of Engineering, Harvard University, 1941. Unpublished.

"GROWING PAINS" OF A SEWAGE TREATMENT PLANT

By John R. Szymanski*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on December 5, 1951.)

Beginning the year of 1936 the writer has made a practice of visiting from three to five different sewage treatment plants each year. The problems and difficulties found at most plants appeared to be basically the same, and as the years went by the writer found that the New Britain sewage treatment plant could well serve as a model for this exposition.

Webster defines "Growing pains" as "Neuralgic pains or cramps in the limbs occurring during growth". No doubt, a strictly technical title would probably be more apropos wherein references are made to preliminary surveys, design data and operational data; however, to emphasize some of the following information, it is felt that the title is fitting and appropriate.

From 1910 to 1930 the city of New Britain, Connecticut, tried in vain to treat its sewage by means of intermittent sand filters. In 1931 investigations by Morris Knowles, Inc., and in 1934 investigations by Metcalf & Eddy brought about recommendations that the city of New Britain discard the filter beds and construct an activated sludge plant. Both engineering firms stipulated that industrial wastes be definitely excluded since their presence would interfere with the efficiency of the proposed treatment.

The approximate cost of the proposed activated sludge plant was $613,000 (as of 1934 construction costs) with an annual operational cost figure of approximately $80,000. Due consideration was given to this proposal by the respective authorities, and it was decided to study the problem further on the grounds that no assurance had been given on the effectiveness of the Activated Sludge process in the treatment of industrial wastes.

In 1935 the engineering firm of Fuller & McClintock made an extensive investigation of the existing problem and proposed that the city of New Britain construct a plant employing a bio-chemical process of sewage treatment. In their opinion this method of treatment was

*Superintendent, Sewage Disposal Commission, City of New Britain, Conn.

the only method by which the sewage of New Britain could be treated satisfactorily.

Using flow data accumulated in 1930 and 1931, it was estimated that the daily flow of sewage at the time of the Fuller & McClintock report was between 7 and 8 million gallons per day. With these figures in mind it was estimated that by 1950 the population of New Britain would be 80,000 and that a nine million gallon per day plant would be necessary. Design data followed these estimates, and the plant was constructed.

It is noted that from June through September, 1935, actual gauge readings were taken by the Bureau of Engineering FERA project #88-F2-96 and that the daily sewage flow fluctuated from 6.8 m.g.d. to 9.3 m.g.d.

The nine million gallon per day plant was built and placed in operation in March, 1937. It was assumed that the raw sewage, discounting industrial characteristics, would contain about 250 p.p.m. of suspended solids and that the 5-Day bio-chemical oxygen demand would seldom exceed 200 p.p.m.

It was the opinion of the engineers that a plant of this type would effectively abate the gross nuisance caused in Willow Brook and the Mattabasset River, and that for the amount of treatment needed this plant would prove most economical to operate. It was also agreed that in order to meet local requirements, it would be necessary to treat sewage so as to remove about 90 per cent of the suspended solids and 80 per cent of the 5-Day B.O.D.

The units contained in the treatment plant consisted of the following:

1. Screenings removal by means of bar screens
2. Three primary sedimentation tanks allowing one hour of settling at design flow of 9 m.g.d.
3. Dosing tanks
4. Four aeration tanks equipped with carborundum tubes and providing 1.5 hours of retention at design flow
5. Four final sedimentation tanks giving 2 hours of settling at design flow
6. Facilities for mechanical sludge dewatering and incineration.

Description of Performance Records

During the first few years of operation conditions were normal, and the plant performed satisfactorily. It would entail reams of paper to report daily performances, hence for the sake of comparison and study at this point only yearly averages will be used.

Fiscal Year	*Ave. Max. Flow**	*Ave. Min. Flow***	*Total Treated Flow*
1937	10.2	5.7	7.97
1938	15.5	5.5	9.49
1939	11.3	6.2	8.93
1940	11.3	7.0	9.35
1941	11.4	6.0	8.91
1942	13.6	8.0	10.77
1943	13.1	6.4	10.29
1944	11.7	6.8	9.49
1945	12.5	7.0	9.95
1946	12.6	6.3	9.71
1947	14.2	7.3	11.78

(All figures reported in million gallons per day)

*Figures reported for the average maximum flow are taken from daily chart recordings and correspond to the rate of flow in million gallons per day for a period normally extending from 10:00 a.m. to 4:00 p.m.

**Figures reported for the average minimum flow are taken from daily chart recordings and correspond to the rate of flow in million gallons per day for a period normally extending from 4:00 a.m. to 6:30 a.m.

In standard practice average figures are usually recorded as shown above, however, in order to get a more detailed picture of the existing problem of overload at the treatment plant, the following data are herein reported.

a. Normal dry weather flow period:

Date	*Maximum*	*Minimum*	*Total Treated*
Sept. 22, 1947	14.8	5.7	10.97
23	14.1	5.7	9.60
24	13.0	5.7	9.55
25	12.5	6.0	9.44
26	13.4	5.6	9.42
27	11.2	5.2	9.06
29	14.6	5.2	9.58
30	12.3	5.1	9.17

(All figures are reported in million gallons per day)

b. Normal high water flow period:

Date	*Maximum*	*Minimum*	*Total Treated*
March 14, 1948	15.0	10.1	12.86
15	25.0*	12.3	15.97
16	25.0*	16.7	17.40
17	25.0*	16.7	17.40
18	25.0*	16.5	17.00
19	25.0*	16.6	17.00
20	25.0*	16.3	17.00

(All figures reported in million gallons per day)

*Denotes actual recordings of 17.0 or 17.5 on flow charts plus amount estimated to be approximately 25 m.g.d.

After reviewing data covering the past few years, it was found that the above figures are representative of their respective periods.

c. Original engineering estimate of solid load to be handled by New Britain sewage plant by 1950 was 18,765 lbs. per day.

Actual solids load handled by treatment plant in yearly averages:

Fiscal year	
1937	11,267 lbs. per day
1938	12,445
1939	14,002
1940	14,427
1941	19,468
1942	25,148
1943	21,858
1944	19,569
1945	19,850
1946	16,361
1947	21,319

It is apparent from the above figures that beginning in the year 1941 the plant had already gone beyond its anticipated solids load.

The Bio-chemical oxygen demand (B.O.D.) followed the same trend.

d. Plant treatment:

SUSPENDED SOLIDS

Fiscal Year	Raw	Yearly Average Eff.	% Removed	Ave. Raw-Range in p.p.m. Monthly High	Low
1937	178 p.p.m.	28 p.p.m.	83	688	68
1938	157	21	87	192	134
1939	188	24	87	227	187
1940	185	27	85	226	138
1941	262	28	89	373	194
1942	280	36	87	407	181
1943	251	35	87	344	171
1944	247	38	85	323	142
1945	238	40	83	312	182
1946	202	41	80	224	174
1947	217	39	82	292	180
		5-DAY B.O.D.			
1937	152 p.p.m.	37 p.p.m.	75	293	58
1938	148	33	77	211	105
1939	169	28	83	198	140
1940	172	36	79	215	127
1941	207	40	81	250	147
1942	223	49	78	296	149
1943	195	48	76	247	153
1944	180	47	74	253	139
1945	184	48	74	208	156
1946	182	51	72	189	168
1947	184	48	74	233	171

TOTAL IRONS

Tabulating average figures for total iron determinations would not show to any advantage the problem this factor brings about at the plant. It will suffice to say that no two weeks are alike in the amounts of iron reaching the plant and that spot samples have shown that amounts of 400 p.p.m. of Fe have been present in the raw sewage for periods ranging from 5 minutes to 8 hours. Occurring in such quantities at times when high flows occur, enough time is not available to remove enough of this waste during treatment, hence it reoccurs as a precipitate and settles along the banks of the receiving stream, eventually causing odor nuisances.

Originally the state authorities indicated that an effluent having

30 p.p.m. of suspended solids or less and having 50 p.p.m. of B.O.D. or less would be satisfactory for downstream conditions. Recently, it was brought out by the state authorities that in order to maintain ideal downstream conditions an effluent containing 15 p.p.m. of suspended solids and 15 p.p.m. of B.O.D. would be necessary.

In looking over the above data one can readily see that the anticipated 1950 design capacity was reached the second year the plant was in operation and that there has been a gradual increase with some loss in treatment efficiency.

With the onset of war and conditions resulting from highly geared industrial activities the Sewage Disposal Commission became aware of the fact that, although the sewage plant facilities were performing satisfactorily, the end results were not satisfactory enough when downstream conditions were taken into consideration. It was apparent that the plant was overloaded, and the future did not promise any abatement of this heavy load.

Description of Actions Taken

As a result of this, the Sewage Disposal Commission conferred with the interested state authorities and decided that some definite action must be taken to alleviate existing conditions.

After a conference in 1942 and upon recommendation of the Guggenheim Brothers, it was decided to try to improve the aeration facilities at the plant. This changeover required the removal of the carborundum tubes and the installation of glass cloth media for air diffusion. This transfer was made in May, 1943.

Although some improvement in the efficiency of treatment was noted after changing air diffusion media, the results were still not satisfactory enough, and it was decided to engage a firm of consulting engineers for the purpose of studying the problem and for the purpose of making recommendations which would definitely improve existing conditions.

The engineering firm of Keis & Holroyd was engaged to make an intensive survey of the plant facilities to determine the following:

1. Is the existing plant accomplishing as much as is practicable in the treatment of sewage, and if not, what changes in operation can be made to secure maximum efficiency?
2. What changes or additions to the plant are required to accomplish satisfactory treatment of the sewage now and for the next fifteen to twenty years?

3. Should consideration be given to the possibility of treating industrial wastes now going to the storm water sewer?

A complete and detailed report was received in October, 1945, advocating expansion of the existing treatment works from 9 to 15 m.g.d. plus the addition of other essential and vitally needed units. The estimated cost of this expansion program would be $844,500.00.

	Cost of Operating Existing Plant for 1945	*Keis & Holroyd's Estimated Cost of Operating Proposed Plant for 1945*	*Estimated Operating Cost Under Metcalf & Eddy's Pipe Line Proposal 1945*
Sup'n. and Labor	$39,150.00	$39,150.00*	$39,150.00
Chemicals	14,200.00	21,200.00	14,200.00
Electric Power	11,000.00	32,000.00	11,000.00
Fuel	6,000.00	6,000.00	6,000.00
Maintenance	4,900.00	5,000.00	4,900.00
	$75,250.00	$103,350.00	$75,250.00

*This was corrected from the original $34,100.00 estimated by the engineering firm to an actual cost figure.

Several amended proposals as requested by the Commission to reduce costs were studied, but one factor that was apparent was the fact that irrespective of any extensive alterations at the existing plant a guarantee that the effluent produced would be indefinitely satisfactory to the interested authorities was not forthcoming.

During the course of the above mentioned conferences the fact was brought to the Commission's attention that it may be possible to pipe the present plant effluent directly to the Connecticut River and that under these conditions the present effluent would be satisfactory for discharge into so large a body of water.

In order to have a complete picture on this matter the Sewage Disposal Commission engaged the firm of Metcalf & Eddy to study the possibility of piping the plant effluent directly to the Connecticut River, and in November, 1947, a report was submitted to the Commission.

Metcalf & Eddy reported that such a project was feasible and that the estimated cost of it would be about $1,900,000.00. The estimated operating cost would be approximately $85,800.00. The capacity of conduit would be 45.0 m.g.d.

Description of Comparative Costs

The Engineering News-Record figures show that the Construction and Building Cost Indexes were at 310 in October, 1945, and steadily rose to 429.3 in November, 1947. The differential being 119.3, gives us an increase of 38.5 for the above-mentioned period.

	Keis & Holroyd	*Metcalf & Eddy*
Estimated Cost Adjusted to Nov., 1947	$1,169,633.00	$1,900,000.00
Annual Financial Charges Amortized 20 years at 2% interest at Nov., 1947 estimate	$70,178.00	$114,000.00
Annual Operating Charges estimated as of Nov., 1947	118,130.00	86,110.00
	$188,308.00	$200,110.00

(In studying the above it must be remembered that the Keis & Holroyd plan allows for an increased capacity to 15 million gallons daily, while the Metcalf & Eddy plan provides a conduit capable of handling approximately 45 million gallons per day.)

