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OF GENERAL INTEREST

ANNUAL REPORTS

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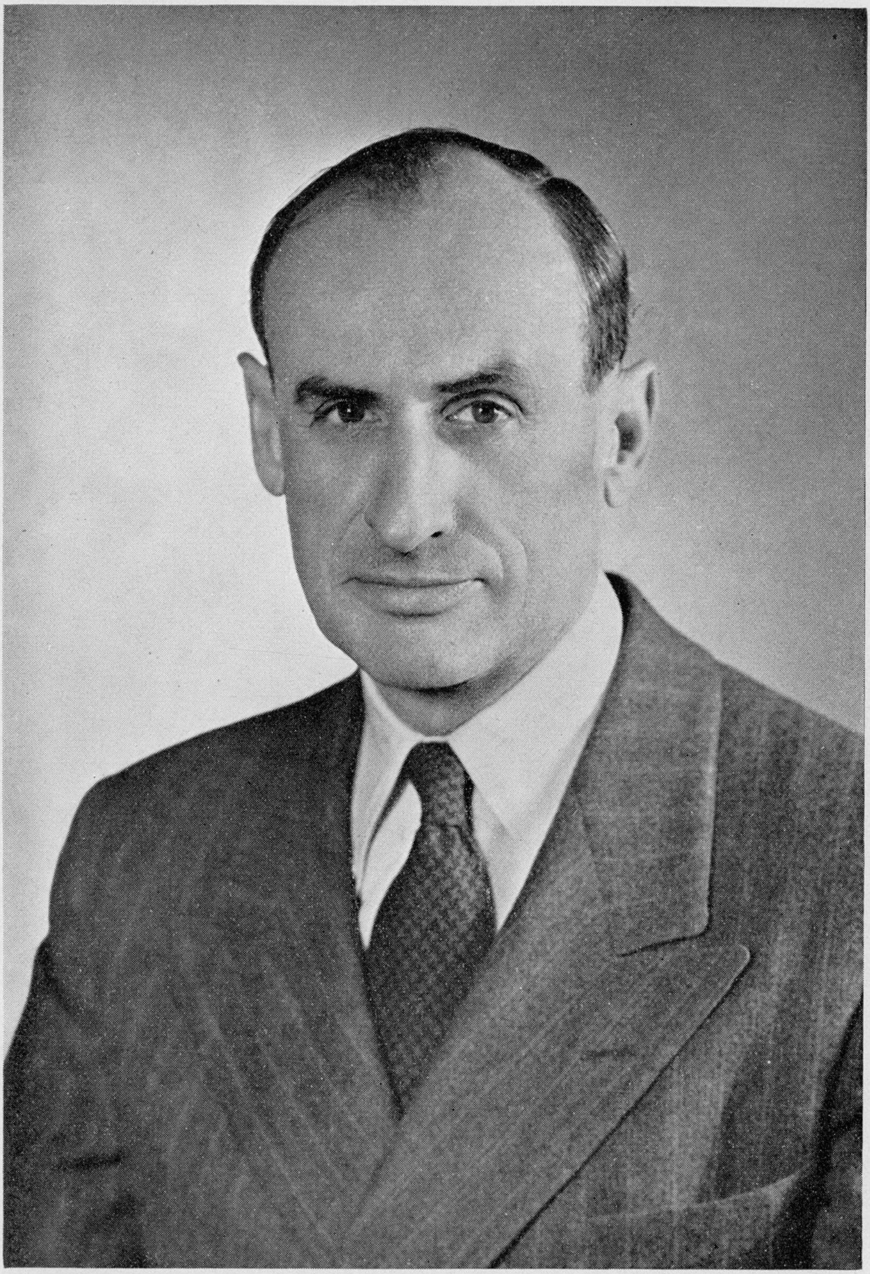

HARRISON P. EDDY, JR.
President
Boston Society of Civil Engineers
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THE SECOND HUNDRED YEARS

PRESIDENTIAL ADDRESS BY FREDERIC N. WEAVER*

(Boston Society of Civil Engineers, March 16, 1949.)

A TIME for stock-taking comes at intervals in the life of every organization; and once in a while even a technical society should consider itself and its surroundings, survey its past, appraise its present, and perhaps speculate as to its future. The end of the first hundred years seems to be a good time for us to do this, and if the investigation leads us farther afield than the ordinary bounds of our activities, that is to be expected; for engineers are accustomed to follow investigations no matter where they lead. The historical phases of this stock-taking, by the way, have been done and done excellently by Professors Babcock, Breed, Fair, Spofford and Turner, and are published in the Centennial Number of the Journal—a book for which we have to thank especially the ability, skill and energy of John B. Babcock, III.

As to the present and the immediate future, I am tempted to analyze the present condition of the Society and to forecast probable trends. But such a study seems unnecessary. I could warn the Society against becoming over-specialized, becoming too complicated, getting too tangled in the red tape of routine procedures, growing too self-satisfied, and failing to keep abreast of the times. I could advocate closer relations with the Northeastern Section of the American Society of Civil Engineers, inducing young men to join the Society and getting them into positions of responsibility, making the meetings and the Journal of such compelling interest that the membership

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problem would take care of itself, urging the Society and its members to take an interest in public affairs while keeping out of politics, and perhaps urging, as I shall do anyway, that the Society and its members take an interest in world affairs.

But all these points are self-evident. No persuasion is needed. We all agree. As far as I can see, the Society will continue to grow and to function and to fill a needed place in the life of the community. On the other hand, the life of this Society is bound up with the life of the nation, and the life of the nation is bound up with the life of the world. If the nation should perish, the Society automatically would perish; and, while nobody knows anything about the future, it is interesting to note present trends which might affect the future, and to speculate upon ways and means of preserving the framework in which we live and operate.

We might discuss the problem of survival itself, and, although few of us will be here to see for ourselves, ponder about the situation as it might well be at the end of the next hundred years. I have an idea that the progress ratio between then and now will not be much greater than the ratio of progress between now and 1848 when this Society was formed. At that time the United States was a young, dynamic, growing nation, expanding, pushing back its frontiers, finding the strength that shortly would be tested in the great conflict of the Civil War; but today we are still a young and vigorous nation, and one hundred years hence, on the scale that measures the life of nations, we shall be a young nation. The only difficulty is to last another hundred years.

Edward Gibbon says, "All that is human must retrograde if it do not advance."[1] This dictum seems to have applied also to some animal and insect societies. For example, the great reptile rulers of the Mesozoic era evolved and specialized until they were unable to meet changing conditions, and so they perished. On the other hand the lowly cockroach has lived unchanged for millions of years, and the societies of the ants have been static for untold ages. What lesson there is in this for the Boston Society of Civil Engineers I am not sure, but suspect that it is a negative one.

Still, it is a point to be considered. Are we in the United States approaching a static condition? On the one hand we note the active inventive spirit shown by physicists and scientists, the advancements

[1]Edward Gibbon, "The History of the Decline and Fall of the Roman Empire".

in engineering and all forms of knowledge, the organization of business, industry and manufacturing, and the spread of education; but, on the other hand we note the increasing desire for security, proposals for socialized medicine, a tendency toward state control of education, increased old-age benefits and veterans' benefits beyond the capacity of the country to pay, the thousands of workers already resembling ants, men and women in huge factories, working at uninspired tasks, thinking only of how they can get a little more than their share, with little sense of responsibility and with slow growth in maturity. We have left behind forever the savage laissez-faire days of the 18th and 19th centuries and have gone quite a way toward socialism, a condition in which the state takes care of its own; but we note with concern the increasing number of loafers, people who do not pull their own weight, those dishonestly on relief, those taking vacations in Florida at the expense of workers in other parts of the country, those who prefer to loaf on a dole rather than to work for more money, and those who can get more on relief than they can by working.

This trend, kept up, would lead to a static condition. Men do their best, according to one viewpoint, when relieved of financial worries; but, on the other hand, most men do not work unless forced to by some kind of spur. The spur may be the desire for financial gain, the desire for achievement, recognition by others, or the desire for power, but, except in the case of geniuses, some spur is needed. Will there be enough incentive in the socialized state, and will such a condition provide more fertile ground for a dictatorship, or will it provide ground in which dictatorship cannot flourish? These are some of the questions to think about as we swing toward the socialized state, and there is no doubt but that we are swinging that way. The attempts of each clique and bloc, whether composed of farmers, organized labor, or big business, to get all that it can for its own members at the expense of the general welfare is a general phenomenon present in all ages and under all forms of government. And, in parenthesis, I shall risk being accused of favoring my own bloc to say that I think that engineers, from the executives down to the draftsmen, should be better paid. Theirs is important work, and their education, training, experience, ability, honesty, and social value should be recognized.

The long-term worry might be deterioration, but the short-term worry is war. Writers and observers would have us believe that the two different ways of thought of Russia and the United States must needs come into conflict. Back in the days of ancient Greece it seemed inevitable that the nations of Athens and Sparta would grapple in a death struggle, as they did—in the end destroying each other and paving the way for Rome to take over. But now, as we look at the map, and note the size of the entire Greek peninsula, the whole thing about the size of a pocket handkerchief, the idea of war between those two states seems silly. In 1785 the idea that the Thirteen Colonies could ever unite under a federal government was silly, but they managed to do it. Today, from the viewpoint of a man from Mars, the idea of war between Russia and the United States is just as silly as was the war between Athens and Sparta; and the idea of a central government, the United Nations, no more silly than the idea of a central government for the Colonies. Analogies prove nothing, but they do indicate in this case that a certain amount of sanity and self-restraint can postpone disaster, and, in the end, avert it.