Let us come back to the "growing pains" of this plant. Like a young lad for whom a size twelve suit was purchased and almost overnight a size sixteen is needed, the New Britain plant shows the shortness of length in the pants' legs and narrowness in the shoulder breadth. Observations and studies on the plant and its operations show that its facilities are deficient in the following respects:

1. Grit removal
2. Primary tank detention capacity
3. Sludge storage capacity
4. Air supply to aerating tanks
5. Adequate return capacity of chemical sludge.

The following design data were used in order to propose improvements and increase operating officiency. The plant's effluent is discharged into Willow Brook which flows into the Mattabasset River near Middletown, Connecticut. The natural flow in the Mattabasset River most of the time is small and affords very little dilution for the plant's effluent. The design capacity was evidently predicated on the assumption that virtually all of the storm water run-off would be

excluded from the sanitary sewer system. Unfortunately this is not the case, and it is not practical economically to undertake the seeking out and disconnecting of all unauthorized and unknown storm water connections from private property, the repair of all leaky joints which permit the infiltration of ground water, the tracing out and disconnecting of all catch basin and drop inlet sewer connections. In general, it has been found to be more expedient to provide treatment facilities for the volume of sewage actually found to exist. In accordance with this conclusion, an allowance of 175 gallons per capita has been used in determining the size for future expansion. The sewer records show that this amount has been realized continuously over periods exceeding one month, and it is therefore believed that provision should be made for handling this volume.

During dry weather periods, the volume of sewage per capita will be less than above, and this will result in more complete treatment when the stream flows are small and a reduction in usage of chemicals and operating costs. This will be advantageous in that during dry weather the amount of diluting water in the Mattabasset is very small, and the plant effluent must be well stabilized to avoid overburdening the stream.

The question as to whether or not industries now discharging process wastes into the storm sewers should be given permission to connect these processes using harmful chemicals to the sanitary sewers has been given much consideration. In general, it is felt that a City has the obligation of maintaining sewerage service for its industries as well as its residents. Expanding industrial activity is a major force to promote greater municipal growth and prosperity. In the case of New Britain, several important industrial plants are now discharging process wastes into the sanitary sewer system while others are connected to the storm water systems discharging into Piper or Willow Brooks. Investigations lead to the conclusion that the general character of the sanitary sewage would not be materially changed if the industrial plants now using the storm sewer system were to be connected to the sanitary sewer system. In view of the fact that on several occasions wastes bearing cyanide salts, in sufficient concentration to injure livestock herds drinking from the streams, have been discharged directly into the storm sewers or Willow and Piper Brooks, it is recommended that plant processes involving the use of similar poisonous chemicals be permitted to discharge these wastes into the sanitary

sewers. Any process water, not noxious in character, nor tending to overburden the streams, should continue in the storm water systems.

The use of the sanitary sewers as outlets for the trade wastes should be regulated by an ordinance designed to provide for the regulation of the discharge of such wastes. The industries affected should be required to dilute strong wastes and discharge them at reasonably slow rates. In some instances, this might require the industry to install a holding tank or vat and supervise the use of the tank to insure the desired result. The realization that the cost of operation of the sewage treatment works is directly affected by the quantity and concentration of industrial wastes and that these costs are certainly reflected in the tax rates applied to the industrial plants should tend to provide better regulation of difficult wastes at their source.

Unforeseen changes in manufacturing processes involving the production of new waste products may, in the future, necessitate adjustments and changes. Such problems cannot be predicted and can only be handled as they occur. The treatment works will permit great flexibility in operation, and it is believed would meet such emergencies with a minimum of adjustment.

The primary objective of sewage treatment is maintenance of a good condition and appearance in the streams receiving the final plant effluent. In the case of New Britain, where the normal flow in the stream is very small, the primary requisite is for an effluent very low in oxygen demand and in suspended dissolved solids which tend to deposit in slowly moving water. In the light of past experiences with the New Britain plant and the Mattabasset River and to meet the requirements of the State Water Commission, it is necessary to produce a plant effluent substantially better than is possible by present facilities. It is the unfortunate geographical location of the City which imposes the duty of providing more complete treatment of its wastes than would be required if a larger stream were available to dilute the final effluent.

In view of the quantity of industrial wastes now received by the sanitary sewers and the possible increase of such wastes in the future, it is doubtful that any of the standard methods of sewage treatment involving biological action only would be suitable for the maintenance of effective and efficient treatment. It is true that the plant now in operation has on occasion been severely shocked by high concentrations of industrial wastes. The plant, however, has recovered from

such shocks in relatively short times, shorter than would have been possible under similar circumstances for activated sludge, high rate or standard rate trickling filter plants.

The bio-chemical process is relatively expensive to operate because of chemicals and electric power costs. This feature is a result, however, of the special requirements for sewage treatment in this location and the difficult character of the sewage and not of the process.

Studies were made of the possibility of using high-rate filters as a buffer or roughing stage ahead of the basic bio-chemical process. As stated above, it is believed that high-rate filters would be affected by the high concentration of industrial wastes to such an extent that continuous satisfactory results could not be assured.

Although all of the surveys, basic data, and operational data have been compiled and are available for use at any moment, we in New Britain are still operating with the plant built in 1936. As in New Britain, a good number of other communities have found themselves in the same position. The actual surveys and proposals are a slow moving procedure which eventually produce end results, however, the time element absorbed in contemplating many of these proposals is so drawn out at times that when a decision to act is made some of the original recommendations have become obsolete.

The foresight of designing engineers and the ingenuity of operating personnel enable many communities to continue operating existing sewage treatment facilities even though they are going through the stages of "growing pains".

SEWER ASSESSMENT PRACTICE AND SEWER SERVICE CHARGES IN MASSACHUSETTS

By Frank L. Flood*, Member

During 1951, we sent out a questionnaire relative to current practice on sewer assessments and sewer rental or service charges to 73 municipalities in Massachusetts. To date, (December 28, 1951) replies have been received from 41. These replies have been tabulated and a copy of the tabulation is attached hereto.

Sewer Assessment Rates. The sewer assessments as received vary from a minimum of $0.50 to a maximum of $7.60 per linear foot of frontage. For comparative purposes, the assessments based on area have been converted to the equivalent cost per linear foot as reported in the tabulation.

In many municipalities, the assessment rate was established years ago and was based upon a percentage of the then prevailing construction costs. In the majority of the municipalities reporting, the assessment rate has not been changed in keeping with increasing costs. In fact, until recently, in most cases it was deemed unlawful to change the assessment rate once it had been established. However, recent legislation has made it possible to re-establish the assessment rate in keeping with the rise in prices.

During recent years, a number of municipalities have increased the sewer assessment charge about in proportion to the increase in construction costs. It is of interest to compare the apparent construction costs in this latter group. This has been done based on the reported percentage of construction costs paid under the general tax levy, the reported equivalent assessment rate per linear foot and the assumption that the assessable frontage is equivalent to 175 percent of the actual length of sewer constructed. The results are as follows:

*Partner, Metcalf & Eddy.

Municipality	*Apparent construction cost of sewers per linear foot*
Braintree	$ 10.20
Lexington	11.00
Needham	10.70
Newton	20.30
Quincy	13.80
Northbridge	17.60
Swampscott	12.00
Wellesley	12.22
Winchester	15.66

Omitting Winchester, Northbridge, and Newton, where the estimated costs, as above derived, are apparently somewhat high, the estimated average cost of sewers is about $12 per linear foot. It is reported that the actual average cost of all sewers constructed in Quincy during the five years prior to 1951 was $11.83 per foot.

The reported median equivalent sewer assessment rate is about $2 per foot. This represents only about 29 percent of construction costs at the present time .

Portion of Sewer Costs Assessed Upon Property Benefited. An analysis of the returns indicates that the portion of sewer costs supposedly assessed upon property benefited varies as follows:

Reported portion of sewer cost paid by assessment %	*Number of municipalities*
100	1
75	2
66–⅔	2
60	2
58–½	1
50	9
40	1
35	1
33–⅓	1
30	1
25	3
20	3
17–½	1
15	1
0	4

SEWER ASSESSMENT PRACTICE IN MASSACHUSETTS

Municipality	Population 1950	Portion of Sewer Cost Reported to be Paid out of Tax Levy, %	Assessment on Real Estate: Equivalent per Lin. Ft. of Frontage	Assessment on Real Estate: Exemption on Corner Lots, Ft.	Remarks
Amesbury	9,678		1.015	Longest Side	*Total cost prorated.
Andover	12,261	None	*	75	Assessment rates may be doubled within a short time.
Beverly	28,855	80	1.50	One Side	
Braintree	23,130	40	3.50	100	
Brockton	62,856	41.5	0.875	60	
Brookline	56,952	75	1.50	One Side	
Cambridge	120,676	Varies	3.35	None	$0.85 per front ft. plus $0.02 per sq. ft. to depth of 125 ft.
Canton	4,725	80	1.24	75	Based on 50% of total cost as of 1932.
Chicopee	43,939	50	0.50	100	
Danvers	15,703	50	1.00	100	
Everett	45,789	*	*	40	*(½ the cost but not exceeding $2.00, ½ the cost exceeded
Fall River	111,759	100	None	None	(today the $2.00 max. The extra cost is paid from general
Fitchburg	42,671	100	None	All	(tax levy.
Gloucester	25,048	85	2.90	Short Side	$1.30 per front ft., 0.0016 per sq. ft. to depth of 100 t.
Greenfield	14,993	100	None		
Haverhill	47,213	70	1.10	Longest Side	Rates established Jan. 1, 1916.
Lawrence	80,427	82.5	0.65		No change since 1890.
Lexington	17,098	60±	2.50*		*Cost is based on average for previous 5 years.
Malden	59,779	75±	2.00	100 Max.	Rates established in 1926.
Marlboro	15,741	50	None	All	0.075 per 100 cu. ft. of water consumed.
Maynard	6,687	50	2.60	60 Max.	
Medford	66,109	67±	1.50*	80	This rate is about to be adjusted upward.

Melrose	26,919	50	*	50	*(50% of actual cost pro-rated. Now in process of
Nahant	2,654	50	None	80	(reviewing method of distributing cost of trunk sewers
					(and pumping station.
Natick	21,000		None		$0.25 per 100 cu. ft. of water used
Needham	16,252	33	4.10	100	Rates established in 1949
New Bedford	109,033	25+	2.00	50	*Max. assessment $2.00 per ft. Remainder from general
Newton	80,996	65+	4.05	None	tax levy.
Northbridge	10,000	25	7.60	60	
Pittsfield	53,055	Varies	1.90	None	
Quincy	83,190	33 1/3 to 50	3.94	None	Rates established in April, 1951. Previously they were
Reading	13,879	50-60	1.00	80	$0.96/lin. ft.
Salem	41,842		0.50	60	
Springfield	162,601	33 1/3	1.40	One S.de	
Swampscott	11,535	50	3.44*	Longest Side	*1950 rate.
Waltham	47,198		1.50	One Side	
Wellesley	20,847	80	1.40	70	0.40 per front ft. plus 0.008 per sq. ft. to depth of 125 ft.
Westfield	20,967	100			
Weymouth	32,695	75	2.00	100	
Winchester	15,567	40±	5.38*	One Side	*1950 rate.
Worcester	201,885		5.00		

The above tabulation indicates that 17 out of 33 municipalities are reported to charge 50 percent or more of the construction costs in sewer assessments. Actually, only eight municipalities assess sewer costs at rates representing 50 percent or more of the construction costs.

In Natick for new private developments, 100 percent of the construction cost of new sewers and house connections is paid by the developers. For any existing unaccepted street, 37-½ percent of the cost is assessed on each side of the street based upon the lot frontages. The Town pays 25 percent of the cost of sewers in these streets.

Exemption on Corner Lots. The assessment of sewer benefits on corner lots has frequently been the cause of strenuous objection on the part of the property owners affected. Needham has increased the exemption on corner lots from 60 ft. to 100 ft. In nine of the municipalities, the exemption includes the depth of the lot on one street. In six cases, the exemption is 100 ft. and in three cases it is 80 ft. In two municipalities, it is 75 ft.; in one, 70 ft.; in four, 60 ft.; in two, 50 ft.; and in one, it is 40 ft. No exemption is allowed on corner lots in five municipalities.

Sewer Service Charges. From the returns received from the municipalities in Massachusetts, we have summarized the sewer rental practice as follows:

Wellesley— Same as water service charge.

Reading— Unmetered service $6 per annum. For metered service as follows:

First	3,000	cu. ft.	$0.23	per	100	cu. ft.
Next	3,000	"	0.20	"	"	"
Next	4,000	"	0.16	"	"	"
Next	10,000	"	0.12	"	"	"
Next	10,000	"	0.08	"	"	"
Next	70,000	"	0.04	"	"	"

All in excess of 100,000 cu. ft. $0.02 per 100 cu. ft.

Brockton— $0.15 per 100 cu. ft. as measured by water meters.

Gloucester— Minimum charge $5; rate $0.05 per 100 ft. of water used up to 480,000 cu. ft. and $0.02 per 100 cu. ft. of water used over 480,000 cu. ft.

Natick— $0.25 per 100 cu. ft. of water used.

Marlborough— $0.075 per 100 cu. ft. of water used.

Danvers

Quarterly basis:

First	2,000	cu. ft.	at	$0.18	per	100	cu. ft.
Next	3,000	"	"	0.14	"	"	"
Next	5,000	"	"	0.10	"	"	"
Next	90,000	"	"	0.06	"	"	"
Over	100,000	"	"	0.03	"	"	"

Bridgewater—

Quarterly assessment at rate of 40 percent of cost of first 1,700 cu. ft. of water plus a charge of 25 percent of cost of water consumed in excess of 1,700 cu. ft.

Cambridge—

The City Council has failed to approve City Manager recommended sewer rental charges for the last two years.

Of General Interest

COUNT OFF

By Robert W. Moir, *Secretary*

At the close of the fiscal year, the membership count for all grades was 872. Have *you* ever wondered where the other 871 members were? The following tabulation reveals that in addition to Massachusetts members we have members in 28 states, the Dist. of Columbia and 6 foreign countries as follows:

New England	
Maine	18
New Hampshire	11
Vermont	3
Massachusetts	701
Rhode Island	21
Connecticut	8
Outside New England	
California	15
Delaware	1
Florida	4
Illinois	5
Indiana	1
Kentucky	2
Louisiana	2
Maryland	4
Michigan	3
Missouri	2
Montana	1
Nebraska	1
New Jersey	11
New York	24
North Carolina	3
Ohio	3
Oregon	1
Pennsylvania	4
Tennessee	2
Utah	1
Texas	1
Virginia	5
Washington	3
Dist. of Columbia	3
Foreign	
Brazil	1
Canada	1
Cuba	1
Puerto Rico	2
Turkey	1
Venezuela	1

In addition, our Journal goes (by subscription) to 33 states, the Dist. of Columbia and 20 foreign countries as follows:

United States	
Alabama	2
California	8
Colorado	3
Connecticut	2
Florida	1
Georgia	1
Idaho	1
Illinois	5
Iowa	2
Kansas	2
Louisiana	2
Maryland	2
Massachusetts	3
Michigan	4
Minnesota	1
Mississippi	1
Missouri	3
Montana	1
Nebraska	1
New Jersey	1
New York	11
North Carolina	2
Ohio	3
Oklahoma	1
Oregon	1

Pennsylvania	3
Rhode Island	1
Tennessee	3
Texas	6
Utah	1
Virginia	1
Washington	1
Wisconsin	1
Dist. of Columbia	4
Canadian Provinces	
Alberta	1
Ontario	2
Quebec	1
Saskatchewan	1
Foreign	
Argentina	1
Australia	5
Brazil	3
Ceylon	1
Egypt	1
England	4
Formosa	1
France	1
Germany	1
India	6
Indonesia	1
Israel	1
Italy	1
Japan	1
Portugal	1
Russia	9
So. Africa	2
Uruguay	1
Venezuela	2
Total Subscriptions	133

55 of these subscriptions are from Universities and Colleges, including 6 from foreign universities. Most of the remainder go to public libraries scattered throughout the world.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

JANUARY 30, 1952.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts & Sciences, 28 Newbury Street, Boston, Mass., and was called to order by President John B. Wilbur, at 7:05 P.M.

President Wilbur announced that the reading of the minutes of the previous meeting held December 12, 1951 would be waived unless there was objection.

It was *VOTED* "to dispense with the reading of the minutes of the December 12, 1951 meeting".

President Wilbur announced the death of the following members:—

George F. Haskell, who was elected a member April 21, 1920 and who died December 17, 1951.

Chester S. Allen, who was elected a member April 20, 1910, and who died January 6, 1952.

John N. Ferguson, who was elected a member April 15, 1896 and who died January 6, 1952.

President Wilbur stated that at the last meeting of the Society held on December 12, 1951, amendments to the By-Laws had been acted upon favorably and that now final action would be taken, in accordance with the provisions of the By-Laws requiring action at two successive meetings. The President referred to the proposed amendment which was sent to members in ESNE bulletin dated December 3, 1951.

It was moved and seconded that the Proposed Amendment to the By-Laws be adopted by the Society. The motion was carried unanimously.

President Wilbur stated that this was the final action on this matter.

Amendment to the By-Laws read as follows:—

Section 9 (Fees and Dues) Line 5—"there shall be no entrance Fee for Student Members, or for Members of Student Chapters seeking admission, within six months of graduation, to the Grade of Junior".

Secretary Robert W. Moir announced that the following had been elected to membership:—

Grade of Member—Leo Casagrande, Joseph P. Dever, George E. Dionne, Aiden R. Gierasch, Mortimer W. Meyer, Jr., Joseph G. A. Riccio.

Grade of Junior—W. Garrett Haggerty

President Wilbur called upon Secretary Robert W. Moir to announce the names of applicants for membership in the B.S.C.E.

President Wilbur made an announcement of the "Essay Contest for Young Engineers, sponsored by the ESNE. Subject to be "The Relationship Between the Young Engineer and his Professional Society".

President Wilbur then introduced the speaker of the evening:—

Ralph E. DeSimone, President, Merritt-Chapman Scott, Corp., New York. Subject—"The Delaware Memorial Bridge Substructure". Illustrated with slides.

A short discussion period followed after which members gathered in the Lounge where a collation was served.

One hundred twenty members and guests attended the meeting. The meeting adjourned at 8:15 P.M.

ROBERT W. MOIR, *Secretary*

FEBRUARY 20, 1952.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts & Sciences, 28 Newbury Street, Boston, Mass., and was called to order by President John B. Wilbur, at 7:05 P.M.

President Wilbur announced that the reading of the minutes of the previous meeting held January 30, 1952, would be waived unless there was objection.

It was *VOTED* "to dispense with the reading of the minutes of the January 30, 1952 meeting".

President Wilbur announced that this was a Joint Meeting with the Hydraulics Section, Gardner K. Wood, Chairman.

Secretary Robert W. Moir announced that the following had been elected to membership:—

Grade of Member—*Robert M. Beall, Henry R. DiCicco, Herbert E. Fletcher, 2nd, Robert A. Glines, Charles E. Hepler, *Leonard B. Loitherstein, James L. Sherard, *Carney M. Terzian.

Grade of Junior—Melvin R. Rubin, Frederick A. Tough, Jr.

President Wilbur called upon Secretary Robert W. Moir to announce the names of applicants for membership in the B.S.C.E.

President Wilbur called upon Gardner K. Wood, Chairman of the Hydraulics Section to conduct any business matters necessary for that Section.

President Wilbur then introduced the speaker of the evening:—

Brigadier General Don G. Shingler, Division Engineer, Dept. of the Army, Omaha, Nebr. Subject—"Missouri River Project".

A short discussion period followed after which members gathered in the Lounge where a collation was served.

One hundred thirty-one members and guests attended the meeting. The meeting adjourned at 8:25 P.M.

ROBERT W. MOIR, *Secretary*

MARCH 19, 1952.—The one hundred fourth annual meeting of the Boston Society of Civil Engineers was held today at the Hotel Vendome, 160 Commonwealth Avenue, Boston, Mass., and was called to order at 4:30 P.M., by President John B. Wilbur.

President Wilbur announced that the

reading of the minutes of the Society meetings have been omitted during the year. The minutes of the December, 1951 and January and February, 1952 meetings will be published in a forthcoming issue of the JOURNAL. The minutes of the April, May, June, September, October and November, 1951 meetings were declared approved as published.

The Secretary announced the following had been elected to membership:—

Grade of Member—David F. Carpenter, Thomas K. Dyer, Howard L. Millard, Charles E. Carver.

Grade of Junior—Thomas E. Rodgers, Jr.

President Wilbur called upon Secretary Robert W. Moir to announce the names of applicants for membership in the Society.

The Annual Reports of the Board of Government, Treasurer, Secretary, Auditors and Editor were presented. Reports were also made by the following committees:—Hospitality, Library, John R. Freeman Fund, Subsoils at Boston, Membership, Advertising, Publicity, Committee on Competitive Bidding for Engineering Services and final report of Quarters Committee.

It was *VOTED* "that the reports be accepted with thanks and placed on file".

The Annual Reports of the various sections were read.

It was *VOTED* "that the Annual Reports of the various Sections be accepted and placed on file".

The report of the Tellers of Election, Edward C. Keane and George G. Bogren was presented and in accordance therewith the President declared the following had been elected officers for the ensuing year.

President—Emil A. Gramstorff
Vice-President (for two years) Chester J. Ginder
Secretary (for one year) Robert W. Moir
Treasurer (for one year) Charles O. Baird, Jr.
Directors (for one year) Leslie J. Hooper; (for two years) James F. Brittain, Edwin B. Cobb
Nominating Committee (for two years) Charles E. Knox, Harold A. Thomas, Jr., Oliver G. Julian

The retiring President John B. Wilbur then gave his address on "The Relationship Between Civil Engineering and Architecture".

Eighty-one members and guests attended this part of the meeting.

The meeting adjourned to assemble at 7:30 P.M., the Annual Dinner being held during the interim.

President Wilbur called the meeting to order at 7:30 P.M.

President Wilbur then presented a quartet of Northeastern Students (Bob McGauley, Dick Mudgett, Walt Arvidson, Dick Ermerzian) who sang four numbers to the delight of all present.

Following general remarks and the introduction of the newly elected President Emil A. Gramstorff, and other guests at the head table President Wilbur announced that Honorary Membership in the Society had been conferred on two of the Society's distinguished members, in accordance with the vote of the Board of Government on February 18, 1952 and March 3, 1952:

Howard M. Turner, who has been a member since March 17, 1920
Karl Terzaghi, who has been a member since April 21, 1926.

President Wilbur announced that both the newly elected Honorary Members were unable to be present at this meeting but that they were ably represented by their wives. He then presented the certificates to Mrs. Howard M. Turner and Mrs. Karl Terzaghi.