A nation of sound, intelligent, patriotic citizens blessed with wise leadership need have no fears either of internal or external troubles. That is an axiom. As to leadership, in this day and age when the bursting of atomic bombs may signal the opening of hostilities, we need more and more the right kind of leadership, the kind that will keep that sort of thing from happening. Especially in democracies there is no assurance that a Lincoln or a Churchill will be available at the right moment, and under our American system of elections there is no assurance that even a moderately capable man will be at the helm. But if a body of citizens, composed of military officers, engineers and scientists, were to map out a scheme for defense, perhaps that action would save the nation in a critical period. I might say that twenty-five years ago I was all for disarmament, but events since then have convinced me that we should keep in mind the old slogan, "Trust in God and keep your powder dry."

I know that we in this country are of two minds concerning the proposed Atlantic Pact. Some people think that if Germany in 1914 had known that the United States would have acted the moment the frontiers of Belgium were crossed, and that later if Japan and Germany had known that the United States would react in the way that she did, two world wars would not have occurred; and that now, if Uncle Joe knew that any act of aggression on his part meant instant action by the United States, there would be no act of aggression.

Others feel that if we invite trouble we shall get trouble. This is a big question to which at the moment there is no definite answer. I imagine that we would not sit still very long while atomic bombs rained upon our cities, but I wonder if we could not do something to prevent this from happening.

Thus far this has been a rather gloomy picture, an uncertain and divided electorate, chiselers and cheaters riding along at the expense of others, and grafters and crooked politicians taking their cut in every public enterprise. Yet we must not make the mistake of thinking that these days are degenerate, that our ethics and morals and patriotism have gone to ruin. There were slackers when the Pilgrims were trying to get a foothold in the wilderness; there were Tories, even traitors, when this country was trying to gain its independence. In ancient Rome there was a class of loafers so powerful that even emperors solicited their support by means of food and entertainment —bread and circuses. When the Romans took Carthage they found many of its leading citizens crucified to the walls of their homes. It appeared that these men had sought to keep their wealth at a time when their wealth might have saved the city—hence the punishment. And still farther back in the past we find Isaiah thundering, "Because the daughters of Zion are haughty, and walk with stretched forth necks and wanton eyes, walking and mincing as they go, and making a tinkling with their feet . . . the Lord will take away the bravery of their tinkling ornaments about their feet. . . ."[2] And so it has gone, I imagine, since men first gathered together for protection. The good old days!

Still, there are periods in the life of every nation when the composite moral fibre is stronger than at other times. I think that the general situation is better now than it was some seventy-five years ago; and of course the American system whereby certain crooked politicians take from ten to fifteen per cent is better than the Chinese system where they take it all.

I think that the majority of our citizens are still sound. They expect to work for a living and to contribute their share, but they expect to be given a chance to work for a living. They do not seem to be jealous even of those men who, by virtue of being able to bat a ball over a fence or to beat other men into submission with their fists, make as much money as the President of the United States.

[2]Isaiah 3:16.

I think that they feel that a little of the star dust from fairy land still falls into this country and helps some people more than others. Or perhaps they are philosophical and realize that people pay more to be amused than they do to be instructed or to be given water supplies, bridges, automobiles and toasters.

Our trial balance seems to be on the gloomy side, for it appears that all our troubles spring from selfishness, and that our salvation lies in the application of the Golden Rule. If that be the case the future is dark indeed, for the only time that a nation can unite and work for the common good is in times of crisis such as war, and even then there are profiteers. But not every individual has to be sound in order to have a sound nation. If ten righteous men had been found in Sodom the city would not have been destroyed.[3] A sufficient number of intelligent and responsible citizens will insure the safety and prosperity of our own country.

President Truman said on March 8 at Rollins College, Florida: "Never before has this country needed as it does today the leadership of thoroughly trained men and women." I wonder if the engineers could not provide a nucleus of this leadership? And I do not mean only the civil engineers. I mean all engineers; yes, and scientists and business men and members of labor unions—wherever we have men of good will working together to make the country and the world better. Let the engineers transfer some of their knowledge and skill into fields not so narrowly technical. Let them help to eliminate poverty and want, to get rid of the conditions that breed war and class struggle. Let them try to put the country in such shape that it can meet all emergencies, and by voice and example try to induce a frame of mind where dictators can never get a chance to rule, and where initiative and ability are recompensed and honored. And each man might start when he sits down with his conscience and his income tax blank.

Therefore we shall try to keep our own sanity, and we shall try to influence others. In this way we can achieve the breathing space which can turn into an indefinite armistice; and then turn the armistice into a sort of uneasy peace, and then turn the uneasy peace into permanent peace in which each country can work out its own problems.

We have not made much progress this afternoon in forecasting the future. Perhaps accurate forecasting is unnecessary as well as

[3]Genesis 18:32.

impossible. We live one day at a time, and perhaps it is not later than we think. I think that this country is not decrepit, ready to throw in the sponge, but that, using its resources of technical knowledge, industrial organization, intelligence and wisdom, it is on the verge of a new prosperity; and, furthermore, that it will weather the present storm. As engineers, supposedly trained in sound thinking, we can help by keeping abreast of situations as they arise, by judging from facts and not by emotions, by supporting every proposal that seems sane and logical, and by helping as far as we can as individuals and through our engineering organizations.

CONCRETE DETERIORATION DUE TO CARBONIC ACID

By Ruth D. Terzaghi, Member

(Presented at a meeting of the Boston Society of Civil Engineers, held on February 18, 1948.)

Abstract

In many regions, ground water and/or surface water, such as that of streams, lakes, or marine estuaries, contains a considerable quantity of free carbon dioxide. Under certain circumstances, a part of this, known as "aggressive carbon dioxide," can attack concrete. The ratio of aggressive carbon dioxide to total carbon dioxide varies with the concentration of carbon dioxide, calcium bicarbonate and other dissolved substances. For a given concentration of carbon dioxide and calcium bicarbonate, the ratio is much greater for sea water than for fresh water. Experience shows that attack is likely to be sufficiently rapid to require consideration if the concentration of aggressive carbon dioxide exceeds about 20 ppm. The rate of attack and the severity of deterioration is increased by the presence of other deleterious substances, such as sulfates. Attack may be prevented by treating the water, for instance by allowing it to react with calcium carbonate, before it comes in contact with concrete; it may also be prevented by maintaining an impervious coating on all surfaces exposed to deleterious water. If sulfates as well as aggressive carbon dioxide are present, the use of sulfate-resisting cement is recommended.

Introduction

The deterioration of concrete exposed to natural destructive agents indicates the need to establish criteria by means of which the existence of such agents can be recognized and the severity of their effect on concrete predicted. A knowledge of such criteria would enable the engineer either to provide protection against the anticipated attack or to avoid the use of concrete altogether wherever attack is likely to be exceptionally severe.

One of the destructive agents for which such criteria are particularly needed is carbon dioxide (CO_2). Unless the preliminary investigation of the construction site is unusually thorough, the presence of CO_2 in water in contact with concrete is likely to remain unnoticed until it has inflicted conspicuous damage on the concrete. Even when its presence is known in advance of construction, it is not invariably possible, at the present state of our knowledge, to forecast accurately its effect on concrete. However, the general principles involved in the deterioration of concrete due to exposure to carbonic acid are fairly well understood, and a considerable amount of empirical data has been accumulated regarding the process. These are of great value as a guide in applying the general principles to specific cases and more are urgently needed.

This article contains a summary of available information regarding the occurrence of carbonic acid in natural waters, the conditions under which it can attack concrete, and its effects on concrete.

Occurrence of Carbon Dioxide

A discussion of carbonic acid presented to this Society should logically begin with an account of its occurrence in New England. Unfortunately, very little information regarding this topic is available. However, fragmentary data regarding some incompletely diagnosed cases of concrete deterioration in this part of the country suggest that disintegration was the result, at least in part, of carbonic acid attack. Systematic investigations may reveal the presence of a notable concentration of carbonic acid in the water in some localities in New England. In this connection, it is of interest to note that investigations carried out in Germany, mainly between 1910 and 1920, indicated that concrete deterioration due to carbonic acid was widespread. Partly as a result of these discoveries, the German railroads issued a regulation requiring analysis of water at the site of all concrete structures, prior to construction, in order to detect the presence of dangerously large concentrations of carbonic acid or other deleterious substances.

Water containing a high concentration of carbonic acid occurs under a variety of conditions, and it is seldom possible to predict with assurance whether or not it will be encountered in a given locality. It is, however, very commonly found under certain conditions, described below.

Ground water in regions of volcanic activity, as well as in regions where there has been volcanic activity in the geologically recent past, may contain a high concentration of carbonic acid because carbon dioxide is, next to water, the most abundant constituent of volcanic exhalations. Analyses indicate that carbon dioxide may constitute over 5 per cent by weight of vapors of volcanic origin (Keller and Valduga, 1946). As the vapors condense and become mixed with meteoric water, the concentration of free carbon dioxide is likely to be reduced, owing to escape of gaseous carbon dioxide, as well as to dilution and to reaction with the country rock. Hence, even in volcanic regions, the concentration of free carbon dioxide in ground water is not as high as it is in steam of volcanic origin. Nevertheless, ground water with a carbon dioxide concentration in excess of 2 g per liter (2,000 ppm) has been reported from a region of geologically recent volcanic activity (Wagner, 1898).

Much of the carbonic acid which finds its way from the depths of the earth into the atmosphere is taken up by plants, and is later released during the decomposition of the plant remains. Carbonic acid is therefore likely to be present wherever organic matter accumulates and decays, as in forests, marshes, some ponds and some protected embayments of lakes or oceans.