The certificates read as follows:

"In recognition of his outstanding achievements in the field of Hydraulic Engineering including Water Power and Water Supply Engineering,

HOWARD MOORE TURNER

has been duly elected an

HONORARY MEMBER

By direction of the Board of Government
February 18, 1952

ROBERT W. MOIR (Seal) JOHN B. WILBUR
Secretary *President*

"In recognition of his founding and subsequent outstanding contributions to the development of the Science of Soil Mechanics in the United States and throughout the world,

KARL TERZAGHI

has been duly elected an

HONORARY MEMBER

By direction of the Board of Government
March 3, 1952

ROBERT W. MOIR (Seal) JOHN B. WILBUR
Secretary *President*

President Wilbur stated that a number of prizes are awarded annually for worthy papers presented at the Society and Section meetings. The Secretary read the names of recipients and asked them to come forward. President Wilbur presented the following awards:—

AWARD	RECIPIENT	PAPER
Desmond FitzGerald Medal	John M. Biggs	"The Design of Eccentrically Loaded Steel Columns".
Clemens Herschel Award	George R. Rich	"The Hudson River Emergency Pumping Plant of the City of New York".
Structural Section Award	Myle J. Holley, Jr.	"Conical Shell Theory Applied to Concrete Tanks".
Transportation Section Award	Howard J. Williams	"The New Jersey Turnpike".
Hydraulics Section Award	Carroll T. Newton	"An Experimental Investigation of Bed Degradation in an Open Channel".
Northeastern Student Chapter Award	Angelo J. Polvere	"The Improvements to the Cambridge Water Works".

President Wilbur introduced the guest speaker of the evening Prof. Albert G. H. Dietz, who gave a most interesting talk on "Japan in Transition: Architecture; Landscaping; and Geishas". The talk was illustrated by three dimensional slides.

At the conclusion of the address President Wilbur on behalf of the Society thanked Prof. Dietz for a most enjoyable talk and then turned the meeting over to President elect, Emil A. Gramstorff.

President elect Emil A. Gramstorff presented retiring President John B. Wilbur with a certificate for services rendered and then adjourned the meeting at 9:55 P.M.

Two hundred fifty-two members and guests attended the dinner meeting.

ROBERT W. MOIR, *Secretary*

STRUCTURAL SECTION

NOVEMBER 14, 1951.—A meeting of the Structural Section was held in the Society Rooms with Chairman Frank A. Cundari presiding.

The Chairman introduced the speaker, Prof. Harl P. Aldrich, Jr. of Massachusetts Institute of Technology, who spoke on the subject "Importance of the Net Load to the Settlement of Buildings in Boston".

Prof. Aldrich described a soils investigation at the new Charles Hayden Memoral Library at M.I.T. Thirty-two settlement observation points were placed throughout the basement during the early stages of construction and ten piezometers were installed in the 90 foot thick layer of Boston blue clay which underlies the building. Results of the investigation were discussed and the importance of the net building load (the net building load being equal to the dead load plus the line load minus the weight of excavated soil) and the past history of the clay was demonstrated. Time settlement curves for eight buildings in the Boston area were compared with their respective net stresses deep in the clay where the precompression is very small. It appears that if these net stresses are below approximately 0.2 tons per sq. ft., settlement as a result of consolidation of the clay will not be excessive.

A short discussion period followed. The attendance was 73.

J. M. BIGGS, *Acting Clerk*

DECEMBER 12, 1951.—A joint meeting of the Structural Section with the main Society was held in the Society Rooms with Chairman Frank A. Cundari of the Structural Section presiding.

The speaker was Dr. Karl Terzaghi, Professor of Practice of Civil Engineering at Harvard University, whose subject was "Permafrost".

Dr. Terzaghi first discussed and illustrated with slides the fundamental nature of the permafrost phenomena. He then discussed the application of these ideas to several specific foundation problems which he had encountered in his consulting experience. He also entertained the group with some of his own colored movies taken on his airplane trips to Alaska in connection with these projects.

This very interesting and informative talk stimulated considerable discussion which was enjoyed by all of the 124 members and guests present. Chairman Cundari turned the chair over to President John B. Wilbur who thanked Dr. Terzaghi and then adjourned the meeting.

CHARLES H. NORRIS, *Clerk*

JANUARY 9, 1952—A meeting of the Structural Section was held in the Society Rooms with Chairman Frank A. Cundari presiding.

After the minutes of the previous meeting had been read and approved, Chairman Cundari announced that the next meeting would be held jointly with the Boston Section of the American Welding Society at the M.I.T. Graduate House on February 11, 1952.

The Chairman then introduced the speaker, Professor Albert G. H. Dietz

of Massachusetts Institute of Technology who spoke on the subject "Japanese Earthquake Resistant Construction."

Professor Dietz illustrated his talk with slides which he took last summer when he toured Japan for two months as a member of a panel of American Engineering Educators. These slides showed typical features of Japanese building construction and he discussed the code provisions which governed their earthquake resistant design. He showed a number of slides of a very interesting Tokyo building in which the contractor proceeded downward with the substructure and upward with the superstructure simultaneously.

An extensive and interesting discussion period followed. The attendance was 25.

CHARLES H. NORRIS, *Clerk*

FEBRUARY 11, 1952.—A joint meeting of the Structural Section with the Boston Section of the American Welding Society was held in the Campus Room at the Massachusetts Institute of Technology with Mr. H. Hugs Stahl of the Welding Society presiding.

Mr. Stahl turned the meeting over briefly to Chairman Frank A. Cundari for a short business meeting of the Structural Section. It was voted that the Chairman appoint a committee to bring in nominations for the coming year at the March Section meeting. The Chairman appointed the following committee: Arthur E. Harding, Chairman, Ernest L. Spencer and Oliver G. Julian.

Mr. Stahl introduced the technical speaker of the evening, Mr. Van Rensselaer P. Saxe of Baltimore whose subject was "How to save 30 per cent on Structural Steel". Mr. Saxe has had a vast experience in the design of welded buildings of all types, having designed his first welded steel building in 1927. He illustrated his talk with slides showing numerous buildings, and typical details which he uses. It was particularly interesting to see how his detailing and connections have changed over the twenty-five years of his experience. Today practically all of his connections are made with butt welds.

There was a total attendance of approximately 120 of whom about 25 were from the Structural Section.

CHARLES H. NORRIS, *Clerk*

MARCH 12, 1952.—The annual meeting of the Structural Section was held at the Society rooms with Chairman Frank A. Cundari presiding. The Nominating Committee submitted its slate of officers for the following year. There being no further nominations, this slate of officers was duly elected: Chairman, Edward C. Keane; Vice Chairman, Charles Norris; Clerk, Dr. Ruth D. Terzaghi; Executive Committee, Jean Ducharme, E. B. Myott, and C. J. Kray.

The speaker was Mr. Thomas E. Dalby of Bethlehem Steel Co., who gave an illustrated talk on "Production and Disturbance of Structural Shapes."

The attendance was 32.

CHARLES H. NORRIS, *Clerk*

HYDRAULICS SECTION

FEBRUARY 20, 1952.—A joint meeting of the Hydraulics Section with the Boston Society of Civil Engineers was held at the American Academy of Arts and Sciences.

The meeting was called to order at 7:05 P.M. by Mr. John B. Wilbur, President of the Society. A short business meeting of the Hydraulics Section was held with Gardner K. Wood in the chair. The minutes of the previous meeting were approved without being read.

The Nominating Committee presented the following slate of officers for the Hydraulics Section for the coming year:

Byron O. McCoy, Chairman
Lincoln W. Ryder, Vice-Chairman
Arthur T. Ippen, Clerk
Ralph S. Archibald, Julian H. White, and Frederick C. Merrikin—Executive Committee

It was voted to approve the Nominating Committee's report and there being no further nominations from the floor, the Clerk was instructed to cast 1 ballot for the slate nominated, and they were declared elected.

Announcement was made of the next meeting of the Hydraulics Section to be held at the Alden Laboratories at Worcester Polytechnic Institute in May; and there being no further Section business, the meeting was turned back to President Wilbur for Society business.

President Wilbur announced the annual meeting would be held on March 19 at the Hotel Vendome and that Professor Dietz of M.I.T. would present a travelogue with colored slides of his trip to Japan.

The President, Mr. Wilbur, introduced the speaker of the evening, Brigadier-General Don G. Shingler who spoke in place of Major-General Louis A. Pick, on the subject of the Missouri River Project. This talk was illustrated with slides.

After a short question period, the meeting was adjourned at 8:25 P.M. One hundred thirty-one members and guests were present. Refreshments were served in the upstairs meeting hall after the adjournment of the Society's meeting.

L. W. Ryder, *Clerk*

SURVEYING AND MAPPING SECTION

October 24, 1951.—The fifteenth meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:15 P.M. Wednesday, October 24, 1951.

Fifty-four members and guests were present.

The meeting was called to order by the Chairman, John J. Vertic. The minutes of the April 4th meeting were read by the clerk and approved. The business brought before the Section was the appointment of a Nominating Committee to submit a slate of officers for the coming year.

The speaker of the evening was Mr. H. Lowell Crocker of Fay, Spofford and Thorndike, whose talk was entitled "Location Planning and Land Takings for the John F. Fitzgerald Expressway."

A long question and answer period followed the speaker's illustrated talk which indicated an unusual interest in the topic presented.

Meeting adjourned at 9:00 P.M.

George W. Hankinson, *Clerk*

January 16, 1952.—The sixteenth meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:15 P.M.

Approximately 37 members and guests were present. Chairman John J. Vertic called for the reading of the minutes of the October 24th, 1951, meeting. The meeting minutes were approved as read. The report of the Nominating Committee was called for by the chairman and was read by Charles M. Anderson.

On motion duly made and seconded it was voted: to accept the report of the Nominating Committee.

On motion made and seconded it was voted that nominations be closed.

On motion duly made and seconded it was voted that the secretary be directed to cast one ballot for the slate of officers named in the report of the Nominating Committee.

The clerk cast one ballot for the following who were thereupon declared duly elected:

Chairman—George W. Hankinson
Vice Chairman—C. Frederick Joy, Jr.
Clerk—Wilbur C. Nylander
Executive Committee—John J. Vertic, Llewellyn T. Schofield, and Alexander J. Bone

Chairman Vetic introduced Edward H. Cameron of Jackson and Moreland who spoke on the subject of "Hoosac Tunnel". Mr. Cameron was assisted by Mr. Vetic who spoke on the surveying and construction problems encountered in the building of the Tunnel. An added attraction was an interesting personal insight of the life of a member of

the construction crew of the Hoosac Tunnel given by Mr. Douglas K. Merrill. Several pieces of surveying equipment used in the construction of the Tunnel were on display.

The meeting adjourned at 9 P.M.

GEORGE W. HANKINSON, *Clerk*

ADDITIONS

Members

Thomas H. Murphy, 46 Cornhill, Boston, Mass.
Jeronimas V. Dabrila, 23 Romsey Street, Dorchester, Mass.
Otto Germanis, 32 Tower Street, Jamaica Plain, Mass.
Frank Holzer, 51 President's Lane, Quincy, Mass.
Roland S. Burlingame, 63 Mountfort Street, Boston, Mass.
Shih Hus Wang, 119 Magazine Street, Cambridge, Mass.
William Doubleday, 243 Charles Street, Reading, Mass.
Clarence H. Pratt, 477 Essex Street, Lawrence, Mass.
Lee Marc Wolman, 10A Bellevue Avenue, Cambridge, Mass.
George M. Castaldo, 25 Englewood Avenue, Brookline, Mass.
William J. Faulkner, 207 Everett Street, Wollaston, Mass.
Richard W. Albrecht, 23 Greenwood Road, Natick, Mass.
Salvator P. Grasso, Town Hall, Milford, N. H.
Edmond R. Foster, 24 Manchester Road, Newton Highlands, Mass.
Paul O'Connell, 28 Grove Street, Quincy, Mass.
Henry L. Kennedy, 69 Radcliff Road, Belmont, Mass.
Joseph P. Dever, 752 Boylston Street, Brookline, Mass.
Joseph G. A. Ricco, 932 Hope Street, Bristol, R. I.
Aiden R. Gierash, 62 Warren Street, Needham, Mass.
George E. Dionne, 60 Parker Street, Chelsea, Mass.
Joseph W. Dill, Box 600, West Groton, Mass.
Robert A. Glines, 53 Lawrence Street, Danvers, Mass.
Herbert E. Fletcher, 2nd, H. E. Fletcher Company, Chelmsford, Mass.
Henry R. DiCicco, 118 Florence Avenue, Arlington, Mass.
Wilfred M. Hall, Chas. T. Main, 80 Federal Street, Boston, Mass.