A high concentration of gases in standing water may be indicated by the evolution of gas bubbles, which commonly consist of a mixture containing carbon dioxide, methane, and possibly hydrogen sulfide. Such bubbles may, however, easily escape observation if the surface of the water is not perfectly calm.

The concentration of carbonic acid in a body of standing water is likely to be considerably higher near the bottom than at the surface, particularly if the bottom is covered by a deposit of decaying organic matter. These differences may commonly be detected by measuring the pH[1] of samples of water taken from different depths. Other things being equal, this value decreases with increasing concentration of free carbonic acid. As an example of the variation of pH with depth, measurements made in New York Harbor may be mentioned (Atwood and Johnson, 1924, Figs. 70 and 73). These showed that the pH of samples taken from the bottom in certain localities was at times nearly one unit lower than that of samples taken near the surface in the same place.

[1]The symbol pH denotes the common logarithm of the reciprocal of the hydrogen ion concentration.

Variations in carbon dioxide content with depth are due to several factors. Foremost among these is the fact that carbon dioxide is generally produced at the bottom as the result of the decomposition of organic matter which has accumulated there. It reaches the top only by upward migration. Furthermore, the solubility of gases in water increases with increasing pressure, and their solubility is therefore greater at the bottom than near the surface of a body of water. The solubility also increases with decreasing temperature, and during the warm seasons, the solubility of carbon dioxide is therefore generally highest at the bottom of a body of water, where the temperature is lowest.

Seasonal variations in the carbon dioxide content of surface waters (streams, lakes, and bays) may also be notable. They may be due to variations in the rate at which carbon dioxide is produced, or they may be caused by convective stirring of bodies of standing water which takes place in the autumn and tends to equalize the carbon dioxide content of top and bottom layers of water.

The acidity of river water subject to industrial pollution may vary conspicuously with the activity in the plants concerned. In a study of the water of the Housatonic River at Shelton, Connecticut, for instance, it was found that 90 per cent of the samples taken on weekdays had an acid reaction, whereas only 67 per cent of those taken on a Saturday or a Sunday had an acid reaction (Binger, 1927).

Chemistry of Attack of Carbonic Acid on Concrete

Equilibrium Between Carbonic Acid and Its Calcium Salts

Carbon dioxide occurs in three different compounds in natural waters. One of these is carbonic acid (H_2CO_3), produced when carbon dioxide is dissolved in water. This form is commonly called "free carbon dioxide." Carbon dioxide is also contained in calcium carbonate, a very slightly soluble compound present in minute quantities in practically all natural waters, and in calcium bicarbonate, a relatively soluble compound which occurs in varying concentrations in natural water.[2]

Calcium bicarbonate can exist in aqueous solution only in the presence of carbonic acid. Unless there is a sufficient quantity of

[2]One-half of the carbon dioxide contained in dissolved bicarbonates is called "half-bound carbon dioxide" by some authors, because it is readily driven off by boiling the solution. In the older literature, it has also been referred to as "free carbon dioxide," a term now reserved for carbon dioxide which has not reacted with any substance other than water.

this acid in solution, calcium bicarbonate decomposes to form carbonic acid and calcium carbonate. As a result of this reaction, calcium carbonate is precipitated. The quantity of carbonic acid (free carbon dioxide) required to prevent such decomposition increases with increasing calcium bicarbonate concentration and in general decreases with the total concentration of dissolved substances other than calcium compounds. The relation between calcium bicarbonate concentration in different natural waters and solutions and the minimum concentration of carbonic acid required to prevent its decomposition is represented by the curves show in Figure 1. Such curves will be

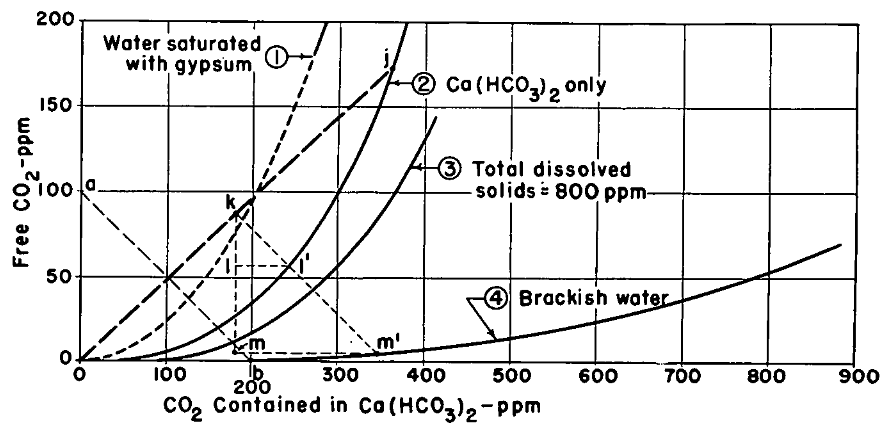

FIG. 1 - RELATION BETWEEN FREE CARBON DIOXIDE AND CALCIUM BICARBONATE CONCENTRATION

referred to as carbonic acid-calcium bicarbonate equilibrium curves.

Curve 1 in Figure 1 refers to a solution saturated with respect to gypsum ($CaSO_4 \cdot 2H_2O$). The plain part of the curve is based on observed values (Frear and Johnston, 1929) and the dashed section represents an interpolation between these observed values and the origin. This part of the curve, which is based on a very rough approximation, is included merely to illustrate the wide variations in solubility of calcium bicarbonate with variations in the concentration and nature of other dissolved substances. As this curve indicates, the concentration of carbonic acid required to stabilize calcium bicarbonate is increased by the presence of another calcium salt. Curve 2 refers to water containing no dissolved substances other than carbonic acid and calcium bicarbonate. It is based on the results of experimental work of Frear and Johnston (1929) which differ only slightly from data previously published by Tillmans and Heublein (1912) and others. Curve 3 is an approximate representation of equilibrium conditions in a hypothetical water containing 800 ppm total dissolved solids, including calcium bicarbonate which is assumed to be the only calcium compound present. It is based on values computed by means of constants given by MacInnes and Belcher (1933) and by Langelier (1936). Like curve 1, it is intended merely to illustrate the possible variations in the relationship between carbon dioxide and calcium bicarbonate in natural waters. Curve 4 refers to brackish water containing about 50 per cent normal sea water. It is based on data given by Revelle and Fleming (1934) and by Sverdrup, Johnson, and Fleming (1942). It represents with fair approximation equilibrium conditions in brackish water containing roughly 40 to 60 per cent sea water. Similar curves can be constructed for water containing other proportions of sea water on the basis of data given by the same authors.

The work of other investigators (Wattenberg and Timmerman, 1936) indicates that the equilibrium curve for brackish water may lie somewhat closer to that for fresh water than the one shown in Figure 1. However, the curve shown in Figure 1 represents the more unfavorable conditions for concrete, and it therefore should be used in the absence of definite evidence that the data of Wattenberg and Timmerman are applicable to the case under consideration.

Conditions for Attack on Concrete

In the presence of water, both carbonic acid and calcium bicarbonate can react with calcium hydroxide contained in hydrated cement to form calcium carbonate. The reactions may be represented as follows:

$$Ca(OH)_2 + H_2CO_3 = CaCO_3 + 2H_2O \quad (1)$$

$$Ca(OH)_2 + Ca(HCO_3)_2 = 2CaCO_3 + 2H_2O \quad (2)$$

Calcium carbonate produced in this way is deposited on the surface of the concrete or on the walls of the pores near the surface. If the water in contact with the concrete contains just enough carbonic acid to prevent decomposition of the calcium bicarbonate present in solution, deposition of calcium carbonate is likely to continue until the pores located near the surface of the concrete are filled with it.

If, on the other hand, the water contains more carbonic acid than is necessary to prevent decomposition of calcium bicarbonate already present in solution, some of the excess acid can react with calcium carbonate formed by reactions (1) and (2) as follows:

$$CaCO_3 + H_2CO_3 = Ca(HCO_3)_2 \tag{3}$$

In this way, the relatively soluble calcium bicarbonate is formed and removed in solution, and the porosity and permeability of the concrete are increased.

It should be noted that not all of the excess carbonic acid originally present can take part in reaction (2). Some of it must be, so to speak, held in reserve in order to prevent decomposition of the additional calcium bicarbonate formed by that reaction. The portion which can take part in the reaction is known as *aggressive carbon dioxide* because it is this portion which attacks concrete. Methods of estimating the concentration of aggressive carbon dioxide will be discussed briefly in the following paragraphs.

Estimating Concentration of Aggressive Carbon Dioxide

The concentration of aggressive carbon dioxide in fresh water or in sea water, including brackish water produced by the dilution of sea water with fresh water, can be estimated by a graphical method first suggested by Auerbach (1912). The following data are required: concentration of total free carbonic acid, concentration of calcium bicarbonate, and the position of the carbonic acid-calcium bicarbonate equilibrium curve for the water in question. The first two items may be obtained with sufficient accuracy for most purposes on the basis of an ordinary chemical analysis, provided that sufficient precautions were taken to prevent escape of carbon dioxide gas prior to and during the analysis. Methods of computation and the necessary constants have been given by Sverdrup, Johnson, and Fleming (1942) for sea water and by Frear and Johnston (1929) and by MacInnes and Belcher (1933) for fresh water.

As a basis for estimating the concentration of aggressive carbon dioxide in water containing little or no dissolved material other than calcium bicarbonate, curve 2 (Figure 1) may be used. If the water contains an appreciable quantity of dissolved material other than calcium bicarbonate, but less than about 800 ppm total dissolved solids, the approximate position of the equilibrium curve may be estimated on the basis of data given by MacInnes and Belcher (1933) and by Langelier (1936). For brackish water containing roughly 40 to 60 per cent sea water, curve 4, Figure 1, may be used. For less concentrated brackish water and for normal sea water, similar curves may be drawn on the basis of data given by Sverdrup, Johnson and Fleming (1942). For other types of water, the graphical method cannot be used because the necessary constants have not yet been determined.