Juniors

Sanford S. Johnson, 138 Marlborough Street, Boston, Mass.
Henry H. Yung, 68 Pleasant Street, Cambridge, Mass.
Frank T. Smith, Jr., 865 V.F.W. Parkway, W. Roxbury, Mass.
Salvatore A. Martonana, 345 E. 8th Street, So. Boston, Mass.
Charles E. Serpis, 52 Charlesgate East, Boston, Mass.
Stanley B. Goldberg, 26 Millet Street, Dorchester, Mass.
George Millman, 100 Kilsyth Road, Brighton, Mass.
William A. Cawley, 48 Boundary Road, Malden, Mass.
Robert B. Nunley, Summer Street, Marshfield, Mass.

DEATHS

Fred J. Nebbiker, June 29, 1951
Dugald C. Jackson, July 1, 1951
Dean Peabody, Jr., Aug. 7, 1951
G. Nelson Perry, Sept. 5, 1951
Chester A. Richardson, Sept. 7, 1951
Charles F. Knowlton, Sept. 19, 1951
Samuel P. Coffin, Nov. 2, 1951
William T. Morrissey, Nov. 6, 1951
William R. Cuff, April, 1951
George F. Haskell, Dec. 17, 1951
Chester S. Allen, Jan. 6, 1952
John N. Ferguson, Jan. 5, 1952
John R. Carroll, Dec. 23, 1950

ANNUAL REPORTS

REPORT OF THE BOARD OF GOVERNMENT FOR YEAR 1951-1952

Boston, Mass., March 17, 1952

To the Boston Society of Civil Engineers:

Pursuant to the requirements of the By-Laws the Board of Government presents its report for the year ending March 19, 1952.

The following is a statement of the status of membership in the Society:—

Honorary	10	Juniors	49
Members	786	Students	4
Associates	3	Student Chapters	1

Summary of Additions

New Members	53	New Juniors	12

Reinstatements

Members	4

Summary of Transfers

Juniors to Member	8	Students to Junior	3

Summary of Loss of Members

Deaths	18	Dropped for non-payment of dues	13
Resignations	6	Dropped for failure to transfer	4

Members exempt from dues	92
Remission of dues	3
Applications pending on March 17, 1952	19

Honorary Membership is as follows:

Dr. Karl T. Compton, elected, February 13, 1932
Arthur W. Dean, elected, January 14, 1942
Charles M. Spofford, elected, December 19, 1945
Harold K. Barrows, elected, February 15, 1950
Frank M. Gunby, elected, February 15, 1950
Richard K. Hale, elected, February 15, 1950
Charles W. Sherman, elected, February 19, 1947
Karl R. Kennison, elected, February 7, 1951
Howard M. Turner, elected, February 18, 1952
Karl Terzaghi, elected, March 3, 1952

The following members have been lost through death:

Chester S. Allen, Jan. 6, 1952
John R. Carroll, Dec. 23, 1950
Samuel P. Coffin, Nov. 2, 1951
William R. Cuff, April, 1951
John T. Donahoe, Mar. 26, 1951
John N. Ferguson, Jan. 5, 1952
George F. Haskell, Dec. 17, 1951
Dugald C. Jackson, July 1, 1951
Charles F. Knowlton, Sept. 19, 1951
Edward L. Moreland, June 17, 1951
William T. Morrissey, Nov. 6, 1951
Fred J. Nebbiker, June 29, 1951
Dean Peabody, Jr., Aug. 7, 1951
G. Nelson Perry, Sept. 5, 1951
Chester A. Richardson, Sept. 7, 1951
Arthur T. Safford, Apr. 3, 1951
Ray L. Schoppe, Feb. 27, 1951
Arthur L. Sparrow, Apr. 6, 1951

Meetings of the Society

March 21, 1951.—Address of the retiring President, Thomas R. Camp. "The Professional Engineer in a Free Society".

April 25, 1951.—Joint Meeting with American Society of Civil Engineers, Northeastern Section and Transportation Section, BSCE. "The Proposed Boston Central Artery", Dr. John B. Wilbur and Edward C. Keane.

May 16, 1951.—"Development of the Port of Boston", by George L. Wey, Chief Engineer, Port of Boston Authority.

September 26, 1951.—"Power Production and Transmission in Turkey", Wilfred M. Hall, Chas. T. Main, Inc.

October 17, 1951.—Student Night. Joint Meeting with American Society of Civil Engineers, Northeastern Section. "Romance of Bridges", D. B. Steinman, Consulting Engineer, New York.

November 19, 1951.—Symposium—"Engineering in Connection with Transportation Facilities of the M.T.A.," E. B. Myott, Supt. of Engineering and Maintenance of M.T.A. "Design and Construction Methods of the Proposed Tunnel Between Park Street and Scollay Square for the M.T.A.", Ole Singstad, Partner of Singstad and Baillie, N. Y. "Basis of Structural Design for Proposed Subways for the M.T.A.", Emil H. Praeger, Partner of Praeger-Maguire.

December 12, 1951.—"Permafrost", Dr. Karl Terzaghi, Harvard University.

January 30, 1952.—"The Delaware Memorial Bridge Substructure", Ralph E. DeSimmone, President of the Merritt-Chapman & Scott Corp., N. Y.

February 20, 1952.—"Missouri River Project", Brigadier General Don G. Shingler, Division Engineer, Dept. of the Army, Washington, D.C.

Attendance at Meetings

DATE	PLACE	MEETING	DINNER
March 21, 1951	Hotel Vendome	71	253
April 25, 1951	Ninety-nine Club	89	89
May 16, 1951	American Academy of Arts & Sciences	51	
September 26, 1951	American Academy of Arts & Sciences	110	*
October 17, 1951	Northeastern University	405	337
November 19, 1951	American Academy of Arts & Sciences	156	*
December 12, 1952	Tremont Temple	124	
January 30, 1952	American Academy of Arts & Sciences	120	*
February 20, 1952	American Academy of Arts & Sciences	131	*

*Collation served in Lounge after meeting.
Average attendance—140

Sections

Twenty-two meetings were held by the Sections of the Society during the year. These meetings of the Sections offering opportunity for less formal discussion have continued to demonstrate their value to their members and to the Society. The variety of subjects presented has made an appeal to the members as indicated by the general attendance at these meetings. The Annual Reports of the various Sections will be presented at the Annual Meeting and will be published in the April issue of the JOURNAL.

*Funds of the Society**

Permanent Fund. The Permanent Fund of the Society has a present value of $58,713.25. The Board of Government authorized the use of as much as necessary of the current income of this fund in payment of current expenses.

John R. Freeman Fund. In 1925 the late John R. Freeman, a Past President and Honorary Member of the Society, made a gift to the Society of securities which was established as the John R. Freeman Fund. The income from this Fund is to be particularly devoted to the encouragement of young engineers. Mr. Freeman suggested several uses, such as the payment of expenses for experiments and compilations to be reported before the Society; for underwriting meritorious books or publications pertaining to hydraulic science or art; or a portion to be devoted to a yearly prize for the most useful paper relating to hydraulics contributed to this Society; or establishing a traveling scholarship every third year open to members of the Society for visiting engineering works, reports of which would be presented to the Society. No additional scholarship was authorized during the year.

Edmund K. Turner Fund. In 1916 the Society received 1,105 books from the library of the late Edmund K. Turner, and a bequest of $1,000, "the income of which is to be used for library purposes". The Board voted to use $25 of the income for the purchase of books, for the library. The expenditure from this fund during the year was $18.80.

Alexis H. French Fund. The Alexis H. French Fund, a bequest amounting to $1,000 was received in 1931, from the late Alexis H. French of Brookline, a former Past President of the Society. The income of the fund is "to be devoted to the library of the Society". The Board voted to use $65 of the available

*Details regarding the values and income of these funds are given in the Treasurer's report.

income for the purchase of books for the library. The expenditure from this fund during the year was $61.61.

Tinkham Memorial Fund. The "Samuel E. Tinkham Fund" established in 1921, at the Massachusetts Institute of Technology by the Society, "to assist some worthy student of high standing to continue his studies in Civil Engineering", had a value of $2,596.86 on June 30, 1951. Jack H. King of Everett, Washington, a student in Civil Engineering, class of 1953 was awarded this Scholarship of $100 for the year 1951-52.

Desmond FitzGerald Fund. The Desmond FitzGerald Fund, established as a bequest from the late Desmond FitzGerald, a Past President and Honorary Member of the Society, provided that the income from this fund shall "be used for charitable and educational purposes." The Board voted on April 2, 1951 to appropriate from the income of this fund the sum of $100 to be known as the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald, and to be given to a student at Northeastern University.

Clemens Herschel Fund. This fund was established in 1931, by a bequest of $1,000 from the late Clemens Herschel, a former Past President and Honorary Member of the Society. The income from this fund to be used for presentation of prizes for papers which have been particularly useful and commendable and worthy of grateful acknowledgment. The Board of Government voted on April 2, 1951 "to use this fund for Clemens Herschel Prizes only, for the next few years until this fund is built up substantially". No expenditure from this fund was made during the year.

Edward W. Howe Fund. This fund, a bequest of $1,000 was received December 2, 1933, from the late Edward W. Howe, a former Past President of the Society. No restrictions were placed upon the use of this money, but the recommendation of the Board of Government was that the fund be kept intact, and that the income be used for the benefit of the Society or its members. The expenditure made during the year from this fund was $83.86 for prizes.

William P. Morse Fund. This fund, a bequest of $2,000 was received April 8, 1949, from the late William P. Morse, a former member of the Society. No restrictions were placed upon the use of this money but the recommendations of the Board of Government were that the fund be kept intact and that the income be used for the benefit of the Society or its members. No expenditure was made this year from this fund.

Prizes

AWARD	RECIPIENT	PAPER
Desmond FitzGerald Medal	John M. Biggs	"The Design of Eccentrically Loaded Steel Columns".
Clemens Herschel Award	George R. Rich	"The Hudson River Emergency Pumping Plant of the City of New York".
Structural Section Award	Myle J. Holley, Jr.	"Conical Shell Theory Applied to Concrete Tanks".
Transportation Section Award	Howard J. Williams	"The New Jersey Turnpike".
Hydraulics Section Award	Carroll T. Newton	"An Experimental Investigation of Bed Degradation in an Open Channel".
Northeastern Student Chapter Award	Angelo J. Polvere	"The Improvements to the Cambridge Water Works".

Journal

The complete report of the Editor of the JOURNAL will be published in the April, 1952, JOURNAL.

Library

The report of the Library Committee contains a complete account of the library activities during the past year.

Committees

The usual special committees dealing with the activities and conduct of the Society were appointed. The membership of these committees is published in the JOURNAL and the reports of the committees will be presented at the Annual Meeting.

During the past year an Amendment to Section 9 (Fees and Dues) of the By-Laws was completed. Amendment reads as follows:—

Line 5; "there shall be no entrance fee for Student Members, or for Members of Student Chapters seeking admission, within six months of graduation, to the grade of Junior".

In addition to routine business of the Society the Board of Government took action as follows:—

Meeting of May 14, 1951

The Board voted "that President John B. Wilbur, be officially designated a delegate to represent the Boston Society of Civil Engineers at the Building Research Congress to be held at the Institution of Civil Engineers, London, England, September 11-20 inclusive".

Meeting of September 24, 1951

The Board voted "that the President and Secretary contact the Engineering Societies of New England concerning increase in rental paid to the Boston Society of Civil Engineers." Subsequently the ESNE Council approved an increase of $200 rental annually to the BSCE, retroactive to January 1, 1952.

Meeting of October 15, 1951

The Board voted "that the Boston Society of Civil Engineers accept the invitation of the Column Research Council to become a member of the Council".

Voted "to exchange publications with the United Nations Bureau of Flood Control Economic Commission for Asia and the Far East and to send them a Cumulative Index".

Meeting of December 10, 1951

The Board voted "that the Secretary be authorized to grant permission to the Department of the Army to reproduce certain material from our publication 'Contributions to Soil Mechanics' ".