In order to illustrate the graphical method of estimating the concentration of aggressive carbon dioxide, let us suppose that we have two samples of water, one of fresh water containing little or no dissolved material other than carbonic acid and calcium bicarbonate, and one of brackish water consisting of equal parts of sea water and fresh water. The concentration of free carbonic acid and calcium bicarbonate is the same in both specimens, and is represented by point *k* in Figure 1. The concentration of aggressive carbon dioxide in each of these two samples may be determined graphically on the basis of the following considerations. If the concentration of carbonic acid exceeds that required to maintain equilibrium, the following reaction can take place:

$$CaCO_3 + H_2CO_3 = Ca(HCO_3)_2$$

In this reaction, one molecule of carbonic acid is consumed for every molecule of calcium bicarbonate which is produced. Each molecule of bicarbonate thus formed contains two molecules of carbon dioxide, of which one was present in the solid calcium carbonate and the other was initially free. In other words, the decrease in the concentration of free CO_2, expressed in ppm or any other convenient units, is equal to one-half the increase in the CO_2 present in the form of dissolved calcium bicarbonate. Hence, if the original composition of a solution is represented by some point such as *k* in Figure 1, its composition after reaction with $CaCO_3$ is represented by some other point such that

$$\frac{\text{Decrease in free } CO_2}{\text{Increase in bicarbonate } CO_2} = \tfrac{1}{2}.$$

All points which satisfy this condition lie on lines parallel to ab in Figure 1. The ordinate of a is 100 and the abscissa of b is 200.

For the sake of convenience, the scale of Figure 1 was so chosen that line ab rises at an angle of 45 degrees to the horizontal axis. A line through k parallel to ab intersects curve 2 at l′. In fresh water, represented by this curve, reaction ceases when point l is reached. The quantity of carbonic acid which has been consumed in the reaction, i.e., the aggressive carbon dioxide, is represented by the length of the line kl, corresponding to 30 ppm CO_2. Similarly, the line k parallel to ab intersects curve 4, representing brackish water, at point m′, and the reaction between brackish water and calcium carbonate ceases as soon as this point is reached. The corresponding concentration of aggressive carbon dioxide is 83 ppm.

If the composition and quantity of substances dissolved in the water is such that the equilibrium curve cannot yet be plotted at the present state of our knowledge, then the concentration of aggressive carbon dioxide can be estimated by determining the calcium bicarbonate content of the water before and after prolonged shaking with powdered calcium carbonate. This method, which was first suggested by Heyer (1886), is likely to result in too low a value for the aggressive carbon dioxide unless the analyst is highly skilled. The chief sources of error are loss of carbon dioxide during shaking, and incomplete reaction between the dissolved carbon dioxide and the solid calcium carbonate. Several days may be required for the completion of this reaction.

Carbon Dioxide in Sea Water

As indicated by the results of the preceding discussion, brackish water (or sea water) with a given concentration of carbon dioxide will, other things being equal, have a much more detrimental effect on concrete than fresh water with the same concentration of carbon dioxide because a greater proportion of the total CO_2 is aggressive. Hence greater caution is indicated in dealing with sea water than with fresh water.

The carbon dioxide content of sea water varies from place to place. In the open ocean, it is generally small, and it is commonly in equilibrium with calcium carbonate. The pH of such water is generally between 8 and 8.2. In sheltered bays and estuaries, on the other hand, the carbon dioxide content may rise above that of normal sea water. As a consequence, the pH decreases. If the increase in carbon dioxide content is accompanied by a notable increase in the calcium bicarbonate content, the drop in pH is small, commonly less than one unit. However, if the rise in the CO_2 content is not accompanied by the maximum possible increase in calcium bicarbonate concentration, i.e., if the water contains aggressive carbon dioxide, the decrease in pH may exceed one unit; in other words, the pH of sea water may be reduced to 7 or less.

The approximate relationship stated in the preceding paragraph may be formulated as follows. If sea water or brackish water is characterized by a pH in excess of 7.5, it contains so little aggressive CO_2 that it is not likely to damage well-made concrete. If the pH is between 7 and 7.5, the concentration of aggressive CO_2 may be close to the maximum tolerable concentration and may even exceed it. Hence if such water is found at the site of a proposed structure, it should be investigated carefully. If the pH is below 7, the concentration of aggressive carbon dioxide is almost certain to be excessive. More accurate rules concerning the relation between the pH values of sea water and the concentration of aggressive CO_2 cannot be established because this relation depends on several factors other than the pH.

In connection with marine construction, the following possibility should be considered. If the proposed construction operations are such that they will prevent daily flushing of the site by tides and other currents, the aggressive carbon dioxide content of the water may increase.

Effect of Dilution on Non-Aggressive CO_2-Bearing Fresh Water

In nature, carbon-dioxide-bearing fresh water may become saturated with calcium bicarbonate if it flows over or percolates through calcareous sediments. It may thus be rendered harmless to concrete. If this process were not fairly widespread, aggressive acid water would be much commoner than it is. However, water which has lost its aggressive character by reaction with calcareous deposits may again become aggressive if it is diluted with water containing little or no calcium bicarbonate, even if this water is non-aggressive. If the diluting water is fresh, only a moderate degree of aggressiveness is likely to be developed, whereas mixture with sea water may produce a highly aggressive brackish water. This fact is also a consequence of the relations shown in Figure 1. In this figure, point j represents water in which the only impurities are carbonic acid and calcium bicarbonate. Since the concentration of these substances is such that the water is represented by a point lying on the equilibrium curve, the water cannot dissolve calcium carbonate. After dilution with an equal quantity of chemically pure water, the point representing the composition of the water moves into position k, which lies above the equilibrium curve. The concentration of aggressive carbon dioxide in the mixture can be determined by the method previously described. By this method, it is found that the concentration is represented by the length of the line kl, corresponding to 30 ppm aggressive carbon dioxide. If, on the other hand, fresh water represented by point j is mixed with an equal quantity of sea water, a similar graphical procedure leads to the conclusion that the concentration of aggressive carbon dioxide in the brackish water is 83 ppm.

In view of the potentially dangerous nature of brackish water formed by the dilution of sea water with carbon-dioxide-bearing fresh water, particular attention should be given to the examination of brackish water if there is any possibility of contamination with carbonic acid. Such contamination may, for instance, take place where rivers draining marshy land enter the sea. Sea water may also be rendered aggressive where fresh spring water discharges on the sea floor after passing through deposits of decaying organic matter. An example of concrete deterioration due to aggressive brackish water, with such an origin, has been described elsewhere (Terzaghi, 1948).

Investigation of Water

If the water at the site of a proposed concrete structure is suspected of containing carbonic acid, samples of the water should be chemically analyzed prior to beginning of construction. However, it must be remembered that such analyses will furnish reliable information regarding the aggressiveness of the water only if the following conditions are fulfilled. First, representative specimens must be taken within the entire space within which concrete will be in contact with water. If, for instance, 100-ft. concrete piles are to be driven, samples of water taken from a shallow well at the site will not necessarily provide complete and accurate information regarding the nature of the water to which the piles will be exposed. Second, the samples must be collected and stored in such a way that dissolved gases will not be lost before the analysis is carried out (see Whipple, 1927, pp. 194-195). Finally, adequate precautions should be taken to prevent escape of dissolved gas during the analysis.

Effect of Carbonic Acid on Concrete

Chemical Changes in Concrete

The most conspicuous chemical effects of carbonic acid attack on concrete are the removal of lime from the paste and the conversion of a part or of all of the remaining lime to calcium carbonate. Silica is also leached out, but much more slowly than lime. As a consequence of these processes, the paste contains considerably more alumina and iron oxide, slightly more silica, and much less lime than the original cement. If the composition of the original cement is known, the quantity of lime lost may be estimated by a method described in the appendix to the writer's previously mentioned paper (1948).

Attack by water containing carbonic acid reduces the strength of concrete. Experiments carried out by Tremper (1931) have shown that loss of strength is proportional to loss of lime; loss of strength becomes complete when the concrete has lost about one-half of its original lime content. A similar relation between loss of strength and loss of lime was found in a group of concrete specimens taken from a shipway (Terzaghi, 1948).

Rate of Attack

Correctly made analyses of properly collected samples of water will provide the data required to decide whether aggressive carbonic acid is present, i.e., whether a deleterious reaction can take place between the water and the concrete. However, it is also necessary to consider the rate at which such a reaction can take place. This rate is influenced by a large number of factors. Foremost among these are the concentration of aggressive carbonic acid, the concentration of other deleterious substances, such as sulphates, the permeability of the concrete and the rate of percolation of the aggressive water through the concrete or the rate of flow at its exposed surfaces. Owing to the fact that adequate quantitative data are rare, no precise evaluation of the effect of these various factors is possible at the present time. On the basis of experience, it is, however, possible to give a few general indications regarding the effect of water containing a given quantity of aggressive carbon dioxide. A few typical examples of such experience will be given in the following paragraphs.