Meeting of February 18, 1952

The Board *voted* "to authorize the committee on Subsoils of Boston to proceed with the collection of papers and to obtain cost estimates for the publication of a second volume of 'Contributions to Soil Mechanics', and to procure estimates for re-publication of the first volume".

Voted "to release all papers of the Freeman Hydraulic Lectures for publication by their respective authors".

Your Board in conclusion, wishes to express its appreciation of the excellent work done by the Officers of the Sections and by the Committee of the Society.

JOHN B. WILBUR, *President*

REPORT OF THE TREASURER

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

The report that follows is the report of the treasurer for the fiscal year March 1, 1951 to March 1, 1952.

At the May and June 1950 meetings of the Society it was voted to appropriate a sum not to exceed $8,000 for the use of the Quarters Committee. During the fiscal year ending March 1, 1951, the Committee expended $6,966.09. The job of remodelling, redecorating and refurbishing the Society rooms was completed this year, with an additional expenditure of $932.28. An unexpended balance of $101.63, from the original $8,000 appropriation, was returned to the Permanent Fund.

The accompanying table compares the receipts to the current fund and expenditures from the current fund during the last five years.

	1947-48	1948-49	1949-50	1950-51	1951-52
Receipts					
Dues	$ 5,188	$ 5,832	$ 6,172	$ 6,192	$ 6,643
Other than dues	3,964	4,392	4,884	6,171	6,004
Total Receipts	$ 9,152	$10,224	$11,056	$12,363	$12,647
Total Expenditures	12,113	15,112	14,266	14,772	15,760
Deficit	$ 2,961	$ 4,888	$ 3,210	$ 2,409	$ 3,113*

The deficit as above noted has been provided for by the income to the Permanent Fund and by drawing upon the principal of the Permanent Fund. The table below compares the transfer of income and principal from the Permanent Fund for the past five years.

	1947-48	1948-49	1949-50	1950-51	1951-52
Permanent Fund					
Income	$ 2,778	$ 2,754	$ 2,947	$ 2,409	$ 2,887
Principal	183	2,134	263		
Total	$ 2,961	$ 4,888	$ 3,210	$ 2,409	$ 2,887*

*$3,113.11 of which $2,887.31 was transferred from the income of the Permanent Fund and $225.80 from current cash. The Board of Government voted at the April meeting that all expenditures of the Membership Committee be taken out of the income to the Permanent Fund. This amounted to a sum of $225.30. Because March 1, 1952 was a Saturday, $215.63 that normally would have been income as of March 1, 1952 was not collected until March 3, 1952. A few of February's bills were slightly larger than anticipated plus the fact the banks are closed Saturday accounts for March 1, 1952 current fund being $225.80 lower than a normal value of $1,500.00.

The Boston Safe Deposit and Trust Company has continued to act as our investment counsel and as custodian of all of our securities. All purchases of securities have been made upon the recommendation of The Boston Safe Deposit and Trust Company and with the approval of our Board of Government.

The following purchases of securities have been made during the year:—

	Par value
Bonds	
Public Service Electric & Gas Company	$1,000
Province of Ontario	$2,000
Stocks	
3/7 share Central Hanover Bank and Trust Company	
10 shares Union Carbide and Carbon	

The following table shows the book values of securities and bank deposits, but does not include the value of the library and physical property for the last five years.

	Mar. 8, 1948	Mar. 1, 1949	Mar. 1, 1950	Mar. 1, 1951	Mar. 1, 1952
Bonds	$36,307.51	$36,444.73	$35,549.23	$35,549.23	$38,494.23
Coop. Bank	13,150.97	11,380.60	9,570.22	9,764.13	9,984.59
Stocks	56,767.69	53,426.70	53,438.36	45,257.17	45,836.05
Cash	3,078.33	3,203.79	2,538.11	4,473.09	1,513.75
Publication Fund	700.00				
Total	$110,004.50	$104,455.82	$101,095.92	$95,043.62	$95,828.62

The financial standing of the Society on March 1, 1952 is shown in four tables attached to this report.

Table I Distribution of Funds—Receipts and Expenditures
Table II Record of Investments—Bonds
Table III Record of Investments—Stocks
Table IV Record of Investments—Co-operative Banks

At the close of the year the book value of all securities plus cash on hand was $95,828.62. This is an increase of $785.00 over last year. The rate of income from all funds was 4.88% compared with 4.82% last year and 4.55% two years ago.

The treasurer's cash balance of $1,513.75 includes $60.00 withholding tax of the Office Secretary and $21.50 Social Security payments for the Office Secretary, Editor, and Secretary. These taxes and social security payments are a portion of the first quarters payments payable to the Collector of Internal Revenue in April 1952. The Secretary's "change fund" of $30.00 is not included in the Treasurer's report.

CHARLES O. BAIRD, JR., *Treasurer*

Table I—Distribution of Funds—Receipts and Expenditures

	Book Value March 1, 1951 1	Interest and Dividends Cash 2	Interest and Dividends Credit 3	Net Profit or Loss at Sale or Maturity + 4	Net Profit or Loss at Sale or Maturity — 5	Transfer of Funds Purchased + 6	Transfer of Funds Sold — 7	Book Value March 1, 1952 8
Bonds	$35,549.23	$ 896.58	—	—	—	$2,945.00	—	$38,494.23
Cooperative Banks	9,764.13	25.00	$220.46	—	—	—	—	9,984.59
Stocks	45,257.17	3,424.10	—	—	—	589.22	$ 10.34	45,836.05
Available for Investment	2,973.09	—	—	—	—	—	2,733.54	239.55
	$93,543.62	$4,345.68	$220.46	0.00	0.00	$3,534.22	$2,743.88	$94,554.42

Columns 1 + 3 + 6 — 7 = 8

Funds	Book Value March 1, 1951	Allocation of Income—Profit and Loss Columns 2 & 3 Income	Allocation of Income—Profit and Loss Columns 4 & 5 Net Loss	Received	Expended	Book Value March 1, 1952
Permanent	$59,150.53	$2,887.31	—	$ 495.00	$ 3,819.59*	$58,713.25
John R. Freeman	25,863.42	1,262.47	—	50.72	17.20	27,159.41
Edmund K. Turner	1,024.67	50.02	—	—	18.80	1,055.89
Desmond FitzGerald	2,154.81	105.18	—	—	100.00	2,159.99
Alexis H. French	1,066.54	52.06	—	—	61.61	1,056.99
Clemens Herschel	1,007.12	49.16	—	—	—	1,056.28
Edward W. Howe	1,098.92	53.64	—	—	83.86	1,068.70
William P. Morse	2,177.61	106.30	—	—	—	2,283.91
	$93,543.62	$4,566.14	—	$ 545.72	$ 4,101.06	$94,554.42
Current Cash	1,500.00	2,887.31	—	12,646.76	15,759.87	1,274.20
Totals	$95,043.62	$7,453.45	—	$13,192.48	$19,860.93	$95,828.62

Secretary's change fund of $30.00 should be added to show total cash.

Cash balance March 1, 1952

Investment Fund	$ 239.55
Current Account	1,274.20
Total	$1,513.75

*

$2,887.31	From income of Permanent Fund
932.28	From principal (Quarters improvements)
$3,819.59	

TABLE II—RECORD OF INVESTMENTS—BONDS
March 1, 1951 to March 1, 1952

Bonds	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest Received	Par Value	Book Value	Market Value
United States Bonds, Series G	June 1, 1953	2½%	$200.00	$ 8,000.00	$ 8,000.00	$ 8,000.00
United States Bonds, Series G	July 1, 1954	2½%	175.00	7,000.00	7,000.00	7,000.00
United States Bonds, Series G	Nov. 1, 1956	2½%	25.00	1,000.00	1,000.00	1,000.00
United States Bonds, Series G	May 1, 1958	2½%	100.00	4,000.00	4,000.00	4,000.00
United States Bonds, Series G	May 1, 1960	2½%	25.00	1,000.00	1,000.00	1,000.00
American Telephone and Telegraph Company	Dec. 15, 1961	2¾%	55.00	2,000.00	2,033.90	2,140.00
Consumers Power Company First Mortgage	Sept. 1, 1975	2⅞%	43.13	3,000.00	3,140.35	2,910.00
Pennsylvania Railroad Company	June 1, 1965	4½%	45.00	1,000.00	1,017.74	1,010.00
Public Service Electric and Gas	June 1, 1979	2⅞%	86.24	3,000.00	3,107.50	2,972.89
Puget Sound Power and Light	Dec. 1, 1972	4½%	42.50	1,000.00	1,058.44	1,040.00
Southern Pacific Oregon	Mar. 1, 1977	4½%	90.00	4,000.00	4,191.30	3,960.00
Public Service Electric and Gas	June 1, 1979*	2⅞%	19.10	1,000.00	990.00	947.11
Province of Ontario	Sept. 1, 1972**	3¼%	—9.39***	2,000.00	1,955.00	1,960.00
Totals			$896.58	$38,000.00	$38,494.23	$37,940.00

*Purchased April 3, 1951
**Purchased Oct. 23, 1951
***Accrued interest at time of purchase

TABLE III—RECORD OF INVESTMENTS—STOCKS
March 1, 1951 to March 1, 1952

Stocks	Classification	Number of Shares	Dividend Received	Book Value March 1, 1952	Market Value March 1, 1952
American Telephone and Telegraph Company	Common	52	$ 468.00	$ 6,042.74	$ 8,008.00
Hanover Bank	Common*	29*	116.00	2,523.57	2,639.00
Consolidated Edison Co. of New York Inc.	Common	50	100.00	2,556.12	1,650.00
Continental Insurance Co.	Common	50	125.00	2,026.21	3,500.00
General Electric Company of New York	Common	50	150.00	2,341.47	2,700.00
Hartford Fire Insurance Company	Common	14	42.00	839.75	1,946.00
National Dairy Products Corp.	Common	50	145.00	1,154.74	2,500.00
New England Electric System	Common	108	86.40	1,813.86	1,296.00
North American Trust Shares	July 15, 1955	1500	405.00	5,342.00	7,455.00
Pacific Gas and Electric Co.	Preferred	100	150.00	2,704.89	3,300.00
Pacific Gas and Electric Co.	Common	50	100.00	1,381.47	1,750.00
Radio Corporation of America	Preferred	20	70.00	1,720.75	1,520.00
Southern California Edison Co. Ltd.	Preferred	40	80.00	1,161.22	1,800.00
Southern California Edison Co.	Common	45	90.00	1,374.25	1,530.00
Southern Railway Company	Preferred	15	75.00	1,136.80	975.00
Standard Oil of New Jersey	Common	108	445.50	3,328.16	7,992.00
Texas Company	Common	104	317.20	2,956.32	5,512.00
Union Carbide and Carbon Corp.	Common**	100**	195.00	2,958.44	5,700.00
Union Pacific Railroad	Common	44	264.00	2,473.29	4,840.00
Totals			$3,424.10	$45,836.05	$66,613.00

*3/7 Share purchased June 20, 1951
**10 Shares purchased April 3, 1951

TABLE IV—RECORD OF INVESTMENTS—CO-OPERATIVE BANKS
March 1, 1951 to March 1, 1952

Co-operative Banks	Classification	Number of Shares	Dividend Rate*	Dividend Received	Book Value March 1, 1952	Market Value March 1, 1952
Suffolk Co-operative Federal Savings and Loan Association	Matured Certificate	5	2¼%	$ 25.00	$1,000.00	$1,000.00
Suffolk Co-operative Federal Savings and Loan Association	Savings Account		2¼%	220.46	8,984.59	8,984.59
				$245.46	$9,984.59	$9,984.59

*The dividend rate for the fiscal year March 1, 1951 to March 1, 1952 was 2½%.

REPORT OF THE SECRETARY

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

The following is a statement of cash received by the Secretary and of the expenditures approved by the President, in accordance with the budget adopted by the Board of Government.

FOR THE YEAR ENDING MARCH 19, 1952

	Account Number	*Expenditures*	*Receipts*
Office			
Secretary, Salary & Expense	(1)	$ 499.66	
Stationary, Printing & Postage	(2)	356.02	
Incidentals & Petty Cash	(3)	162.70	
Insurance & Treasurer's Bond	(4)	22.73	
Quarters, Rent, Lt. & Tel.	(7)	1880.23	$ 633.34
Office Secretary	(8)	2789.16	
Auditors & Custodian Charge	(9)	418.86	
Social Security	(10)	55.80	
Meetings			
Rent of Halls, etc.	(11)	150.00	
Stationery, Printing & Postage	(12)	51.00	
Hospitality Committee	(13)	837.61	672.53
Reporting & Stereopticon	(14,15)	10.00	
Annual Meeting (March, 1951)	(16)	987.40	809.25
Sections			
Sanitary Section	(21)	64.49	
Structural Section	(22)	49.23	
Transportation Section	(23)	14.81	
Hydraulics Section	(25)	9.69	
Surveying & Mapping Section	(26)	20.00	
Journal			
Editor's Salary & Expense	(31)	499.66	
Printing & Postage	(32)	5255.81	
Advertisements	(34)	166.50	2308.15
Sale of Journals & Reprints	(33,35)	———	1413.12
Library			
Periodicals	(43)	79.67	
Binding	(44)	107.45	
Fines	(45)	———	1.61
Badges for Members	(51)	19.00	19.00
Binding Journals for Members	(52)	52.00	52.31
Bank Charge	(53)	4.10	
		$ 14,563.58	$ 5,909.31
Miscellaneous	(54)	235.40	94.45
Engineering Societies Dues	(59)	735.59	
Dues from BSCE Members	(70)		6,643.00
Transfer Income Permanent Fund			2,887.31
Transfer from Current Cash			.50
		$ 15,534.57	$ 15,534.57

Entrance Fees to Permanent Fund $495.

53 New Members; 12 New Juniors; 8 Juniors transferred to members; 3 Students transferred to Juniors; 1 Associate Member transferred to Member.

The above receipts have been paid to the Treasurer, whose receipt the Secretary holds. The Secretary holds cash amounting to $30 included as payment under item 3 (Petty Cash) to be used as a fixed fund or cash on hand for change at dinners. $81.50 of withholding taxes and Social Security, which is payable to the Collector of Internal Revenue in April, 1952 is not included in the above tabulation.

ROBERT W. MOIR, *Secretary*

REPORT OF THE AUDITING COMMITTEE

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

We have reviewed the records and accounts of the Secretary and Treasurer of the Boston Society of Civil Engineers and we have compared the bank statement of securities held by the Boston Safe Deposit and Trust Company with the enumeration submitted by the Treasurer. We have also reviewed the reports of William J. Hyde, Certified Public Accountant, who has examined said records and accounts.

We have accepted and present herewith, with our approval, the signed report of the Accountant.

ALLEN J. BURDOIN
JOHN G. W. THOMAS

Boston, Mass., March 13, 1952

Mr. Alan J. Burdoin
Chairman of the Auditing Committee
Boston Society of Civil Engineers

Dear Sir:

In accordance with instructions. I have completed the annual audit of the financial records of the Society for the fiscal year March 1, 1951 to March 1, 1952 and report as follows:

All changes in securities owned were found properly recorded in the accounts. Receipts from sales of Rights were applied to reduce the cost of their respective stocks. All receipts of income, including entrance fees, as recorded in the records of the Secretary, together with the interest and dividends were found correctly recorded in the Treasurer's accounts and to have been deposited in the bank.

Earnings of Cooperative Banks were verified and found correct.

All bills paid have been approved by the President and Secretary. Payments of same were vouched by cancelled checks returned by the bank, with the exception of such checks as had not been presented for payment.

In accordance with past practice the Secretary's change fund, $30.00, is not included in the assets reported by the Treasurer. Withheld taxes and social security on employees' salaries for January and February have been retained for the Collector of Internal Revenue.

A verified copy of the Treasurer's report is attached hereto and summaries of his ledger accounts are shown in detail.

A transfer of $225.80 to "Cash Available for Investment" from "Current Fund Cash" leaves a balance in the latter of $1,274.20 instead of the usual $1,500.00. Necessary corrections have been made, the records are in good condition and, in my opinion, they are correct.

Respectfully submitted,

WILLIAM J. HYDE, *Certified Public Accountant*

REPORT OF THE EDITOR

Boston, Mass., March 1, 1952

To the Boston Society of Civil Engineers:

The JOURNAL was issued quarterly, in the months of April, July and October, 1951 and January, 1952 as authorized by the Board of Government on December 20, 1935.

During the year there have been published 18 papers presented at meetings of the Society and Sections. The Constitution and By-Laws are printed in the July issue and the table of Contents and Index for Volume 38 are included in the October issue.

The four issues of the JOURNAL contained 443 pages of papers and proceedings, 6 pages of Index and 50 pages of advertising, a total of 499 pages. An average of 1,325 copies per issue were printed.

The Board of Government voted on October 15, 1951 "that the Editor be instructed to cut the number of JOURNALS ordered from 1350 to 1300 and that JOURNALS to authors of papers be increased from 10 to 25 copies".

The cost of printing the JOURNAL was as follows:

Expenditures		
Composition and printing	$3,629.83	
Cuts	1,126.19	
Wrapping, mailing and postage	234.06	
Editor	499.66	
Advertising Solicitor	166.50	
Printing Constitution and By-Laws	265.73	
		$5,921.97
Receipts		
Receipts from sale of Journal and Reprints	$1,413.12	
Receipts from Advertising	2,308.15	
		$3,721.27
Net cost of Journal to be paid from Current Fund		$2,200.70

Respectfully submitted,

CHARLES E. KNOX, *Editor*

REPORT OF THE LIBRARY COMMITTEE

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

The Library Committee submits the following report for the past year.

The changes in the arrangement of the library due to alterations of the Society rooms were completed during the year. As authorized by the Board of Government the text books and handbooks which were considered obsolete were removed from the shelves and discarded.

The following expenditures were made during the year:

For subscriptions to periodicals	$ 79.67
For binding	107.45
For new books	80.41

Books loaned during the year totaled 99. A total of $1.61 was collected in fines. The following books were purchased and added to the library:

Manual of Water Quality and Treatment, 2nd Ed.
The Conservation of Ground Water, H. E. Thomas
Plane Table Mapping, Julian W. Lowe
Manual of British Water Supply
Route Surveys, Russell R. Shelton
Structural Shop Drafting Vol. 1, (AISCE)
Traffic Engineering Handbook
Foundations of Structures, Dunham
Handbook of Aereal Mapping and Photogrammetry
Hydrology—Physics of the Earth
Water Supply and Sewerage, Steel
Soils Engineering, Spangler
Applied Sedimentation, P. D. Trash
Legal Phases of Engineering, Crawford
Hydraulic Transients, George R. Rich

The following books were donated by the authors:

"Rigid Frame Formulas", A. Kleinlogel
"Hydrodynamics", Birkhoff

Most of the binding expense was in connection with the binding of Water Supply Papers and it is planned to continue this work until all volumes on hand are bound.

Respectfully submitted,

CHESTER J. GINDER, *Chairman*

REPORT OF THE HOSPITALITY COMMITTEE

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

The Hospitality Committee submits the following report for the year 1951-52.

Seven regular meetings, a Student Night Meeting, and an Annual Dinner were held during the year at the locations noted in the following table.

A Collation was served by this Committee in the Lounge of the American Academy of Arts & Sciences at the meetings as noted. The menu of these collations was the subject of considerable constructive criticism and it is suggested that during the following year a survey be taken to determine the items that would satisfy the majority of the members.

It has further been suggested that the Collation be eliminated as an unneeded expense. The cost per member attending is approximately 30¢ while contributions usually amount to less than 10¢ per member. It is suggested that further consideration be given this proposal. However, it is the opinion of this Committee that the social gathering adds considerably to the value and to the appeal of the meetings and although it is expensive it should be continued if at all possible.

The attendance at regular meetings at the Academy and the Society Rooms averaged 115, an increase of 50 over the previous year (an 80% increase).

SUMMARY OF MEETINGS AND ATTENDANCE

Date	*Speaker*	*Place*	*Attendance*	
			Dinner	*Meeting*
Mar. 21, 1951	Thomas R. Camp	Hotel Vendome	235	71
Apr. 25, 1951	John B. Wilbur, Ed C. Keane	Ninety-nine Club	89	89
May 16, 1951	George L. Wey	American Academy		51*
Sept. 26, 1951	Wilfred M. Hall	American Academy		110*
Oct. 17, 1951	D. B. Steinman	Northeastern Univ.	227	405
Nov. 19, 1951	E. B. Myott, O. Singstad, E. H. Prager	American Academy		156*
Dec. 12, 1951	K. Terzaghi	Society Rooms		124
Jan. 30, 1952	R. DeSimmone	American Academy		120*
Feb. 20, 1952	Gen. Schwindler	American Academy		132*

*Collation served after meeting

The average attendance at all nine meetings was 140.

Respectfully submitted,

RALPH S. ARCHIBALD, *Chairman*

REPORT OF SUBSOILS COMMITTEE

Boston, Mass., March 17, 1952

To the Boston Society of Civil Engineers:

During the past year the Committee on Subsoils of Boston has continued work on its program of preparing boring data for publication.

The Committee Report "Boring Data from Greater Boston, Section 3: South Boston and East Boston", containing 58 pages and two large maps, appeared in the October 1951 issue of the JOURNAL of the Society. Work on other sections is in progress.

On February 18, 1952, the Board of Government voted that the Committee be authorized to proceed with the examination and collection of papers suitable for a publication that would be entitled "Contributions to Soil Mechanics, 1941 to 1952". In a report on this assignment, recently submitted to the Board of Government, the Committee has recommended the publication of this volume and the reprinting of the Society publication "Contributions to Soil Mechanics, 1925 to 1940".

Respectfully submitted,

DONALD W. TAYLOR, *Chairman*

REPORT OF MEMBERSHIP COMMITTEE

Boston, Mass., March 17, 1952

To the Boston Society of Civil Engineers:

The Membership Committee of the Society has held no formal meetings during the year, but has activated the Membership Campaign by the distribution of specially prepared brochures to approximately 1400 names and addresses, comprised of recent graduates in civil and sanitary engineering from engineering schools in the New England area, and to members of the American Society of Civil Engineers who are not members of the Boston Society of Civil Engineers.

The Chairman of the Committee has also taken action to acquire the names and addresses of professional engineers already licensed in Massachusetts, for the purpose of forwarding brochures to those licensed engineers who do not appear to be members of the Society.

It is estimated that the membership of the Society now stands at 852, and that during the year the new membership appears as follows:

53 New Members	8 Juniors Transferred to Members
12 New Juniors	3 Students Transferred to Juniors

1 Associate Transferred to Member

The Membership Committee is credited with 21 applications as a direct result of its Membership Drive. Further, the Membership Committee believes that the Drive has indirectly effected an increase in the new membership.

Considerable literature, printed under the name of the present Membership Committee, is available for further distribution. In view of this, it is recommended that the Membership Committee as now constituted be continued for one more year.

Respectfully submitted,

THOMAS A. BERRIGAN, *Chairman*

REPORT OF "COMMITTEE ON COMPETITIVE BIDDING FOR ENGINEERING SERVICES"

Boston, Mass., March 19, 1952

To the Boston Society of Civil Engineers:

At the time of our Committee's 1951 Report to the Annual Meeting of Boston Society of Civil Engineers, the Committee reported that a bill sponsored by the Committee members was before the State Legislature, and awaiting action by the Legislative Committee on State Administration.

In due course of time the bill was favorably reported by the Legislative Committee but with some opposition. On April 10, 1951, the bill came before the House of Representatives for action; it failed to pass and was referred to the 1952 Annual Session of the Legislature. In order for the bill to be considered by the 1952 Session of the Legislature, it would be necessary to again file the bill.

Our Committee considered the advisability of filing the bill for action in 1952 and after discussion with some members of the Legislature who had been favorable to the bill, it was decided not to file the bill for action in 1952. There is no probability of the bill being passed so long as some of its opponents are still members of the Legislature.