Observations made by the Hydroelectric Power Commission of Ontario (B. Kellam, personal communication, November 26, 1948) indicates that river water characterized by a pH of 6.5 to 7.05 and an aggressive carbon dioxide concentration of 3 to 5 ppm has only a very slightly detrimental effect on well-made concrete. The sole effect of such water on concrete exposed to it for a period of seven to fifteen years consisted in the removal of a ⅛ inch layer of mortar. Similarly, Scheelhaase (1908) noted that no deterioration was caused by water containing 8.5 ppm free CO_2, nearly all of which was aggressive. In contrast, conspicuous deterioration was caused within less than one year by water from another source in the same locality which contained about 27 ppm aggressive CO_2 (30 ppm total CO_2). Biehl (1928) examined a number of culverts which had been in service for twenty years, and analyzed specimens of water to which these culverts had been exposed. He found that deterioration had taken place only in concrete in contact with water containing more than about 17 ppm aggressive CO_2. Nehring (cited by Kleinlogel, 1930, p. 234) reported conspicuous deterioration and local failure of concrete pipes which had been exposed for fourteen years to water containing 30 to 33 ppm aggressive CO_2. These observations suggest that water containing less than about 20 ppm aggressive carbon dioxide will not cause rapid deterioration of concrete of average permeability under ordinary circumstances.

On the other hand, if water percolates through concrete owing to the existence of a high hydraulic gradient, notable deterioration may take place even if the water contains less than 10 ppm aggressive carbon dioxide, particularly if the concrete is relatively pervious. Pfeiffer (1911), for instance, noted that soft water containing 9 ppm free carbon dioxide (most of which was aggressive) had disintegrated the concrete walls of a storage tank to a depth of 4 inches in eight years.

If the aggressive CO_2 content exceeds about 20 ppm, conspicuous deterioration within twenty years (or less) must be expected unless the permeability of the concrete is unusually low. For exceptionally impervious concrete, the threshold value is doubtless somewhat higher than 20 ppm. Nevertheless, we may conclude that many natural waters contain sufficient aggressive carbon dioxide to cause deterioration of the most impervious concrete, since even mortars and marble having a very low porosity may be vigorously attacked. A striking demonstration of this fact is provided by experiments carried out in Bonn, Germany (Schiffner, 1900). In these tests, small specimens of marble and of mortar were immersed for periods of twenty-two and twenty-seven months respectively in flowing city water. At the end of the immersion period, the average volume of mortar test specimens had diminished 10 per cent and that of marble specimens 25 per cent. On the basis of these tests, Schiffner concluded that "no cementitious material containing lime can long resist attack by flowing water containing free carbonic acid."

If concrete is exposed to water which is not in motion, deterioration may, under some conditions, be extremely slow. However, if the source of carbonic acid, such as decaying organic matter, is close to the concrete, or if the water table fluctuates notably, the carbonic acid content of the water in contact with the concrete may be maintained at a fairly high level, and deterioration may proceed almost as rapidly as it would if the water were flowing over the surface of the concrete. Probably both of these conditions contributed to the rather rapid deterioration of foundation piers of a building in Seewen, Switzerland. These piers, which were about one square yard in cross section, rested on lake sediments. Thirty years after construction, it was found that the concrete in some of the piers was completely disintegrated due to attack by carbonic acid. The aggregate was embedded in a soft slimy mass of calcium carbonate (Wiegner, 1928; Gessner, 1929).

If carbonic acid is not the sole detrimental constituent of water in contact with concrete, the rate of deterioration is influenced by the nature and concentration of the other deleterious substances present. Among the most important of these are other acids produced during the decomposition of organic matter, and sulfates.

No systematic chemical investigation has been carried out on the deterioration of concrete due to the products of decay of organic matter other than carbonic acid. Hence, little is known regarding the substances and processes involved, and there is no unanimity regarding the deleteriousness of these substances. Nevertheless, the published results of several investigations (Elliott, 1923; Friese, 1925; Wiegner, 1928) as well as more recent unpublished work which has come to the writer's attention, suggest that acid decomposition-products other than carbon dioxide may also play an important part in the deterioration of concrete.

The rate of deterioration due to attack by water containing both sulphates and carbonic acid appears to be considerably more rapid than if the deleterious constituents of the water consist only of sulfates or only of carbonic acid. Ordinary sea water, for instance, commonly has no detrimental effect on well-made concrete in the absence of frost, whereas sea water containing aggressive carbonic acid is highly deleterious. In addition to the loss of strength which is characteristic of concrete exposed to carbonic-acid-bearing fresh water, concrete subject to attack by acid sea water expands and may become extensively cracked if the expansion is not uniform.

The effects of deterioration due to acid sea water may be conspicuous within a few years after construction. Binger (1927), for instance, has described the deterioration of concrete members of a bridge over the Housatonic River at Shelton, Connecticut. Part of this bridge was constructed in 1919-20 and the remainder in 1922. By 1923, the concrete in both parts had become so soft that large pebbles could be picked out by hand with little effort. Analyses of the concrete showed that about 80 per cent of the calcium in the cement had been transformed into calcium carbonate. Deterioration was attributed to the acidity of the brackish water in contact with the concrete at high tide. The present writer (1948) investigated another case of concrete deterioration due to acid brackish water. Analyses indicated that this water contained 56 ppm aggressive CO_2. Although the structure, a shipway built in 1941, is still far from the state of failure, the effects of deterioration had become conspicuous two years after the concrete was placed. These included local loss of strength, expansion, and extensive cracking.

Prevention of Attack by Carbonic Acid

At present there seems to be only one reliable method of preventing water containing aggressive carbon dioxide from attacking concrete with which it is in contact. This method consists in treating the water, for instance by allowing it to react with calcium carbonate, generally in the form of crushed marble, before it reaches the concrete. It was successfully applied in Frankfurt-am-Main, where the city water, containing about 27 ppm of aggressive CO_2, led to severe deterioration of a concrete storage tank. The aggressiveness of the water was greatly reduced by passing it through a filter consisting of crushed marble, which was dissolved by the water at the rate of nearly three tons per day (Viehsohn, 1930).

Any method of waterproofing the surface which provides permanent protection is likely to be prohibitively expensive. Ordinary impervious coatings offer temporary protection to accessible surfaces but they must be renewed frequently. Impregnation with bituminous material under a vacuum has been reported to give good protection to piles exposed to sea water (Rear Admiral Andrew G. Bisset, C.E.C., U.S.N., personal communication), and it is probable that this method of treatment would afford equal protection against acid waters.

The use of trass in concrete exposed to acid waters has been recommended. However, there is no general agreement as to the resistance of concrete containing trass to attack by acid water, and a great deal more research will be required before such admixtures can be used with assurance. If sulfates are present in the water, deterioration may be delayed by the use of a sulphate-resistant cement.

On some structures, it may be technically impossible, or economically prohibitive, to apply a waterproof coating on every surface exposed to water, and to renew it at frequent intervals. If such structures are exposed to aggressive acid water, it should be recognized in advance that the concrete will not be durable.

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Discussion

By Walter C. Voss, Member*

First of all may I congratulate the author of this paper for again bringing to the attention of engineers the importance of the action of carbonic acid in the disintegration of masonry materials, both natural and synthetic. This subject, as suggested by Mr. Clair, is one long discussed by investigators and I am afraid, quite simply laid aside as of minor import. It has been the writer's experience and opinion for many years past that the lack of available lime hydrate in concrete has played an important part in its durability, particularly when subjected to carbonic acid effects at or near the critical point of unbalance. The writer has shown that at this point the formation of a soluble bicarbonate becomes possible; this is the essence of disintegration as far as it may be traced to dissolution action.

It is well known to geologists that granite, perhaps our most stable masonry material, will eventually disintegrate under similar action, and it is only the naive who would maintain that concrete is superior in this respect. It seems quite obvious that a concrete which may be subjected to the conditions which the author of this paper depicts—and they may be quite common—must have an excess of available lime hydrate with which the carbonic acid may react to form the more stable carbonate, if durability is to result.

This thesis is presently a much discussed one. There are those who would blame the presence of lime hydrate in the hardened matrix for the disintegration of concrete. There are others—and your speaker is one of these—who believe that a reasonable excess of lime hydrate in the concrete is desirable, if not necessary. The first contention is quite naive in that we are supposed to assume that in the hydration of cement, at least the tri-calcium silicate breaks down into dicalcium silicate and lime hydrate. If this reaction goes to completion, as assumed, at least 19 pounds of lime hydrate is formed for each bag of a normal cement used. If one is opposed to the addition of lime hydrate to concrete, why should one also argue for a higher percentage of tri-calcium silicate?

It is in bringing this old discussion again to the attention of engineers that this paper does its greatest service. Although cement has been used and known to some extent for over a hundred years,

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it is true that no real progress has been made in the fundamental knowledge of the reactions during burning, during hydration and during use. To be sure many researches, most valuable, have caused us to change our cement composition and technology of concrete, but I am afraid these have been a series of performance tests which have been exceptionally difficult to evaluate and which have resulted all too often in erroneous specifications and further failures. The sad thing is that we are not sure why these failures occurred. The answer to this query is to be found only in fundamental research. The practical tests should stimulate those who are interested, to explore the "why" in the hope that some basic knowledge may guide our thinking and our use of concrete. Concrete is a good material when we learn how to use it intelligently.

Discussion

By Herman G. Protze, Member*

Dr. Terzaghi's paper is an interesting and thought provoking addition to the limited factual data dealing with the variable effect of destructive agents upon concrete durability. (For a more complete treatment of the specific subject the reader should refer to Dr. Terzaghi's companion paper found in the American Concrete Institute Journal, Vol. 19, No. 10, page 977, June 1948.)

Classically we normally think of the disintegrating agents in natural waters to be largely the neutral salts or soluble acid-forming sulfates and alkalies, depending upon the locality. Whereas these agents are certainly of great importance, and possibly of prime importance even in the presence of aggressive carbon dioxide, it is nevertheless true that certain cases have been reported (notably in Sweden and Germany) where disintegration of concrete has occurred due to attack by natural acid waters apparently without sulfate (or prime alkaline) reaction. It is therefore indicated that our preliminary analyses of projects should include an examination for aggressive CO_2 as Dr. Terzaghi has suggested. The writer cannot but help point out parenthetically, however, that the moderate number of past failures of concrete structures attributed to CO_2 attack are very generalized and astoundingly incomplete in data relative to analysis of mixture, constituents, and surrounding attacking media. The writer suspects that whereas aggressive CO_2 may well have been (an important) party

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to these attacks, particularly at Newport News, the failures might still have been reduced or prevented by proper application of concrete technology.

The literature reports no specific data which unequivocally points to aggressive CO_2 as a destructive agent to properly designed concrete, and thus there is need for thorough and sincere investigations to this end. Complete case histories presenting the required data must be studied—if existing—followed by all inclusive laboratory researches to determine affect by all applicable variables in order ultimately to establish proper tolerances for use of concrete under the specific conditions to apply. The writer is of the opinion that the interesting published data for the currently discussed graving dock at Newport News though actually pointing to attack by CO_2, is also unfortunately inadequate for real use as a case history (due, for example, to companion attack of concrete by sulfates, and a confusion in the mix constituents: aggregates are "all from the same source", but what are the hourly data for cleanliness, gradation, etc.; cement tests are bin averages presented by the manufacturers, but what are the engineers' specific analyses for each car, which often vary widely from the average?).

If then, proper field studies and laboratory researches are yet to be conducted, and we believe they are called for as Dr. Terzaghi implies, they should follow the well established paths of proper research. Then with known limitations, aggressive waters may ultimately be analyzed for classification as to potential destructive ability. To that end it is to be emphasized that samples must be obtained during the various seasons of the year to be fully typical, and considerations must be given to the rate of flow of water past the structure and to temperature changes which markedly affect rate of chemical attack.

It is interesting to note that concrete may be affected by waters containing as little as 0.1% of soluble sulfates (as SO_3) or 0.005% free CO_2. Inasmuch as the effect of humic acid (in natural acid waters) upon adequately hardened concrete is slight relative to the effect of aggressive CO_2, the pH measurement does not bear a direct relationship to the potential deleterious action. Suffice to say that the prediction of these effects is rather complicated particularly because of the inter-relation between free CO_2 and calcium bicarbonate. However, it is possible to determine the amount of aggressive CO_2

after titration for calcium bicarbonate, and tables have been published by Werner, Lea, and others indicating incomplete probable potential effects. On the other hand as early as 1812 Vicat and Michaelis led the way to indicate the effect of action of $MgSO_4$ in water upon concretes, where that salt reacts with the free $Ca(OH)_2$ in set cement to form calcium sulfate, at the same time precipitating magnesium hydroxide. The sulfates also react with the hydrated calcium aluminate to form calcium sulfo aluminate. These reactions, in part responsible for the chemical attack of concretes by sea water, in a way parallel the reactions for the attack by carbonic acid. Dr. Terzaghi has described that the acid solutions similarly dissolve part of the set cement, weakening the materal by removal of the cementing constituents and leaving a soft mushy mass surrounding the aggregate. The resulting slime is described as a form of $CaCO_3$. If calcium hydroxide, silicate, or aluminate is attacked by sodium sulfate a slime of $CaSO_4$ (accompanied by expansion) results, and care must be exercised in positively identifying the resultant material properly by analysis for both CaO and SO_3. In the case of sulfate attack a physical aggravation may also occur due to the simulated freezing and thawing action.

The hydration of the so-called kiln compounds (supposedly found in freshly ground Portland cement) may most simply be indicated by the following schematic equations:

$$CaSO_4 \,.\, 2H_2O + \text{Water} \rightarrow CaSO_4 \,.\, x\,H_2O \qquad (1)$$

$$3CaO \,.\, Al_2O_3 + \text{Water} \rightarrow 3CaO \,.\, Al_2O_3 \,.\, x\,H_2O \qquad (2)$$

$$\left.\begin{array}{l} 3CaO \,.\, SiO_2 + \text{Water} \\ 2CaO \,.\, SiO_2 + \text{Water} \end{array}\right\} \rightarrow zCaO \,.\, ySiO_2 \,.\, x\,H_2O + Ca(OH)_2 \qquad (3)$$

$$(1)+(2)+(3) \rightarrow 3CaO \,.\, Al_2O_3 \,.\, 3CaSO_4 \,.\, 31H_2O + 4CaO \,.\, Al_2O_3 \,.\, x\,H_2O$$

$$4CaO \,.\, Al_2O_3 \,.\, Fe_2O_3 + \text{Water} \rightarrow 3CaO \,..\, Al_2O_3 \,.\, x\,H_2O + CaO \,.\, Fe_2O_3 \,.\, x\,H_2O$$

It may be seen that the (crystalline, gelantinous, or amorphous) hydration products are complex even if reduced to simplest form but it is rather clear that the only source of soluble lime (as calcium hydroxide) is from the calcium silicates. To reduce the potential attack, the free and soluble lime must be kept to a minimum. This reduction can best be accomplished by judicious choice of type and brand of cement and/or by the possible addition of pozzolans (to produce less soluble lime-pozzolan compounds).

Incidentally it is understood that most of the attack at Newport

News is limited to concrete employing type I cement with a C_3A content of 12% whereas little or no attack occurs with the type II cement concrete. This is in step with modern thinking. Although the writer has always been a proponent of placing underwater concrete at a wetter consistency than formed concrete (say at 4" slump) in order to provide easy natural consolidation and prevent wash, we cannot help but exclaim over the fact that the tremie concrete here was placed at 7" slump with a cement factor of 6.3 sacks!

The purpose of the preceding, apparently rambling, discussion is to indicate trends for thinking for future investigations or jobs rather than to confuse the reader or reflect the writer's state of mind after trying to properly evaluate previously incomplete and misleading published data on the subject to determine real causes for such disintegration. Dr. Terzaghi, on the other hand, poses the proper question and also indicates the need for establishing criteria by means of which existence and effect of certain destructive agents can be predicted.

Pending the availability of such criteria, and in conclusion, it may be stated that the compounds in hardened Portland cement are attacked by water and many salt and acid solutions, though fortunately in most cases the action on a properly hardened dense mass is so slow to be relatively unimportant. If the use of concrete is not to be avoided altogether, then proper mixtures and protection must be provided. Experience has indicated to the writer that most concrete troubles are due to improper design of the concrete (mixture and constituents) aggravated by physical and chemical attacks, such as abrasion, impact, freezing and thawing, sulfate action, organic action, "carbonic acidosis", etc. The remedy is fundamentally in the design of a proper mixture to meet the conditions obtaining. In general the concrete mixture to meet the attack of aggressive waters should be designed to contain:

a cement factor not less than 6.5 sacks per cu. yd.
a w/c ratio not greater than 5.5 gal. per sack
a proper aggregate ratio to provide dense impervious concrete
a slump between 2 to 3½"

Furthermore the constituents should have the following high qualities:

Cement should be a type II or type V of known service characteristics. The use of Merriman cement would be definitely helpful.

Aggregate should be high quality, clean, well graded, with a mortar strength

ratio over 100% and a $MgSO_4$ loss not to exceed 5% in the five cycle test.
Water should be free of organic matter and harmful radicals. Mortar strength ratio should be over 100%.

The use of pozzolans or air entraining agents should be considered. Proper methods of placement and curing are most important for the building of structures to resist and withstand the elements.

If time permits, accelerated laboratory tests can be made with the specific aggressive water to predict the severity of the potential effect. Usually this is not possible and hence the need for development of complete empirical data for such predictions. In the meantime—or in any event, in such as in other cases—the only real guarantee for concrete durability is to design proper mixtures and see that the construction is carried out with the very best practices. Few people honestly realize that it is possible to increase the life of a structure scorefold by such methods.

Closing Discussion

By Ruth D. Terzaghi

Recognition of Deleterious Waters

Mr. Protze calls attention to the existence of tables for the determination of the concentration of aggressive carbon dioxide. All such tables which have come to the writer's attention are applicable only to fresh water containing little or no dissolved material other than calcium bicarbonate, carbon dioxide, and in one case, calcium compounds other than bicarbonate. As emphasized in this article, such tables would be utterly misleading if used in connection with saline waters; even for potable waters containing more than a few hundred ppm dissolved substances, they are not sufficiently accurate.

Characteristics of Cement

Both discussers have expressed the opinion that the durability of concrete is influenced by the free lime hydrate (calcium hydroxide) content of the set cement. They disagree, however, as to whether the influence is good or bad. Professor Voss finds it obvious that concrete exposed to carbonic-acid-bearing water "must have an excess of available lime hydrate with which the carbonic acid may react," whereas Mr. Protze states that "to reduce potential attack, the free and soluble lime must be kept to a minimum". Probably most concrete technologists will share Mr. Protze's point of view. Inasmuch as removal of the most soluble constituent (lime) as an essential feature of carbonic acid attack, an increase in the quantity of this vulnerable constituent does not appear to be indicated.

It may be possible, as Mr. Protze suggests, to decrease the free lime content by the addition of a pozzuolanic material. Tests have shown that such additions are likely to increase the sulfate-resistance of concrete, but their effect on the resistance of concrete to the attack of carbonic acid is less consistent (see, for instance, Roos at Hjelmsäter, 1931, and Schiffner, 1899).