A copy of the latest draft of the bill is attached.

It is recommended that our Committee be discharged.

Respectfully submitted,

RALPH W. HORNE, *Chairman*

REPORT OF ADVERTISING COMMITTEE

Boston, Mass., March 4, 1952

To the Boston Society of Civil Engineers:

The following advertising has been carried in the JOURNAL during the year.

Issue	April	July	October	Jan. (1952)
Professional cards	36	36	36	34
¼-page ads	24	24	24	23
½-page ads	—	—	—	—
Full-page ads	1	—	—	1
Back cover	1	1	1	1
Total pages of advertising	11	10	10	10
Theoretical gross revenue	$572.00	$520.75	$520.75	$660.25

Over the past year $2,308.15 has been collected from advertisers and commissions totaling $166.50 have been paid to our former solicitor.

On May 7, the Committee voted to terminate the appointment of Mr. William Stowe as solicitor for the JOURNAL advertising. This action was prompted by his failure to obtain adequate results.

Two candidates for appointment as solicitor were interviewed but after investigating the possible income neither was interested in undertaking the work. There has been only limited solicitation for new advertisements during the past

year, and as a result there has been little change in the amount of advertising carried.

With the permission of the Board letters were sent to a number of organizations offering the privilege of publishing a two-page article in the section entitled "Of General Interest" as an incentive for placing a full-page advertisement in the JOURNAL at a cost of $125. The article would be suitably marked to indicate it was advertising. One organization accepted this offer, however, their article contained no advertising and at their request was published as a paper. The Committee feels that this type of solicitation might be pressed to advantage.

The Committee suggested to the Board that an advertisement for a solicitor be placed in the Bulletin of the ESNE. Secretary Moir placed such an advertisement in the January 7, 1952 issue. To our knowledge, no inquiries regarding this position have been received.

A system of direct mail solicitation has been studied but the Committee has been unable to agree on a proposal which it could recommend to the Board.

Consideration has been given to requesting certain members of the Society to do a limited amount of solicitation either for, or without a commission at their option. This field is limited since we feel that it is not exactly ethical for an engineer responsible for making decisions regarding materials or services to solicit advertisements from the suppliers. There are a few members such as teachers or some of the younger engineers who are not charged with such responsibility who might be willing to do a limited amount of soliciting.

The Committee wishes to express its appreciation to all those who have supported the JOURNAL through advertising.

Respectfully submitted,

EDWIN B. COBB, *Chairman*

REPORT OF JOHN R. FREEMAN FUND COMMITTEE

Boston, Mass., March 10, 1952

To the Boston Society of Civil Engineers:

There was no activity of the Fund during the past year.

Of the 150 copies of the Freeman Fund Hydraulic Lectures, all but three have been disposed of. Four of these were sold; the balance were given to members. There was a suggestion that these should be published, but the Committee voted against this. It voted to advise the Board of Government that the Committee had no objection to the authors' publishing these papers.

At a meeting held February 19th the Committee voted to recommend to the Board of Government that Professor Leslie J. Hooper of the Worcester Polytechnic Institute be made a member of the Committee. This appointment was approved by the Board of Government.

There has been no activity considered as yet for the coming year.

Respectfully submitted,

HOWARD M. TURNER, *Chairman*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SANITARY SECTION

Boston, Mass., March 1, 1952

To the Sanitary Section, Boston Society of Civil Engineers:

During the year 4 meetings of the Sanitary Section were held as follows:

February 28, 1951—Annual meeting and election of officers at Society Rooms. The following Executive Committee was elected for the year:

Prof. William E. Stanley, Chairman; F. M. Cahaly, Vice-Chairman; A. C. Bolde, Clerk; Thomas A. Berrigan, John S. Bethel, Jr., and Frank L. Heaney.

There was a symposium held on Sludge Digestion with the following panel:

Joseph McCarthy, Moderator; L. E. Langdon, John G. McDonald, Prof. Rolf Eliassen, Prof. Edward W. Moore and R. S. Rankin. Attendance 75.

June 2, 1951—Joint outing with Main Society. Inspection trip to the new South Metropolitan District Sewage Treatment plant at Nut Island, Quincy, Mass. Attendance 45.

October 3, 1951—Meeting at Society Rooms. Speaker for the evening was Charles Y. Hitchcock, Jr., whose subject was sewage disposal plants visited in England and Holland. Attendance 35.

December 4, 1951—Meeting at Society Rooms. Speaker was Mr. John R. Szymanski, Superintendent of the New Britain, Connecticut Sewage Treatment plant on the subject "Growing Pains of a Sewage Treatment Plant". Attendance 31.

The total attendance for the four meetings was 186. The average attendance for the four meetings was 46.5. Five meetings of the Executive Committee were held during the year.

Respectfully submitted,

A. C. Bolde, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE STRUCTURAL SECTION

Boston, Mass., March 19, 1952

To the Structural Section, Boston Society of Civil Engineers:

During the past year, seven meetings were held as follows:

March 14, 1951.—Annual meeting and election of officers. Mr. T. R. Higgins of the American Institute of Steel Construction spoke on "Bolted Structural Joints with Respect to High-Strength Bolts." Attendance—46.

April 11, 1951—Dr. Ruth D. Terzaghi and Mr. Herman G. Protze spoke on "Recent Progress in Testing and Examining Aggregate and Other Concrete Constituents." Attendance—46.

October 10, 1951.—Joint meeting with Transportation Section. Mr. Eric Reeves of Maguire-Fay Spofford spoke on "Structural Design Problems on the John F. Fitzgerald Expressway." Attendance—76.

November 14, 1951.—Prof. Harl P. Aldrich, Jr., spoke on "Importance of the Net Load to the Settlement of Buildings in Boston." Attendance—73.

December 12, 1951—Joint meeting with Main Society. Dr. Karl Terzaghi spoke on "Permafrost." Attendance—124.

January 9, 1952.—Prof. Albert G. Dietz spoke on "Japanese Earthquake Resistant Construction." Attendance—25.

February 11, 1952.—Joint meeting with American Welding Society. Mr. Van Rensselaer P. Saxe spoke on "How to Save 30 Percent on Structural Steel." Attendance—25 members of Structural Section.

The total attendance was 415; average attendance 59.

A meeting of the Executive Committee of the Structural Section was held on November 14, 1951.

Respectfully submitted,

CHARLES H. NORRIS, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SURVEYING AND MAPPING SECTION

Boston, Mass., January 31, 1952

To the Surveying and Mapping Section, Boston Society of Civil Engineers:

The following meetings were held during the past year:

April 4, 1951.—Mr. Gordon E. Ainsworth spoke on the Method and Procedures of Pipe-line Surveying. Attendance—60.

October 24, 1951.—The speaker for this meeting was Mr. H. Lowell Crocker. His subject was "Location Planning and Land Takings for the John F. Fitzgerald Expressway." Attendance—54.

January 16, 1952.—Annual meeting with the election of officers. The talk for this meeting was entitled "The Hoosac Tunnel." The three speakers of the evening were: Edward H. Cameron, Douglas K. Merrill and John J. Vertic. Attendance—37.

The total attendance was 151; average attendance 50.

Respectfully submitted,

GEORGE W. HANKINSON, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE HYDRAULICS SECTION

Boston, Mass., March 5, 1952

To the Hydraulics Section, Boston Society of Civil Engineers:

The following meetings were held during the past year:

May 2, 1951.—Mr. Nathaniel Clapp spoke on the subject "Hydraulic Features of the Cleveland Brook Reservoir Project." Attendance—34.

November 7, 1951.—Professor Henry M. Paynter spoke on "Unsteady Flow Studies," and Professor Giuseppe Evangelisti of the University of Bologna spoke on "Recent Unsteady Flow Studies in America and Europe." Attendance—40.

February 20, 1952.—Joint meeting with the Main Society, and annual meeting and election of officers. Brigadier-General Don G. Shingler spoke on "The Missouri River Project." Attendance—131.

The total attendance was 205; average attendance 68.

Respectfully submitted,

L. W. RYDER, *Clerk*

PROFESSIONAL SERVICES AND ADVERTISEMENTS

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

Volume 39 APRIL, 1952 Number 2

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROFESSIONAL SERVICES

INDEX TO ADVERTISERS

COMMITTEES

1952-1953

NOMINATING COMMITTEE

Past Presidents (Members of the Committee)

George A. Sampson
Leslie J. Hooper
Byron O. McCoy
Lincoln W. Ryder
(Term expires March 1953)

Harvey B. Kinnison
Frederic N. Weaver
Charles E. Knox
Harold A. Thomas, Jr.
Oliver G. Julian
(Term expires March 1954)

PROGRAM

Emil A. Gramstorff, *Chairman, ex-officio*

Charles T. Main, 2nd.
William D. Henderson
Harold A. Thomas, Jr.
Scott Keith
Rolf Eliassen
Robert W. Moir
Fozi M. Cahaly
H. Lowell Crocker
George W. Hankinson
Edward C. Keane
Byron O. McCoy

PUBLICATION

Charles E. Knox, *Chairman*

John B. Babcock, 3rd.
Miles N. Clair
George C. Houser
Byron O. McCoy
George W. Hankinson
H. Lowell Crocker
Fozi M. Cahaly
Edward C. Keane

LIBRARY

James W. Daily, *Chairman*

John F. Flaherty
Lincoln W. Ryder
Thomas C. Coleman
Clifford S. Mansfield
John J. Vertic
Leonard J. A. Fenocketti
Ariel A. Thomas
George C. Douglas

HOSPITALITY

Stephen E. Dore, *Chairman*

Carney M. Terzian
Edmond H. Brown

DESMOND FITZGERALD AWARD

George R. Rich, *Chairman*

Charles H. Norris
Alexander J. Bone

SANITARY SECTION AWARD

George R. Rich, *Chairman*

Stuart E. Coburn
Edward W. Moore

STRUCTURAL SECTION AWARD

Charles H. Norris, *Chairman*

Frederic N. Weaver
William F. Ryan

HYDRAULICS SECTION AWARD

George R. Rich, *Chairman*

Scott Keith
Allan T. Gifford

SURVEYING & MAPPING SECTION AWARD

Alexander J. Bone, *Chairman*

Harry R. Feldman
George W. Hankinson

TRANSPORTATION SECTION AWARD

Alexander J. Bone, *Chairman*

Edward C. Keane
Herman J. Shea

SUBSOILS OF BOSTON

Donald W. Taylor, *Chairman*

Miles N. Clair
Irving B. Crosby
Lawrence G. Ropes
Chester J. Ginder

INVESTMENT COMMITTEE

Charles O. Baird, Jr., *Chairman*

Emil A. Gramstorff
J. Stuart Crandall

MEMBERSHIP CENTRAL COMMITTEE

Thomas A. Berrigan, *Chairman*

Albert A. Adelman
John B. Babcock, 3rd.
James F. Brittain
Elliot F. Childs
Arthur E. Harding
Frank L. Heaney
Oliver G. Julian
Murray H. Mellish
Walter E. Merrill
Lincoln W. Ryder
Herman J. Shea
Ernest L. Spencer

Robert W. Moir, *ex-officio*

ADVERTISING COMMITTEE

George W. Hankinson, *Chairman*

T. William Lamb
Charles E. Knox
Robert W. Moir
Ralph S. Archibald

PUBLICITY COMMITTEE

Augustine L. Delaney, *Chairman*

Gardner K. Wood
Ernest L. Spencer
Clifford S. Mansfield
Joseph C. Lawler

AUDITING COMMITTEE

John G. W. Thomas
James F. Brittain

COMMITTEE ON REGISTRATION LAW

Thomas R. Camp, *Chairman*

John B. Wilbur
Arthur L. Shaw
Frank M. Carhart
George R. Rich
Ralph W. Horne

COMMITTEE ON PROFESSIONAL PRACTICE AS RELATING TO COMPETITIVE BIDDING

Carroll A. Farwell, *Chairman*

Oliver G. Julian
Harvey B. Kinnison