For concrete exposed to sulfates; Mr. Protze's suggested specification of a Type V or a Type II cement "of known service characteristics" is appropriate. The only possible source of objection to the specification of Type V cement lies in the difficulty of conforming with such a requirement. According to reports of construction engineers practicing in various parts of the country, Type V (sulfate-resisting) cement is virtually unobtainable in the United States, although it can be purchased from many building supply dealers in Canada, particularly in the western provinces where so-called alkali (sulfate-rich) soils are common. Possibly a more widespread and insistent demand for Type V cement would overcome United States manufacturers' reluctance to produce it. Type II cement, which is likely to be moderately sulfate-resistant, is fortunately obtainable in the United States.

The writer is in entire agreement with Mr. Protze's recommendation that the service record of a cement should, if possible, be examined before it is selected for use in an important structure exposed to aggressive water. Conformity to Type II specifications, for instance, indicates a fairly high probability of resistance to moderate sulphate attack, but it does not guarantee such resistance. Definite assurance of the desired degree of resistance can be obtained only from records of long and satisfactory service.

Quality of Concrete

Since very little concrete is perfect in all respects, it is undoubtedly true, as Mr. Protze points out, that many cases of concrete deterioration due to chemical attack would have been less severe if aggregate, cement, mix design, and placing technique had been more nearly perfect. Since it is neither humanly nor economically possible to achieve perfection on every concrete job, it is one of the important functions of a preliminary chemical investigation of potentially deleterious water to call attention to the need for concrete of particularly high quality if the water is even moderately aggressive.

Probably all will agree that Mr. Protze's suggested specifications are a step in the right direction. The proposed reduction of the water-cement ratio from six gallons per sack, as generally specified for concrete exposed to sea water, to 5.5 gallons per sack would undoubtedly delay deterioration due to chemical attack to some extent because it would decrease the permeability of the concrete.

References

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Schiffner (1899), Weitere Mittheilungen über den Einfluss der Kohlensäure auf Portland-Cement und Trass-Mörtel. Verein deutscher Portland-Cement-Fabrikanten, *Protokoll der Verhandlungen,* 1899, pp. 121-125.

HISTORICAL BACKGROUND OF THE BLACKSTONE VALLEY SEWER DISTRICT

By Charles G. Hammann*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on December 1, 1948.)

The events which led to the creation of the Blackstone Valley Sewer District have occurred many times in many places throughout the civilized world. With slight variations their occurrence is an old story to those engaged in the sanitary engineering profession.

Over a period of many years several factors combined to bring about and subsequently intensify the serious problem of water pollution. Among the more significant of these were the following:

1. The establishment of an ever-increasing number of urban communities.
2. The rapid expansion of urban areas due to shifting populations and accelerated birth rates.
3. The rapid growth of liquid waste producing industries.
4. The intensive application of mass production methods in those industries.

In most instances these developments took place in the vicinity of water-courses because they provided economical sources of water for power, transportation and processing. The natural sequence of events was to discharge sewage and industrial wastes into the same water-courses and this was done indiscriminately in most cases.

It was recognized as early as 1866 in Rhode Island that such practices would have adverse effects upon the health, comfort and economy of the people if allowed to continue without restriction. In that year the General Assembly enacted a statute authorizing the City of Providence to levy penalties for rendering impure the waters of the Pawtucket, Ten Mile, Woonasquatucket and Blackstone Rivers. Efforts to accurately evaluate the effects of this action have been fruitless. However, it is known that the pollution problem became more widespread notwithstanding the provisions of this statute.

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Conditions grew steadily worse until 1920 when the Legislature, again taking cognizance of the matter, enacted a comprehensive pollution control statute and created a Board of Purification of Waters to administer its provisions. As a result of successive reorganizations of the state government the powers and duties of the Board were transferred to the Division of Purifications of Waters in the Department of Public Health in 1935, and subsequently to the Division of Sanitary Engineering in the Department of Health in 1939.

The staffs of each of these agencies worked diligently to prevent further pollution of the waters of the state and to improve those already polluted. Appreciable progress was made for a time as a result of their efforts. In come cases the objectives were achieved through the cooperative action of the regulatory agency and offenders. In others it was necessary to resort to litigation. By one means or the other many sources of pollution were eliminated completely. The harmful effects of others were reduced by the construction and operation of sewage and industrial waste treatment plants.

Unfortunately, however, progress was impeded before all existing sources of pollution could be brought under control. Problems typical of those eventually encountered by most pollution control agencies arose. Economic, legal and political factors, singly or in combination, made the solution of each succeeding case more difficult and, in some cases, impossible.

Meanwhile, population growth and industrial expansion continued, making uncontrolled sources of pollution proportionately more significant. The net result was that certain of the waters of the state became grossly polluted.

Numerous investigations and reports were made and remedies proposed at various times during this period but adequate abatement measures were not carried out.

With the exception of spasmodic agitation by unorganized groups, public interest in the matter was more or less apathetic until about three years ago. At that time the many interested groups joined forces to present a concerted and sustained demand for corrective action.

Early in 1946, Governor John O. Pastore directed the Department of Health to provide him with specific recommendations for a pollution control program which would meet present needs, which would be adaptable to future needs and which would involve a minimum expenditure of public and private funds. Such a report was prepared by Walter J. Shea, Chief of the Division of Sanitary Engineering, and transmitted to the governor on December 23 of the same year. A major feature of the report was a recommendation that a sewer district be established for the purpose of combatting pollution in the lower Blackstone Valley and the Moshassuck Valley, adjacent drainage areas which contribute heavily to what Mr. Shea has described as the most vital and pressing pollution problem facing the state.

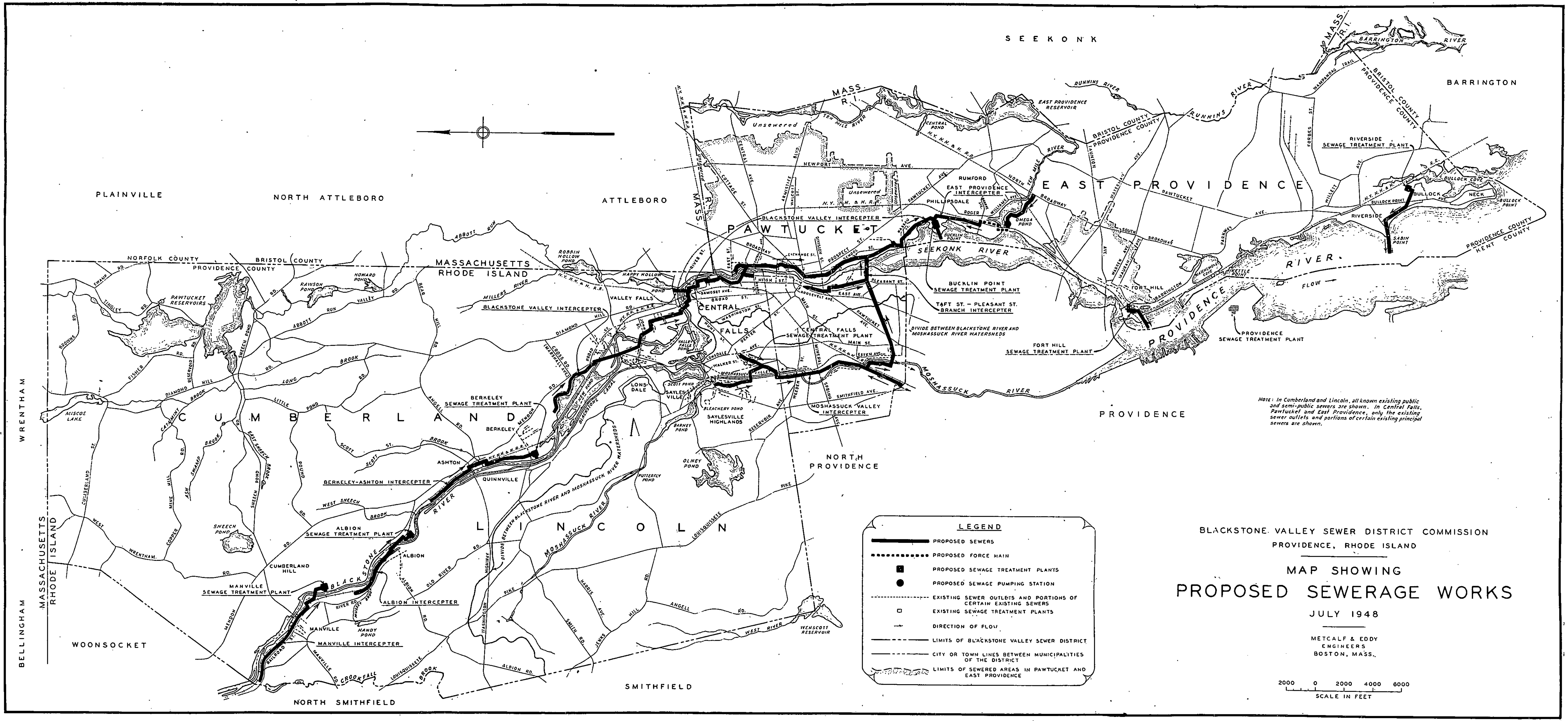
Governor Pastore immediately sponsored legislation essential to the fulfillment of this recommendation in the form of a bill entitled the Blackstone Valley Sewer District Act. The urgent need and general popularity of this action were demonstrated by the fact that the bill was enacted into law on the sixteenth day after its introduction and was subsequently supported in referendum by the overwhelming ratio of fifteen to one. Thus, the Blackstone Valley Sewer District came into being.

The District is composed of the cities of Pawtucket and Central Falls and the towns of East Providence, Cumberland and Lincoln, a group of municipalities located in the northeastern part of the State of Rhode Island which cover an area of approximately seventy-five square miles and have an estimated aggregate population of one hundred and seventy thousand.

The District is administered by a Commission consisting of five members, three of whom are appointed by the Governor for staggered terms of three years, and two of whom are the Director of Public Works and the Director of Health, ex-officiis. The Commission is authorized to plan, construct and operate a group of self-liquidating sewerage works to be known as the Blackstone Valley Sewer District Project. It is prohibited, however, from constructing or enlarging lateral sewer systems. The Act stipulates that funds for the Project shall be obtained by the issuance of bonds not exceeding $5,000,000.00 backed by the full faith and credit of the state. The act also specifies that revenue for self-liquidations shall be derived from reasonable assessments against municipalities and industries having connections with the Project.

The Commission first began to function on August 25, 1947, soon after the appointment of its initial members. Recognizing the magnitude of the problem created by the condition of the streams rising in or flowing through its jurisdiction, the Commission resolved that it would ascertain what should be done within the District to restore them to a reasonable degree of purity, notwithstanding the possibility that some of the required remedial measures might not be the responsibility of the Commission. To this end, the firm of Metcalf and Eddy was engaged on February 10, 1948, to make a comprehensive investigation of the entire situation. The work was completed after several months of diligent work and the final report of the engineers was submitted on July 21, 1948.

FIG. 1



BLACKSTONE VALLEY SEWER DISTRICT COMMISSION
PROVIDENCE, RHODE ISLAND
MAP SHOWING
PROPOSED SEWERAGE WORKS
JULY 1948
METCALF & EDDY
ENGINEERS
BOSTON, MASS.
2000 0 2000 4000 6000
SCALE IN FEET
LEGEND
PROPOSED SEWERS
PROPOSED FORCE MAIN
PROPOSED SEWAGE TREATMENT PLANTS
PROPOSED SEWAGE PUMPING STATION
EXISTING SEWER OUTLETS AND PORTIONS OF CERTAIN EXISTING SEWERS
EXISTING SEWAGE TREATMENT PLANTS
DIRECTION OF FLOW
LIMITS OF BLACKSTONE VALLEY SEWER DISTRICT
CITY OR TOWN LINES BETWEEN MUNICIPALITIES OF THE DISTRICT
LIMITS OF SEWERED AREAS IN PAWTUCKET AND EAST PROVIDENCE
Note: In Cumberland and Lincoln, all known existing public and semi-public sewers are shown. In Central Falls, Pawtucket and East Providence, only the existing sewer outlets and portions of certain existing principal sewers are shown.
PLAINVILLE
NORTH ATTLEBORO
ATTLEBORO
SEEKONK
BARRINGTON
WRENTHAM
BELLINGHAM
WOONSOCKET
NORTH SMITHFIELD
SMITHFIELD
NORTH PROVIDENCE
PROVIDENCE
CUMBERLAND
LINCOLN
CENTRAL FALLS
PAWTUCKET
EAST PROVIDENCE
MASSACHUSETTS
RHODE ISLAND
NORFOLK COUNTY
BRISTOL COUNTY
PROVIDENCE COUNTY
KENT COUNTY
BLACKSTONE RIVER
SEEKONK RIVER
PROVIDENCE RIVER
MOSHASSUCK RIVER
BLACKSTONE VALLEY INTERCEPTER
BERKELEY-ASHTON INTERCEPTER
ALBION INTERCEPTER
MANVILLE INTERCEPTER
MOSHASSUCK VALLEY INTERCEPTER
EAST PROVIDENCE INTERCEPTER
TAFT ST. - PLEASANT ST. BRANCH INTERCEPTER
MANVILLE SEWAGE TREATMENT PLANT
ALBION SEWAGE TREATMENT PLANT
BERKELEY SEWAGE TREATMENT PLANT
CENTRAL FALLS SEWAGE TREATMENT PLANT
BUCKLIN POINT SEWAGE TREATMENT PLANT
FORT HILL SEWAGE TREATMENT PLANT
PROVIDENCE SEWAGE TREATMENT PLANT
RIVERSIDE SEWAGE TREATMENT PLANT
DIVIDE BETWEEN BLACKSTONE RIVER AND MOSHASSUCK RIVER WATERSHEDS
PAWTUCKET RESERVOIRS
VALLEY FALLS
LONSDALE
SAYLESVILLE
SAYLESVILLE HIGHLANDS
ASHTON
BERKELEY
QUINNVILLE
ALBION
MANVILLE
CUMBERLAND HILL
RUMFORD
PHILLIPSDALE
RIVERSIDE
BULLOCK NECK
SABIN POINT
FLOW

THE PROPOSED BLACKSTONE VALLEY SEWAGE AND WASTES COLLECTION SYSTEM AND TREATMENT WORKS

By E. Sherman Chase, Member*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on December 1, 1948.)

The Blackstone Valley Sewer District includes the cities of Pawtucket and Central Falls and the towns of East Providence, Cumberland, and Lincoln (Fig. 1). The total area of these municipalities is approximately 75 sq. mi. This area is drained in part by the Blackstone River which becomes the tidal Seekonk River below the Main Street dam in Pawtucket. The Seekonk in turn enters the Providence River near Fort Hill, East Providence. The Moshassuck River drains parts of Lincoln, Central Falls, and Pawtucket, and portions of Providence. The Ten Mile River drains parts of Pawtucket and East Providence and is tributary to the Seekonk River. The small water courses which drain into the larger streams divide the major drainage areas into several smaller areas. The general topography of the District is irregular and distinctly disadvantageous for interception of the sewage and wastes originating therein.

The areas of both Pawtucket and Central Falls are largely built upon either by residences or by business structures. The other communities are much less densely developed, although they show evidence of receiving the overflow from the major cities. Pawtucket and Central Falls are highly industrialized as is the town of East Providence along its waterfront. Considerable industrial development also occurs in the other municipalities. The predominating industries are textile or related thereto.

Pawtucket is well sewered, most of the sewage being discharged to the Blackstone and Seekonk Rivers, although a substantial amount is discharged into the sewer system of Providence. Central Falls is rather completely sewered, with part of the sewage treated at an old and inefficient plant from which the effluent is discharged into the Moshassuck River. The remaining sewage from Central Falls is dis-

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charged without treatment into the Blackstone River. East Providence is partly sewered with the sewage receiving treatment by sedimentation and chlorination, the effluent being discharged offshore in the Providence River. Most of the industrial wastes of all the municpalities is discharged without treatment directly to the several streams.

Pollution Problem

The Blackstone, Moshassuck, and Seekonk Rivers are grossly and notoriously polluted by the sewage and industrial wastes originating in the District. This is a condition which has long existed and which has become aggravated with the passing years. Numerous investigations and reports have been made and remedies proposed. To date no adequate measures of abatement have been carried out.

Essentially the problem of pollution abatement involves five major items, namely:

1. The extent to which impurities entering the Blackstone, Moshassuck, Seekonk, and Ten Mile Rivers from the District must be reduced in order to restore the stream waters to a reasonable degree of purity;
2. The manner whereby the sewage and industrial wastes from the District should be intercepted and conveyed to treatment facilities;
3. The method or methods which should be employed to adequately treat the sewage and wastes;
4. Estimates of cost of construction and operation of the structures involved in the remedial program; and
5. An equitable method for allocation of cost among the municipalities and industries contributing to the intercepting sewers and treatment works.

Investigations were made early in 1948 by Metcalf & Eddy to determine the volumes and character of the sewage and industrial wastes originating in the District. Flows of sewage were gaged and samples of sewage collected and analyzed. Industries producing wastes were visited and measurements and analyses of the major and more significant flows of wastes were made. Field surveys were made for interceptor routes and for treatment plant sites. Borings along possible locations for interceptors and at proposed treatment plant sites were made by a boring contractor.

SANITARY CONDITION OF BLACKSTONE, MOSHASSUCK, SEEKONK AND TEN MILE RIVERS

Although the Blackstone River as it enters the District is already seriously polluted, the discharge of additional volumes of sewage and wastes further increases its objectionable characteristics. The effect of pollution, including the pollution from Woonsocket as well as that from within the District, is illustrated graphically by Fig. 2 which shows the dissolved oxygen and oxygen demand of the river water in the midsummer of 1947 as determined by the State Department of Health. To understand the significance of this graph it should be pointed out that generally speaking a relatively clean surface water should contain, under summer conditions, not less than about 6 ppm (parts per million) of dissolved oxygen and not over 2 or 3 ppm of 5-day B.O.D. (biochemical oxygen demand). As long as the water contains any amount of dissolved oxygen it is not likely to give off offensive odors, but when the oxygen is exhausted by decomposing organic pollution, objectionable odors are almost sure to be produced. For this reason the dissolved oxygen content of the river is an excellent index of its condition, and as shown by Fig. 2 the oxygen content of the Blackstone was completely exhausted at the time the data were obtained upon which the diagram is based.

The Seekonk River not only receives the pollution brought into it by the Blackstone and Ten Mile Rivers, but also receives direct pollution by sewage and industrial wastes from Pawtucket and East Providence. The condition of pollution is aggravated by the tides carrying pollution back and forth between the Seekonk's entrance into the Providence River and the head of tidewater at Main Street, Pawtucket. The waters of the Providence River also carry pollution from sources below the mouth of the Seekonk into the latter river at times of flood tide.

The Seekonk River itself acts as a large settling basin in which extensive deposits of decomposing, offensive sludge accumulate. The effect of these sludge deposits is possibly even more objectionable from the standpoint of pollution than is the effect of current discharges of sewage and wastes.

The State Department of Health made an extensive survey of the condition of the Seekonk in the summer of 1947. Fig. 3 shows the condition of the surface water at numerous points between Divi-

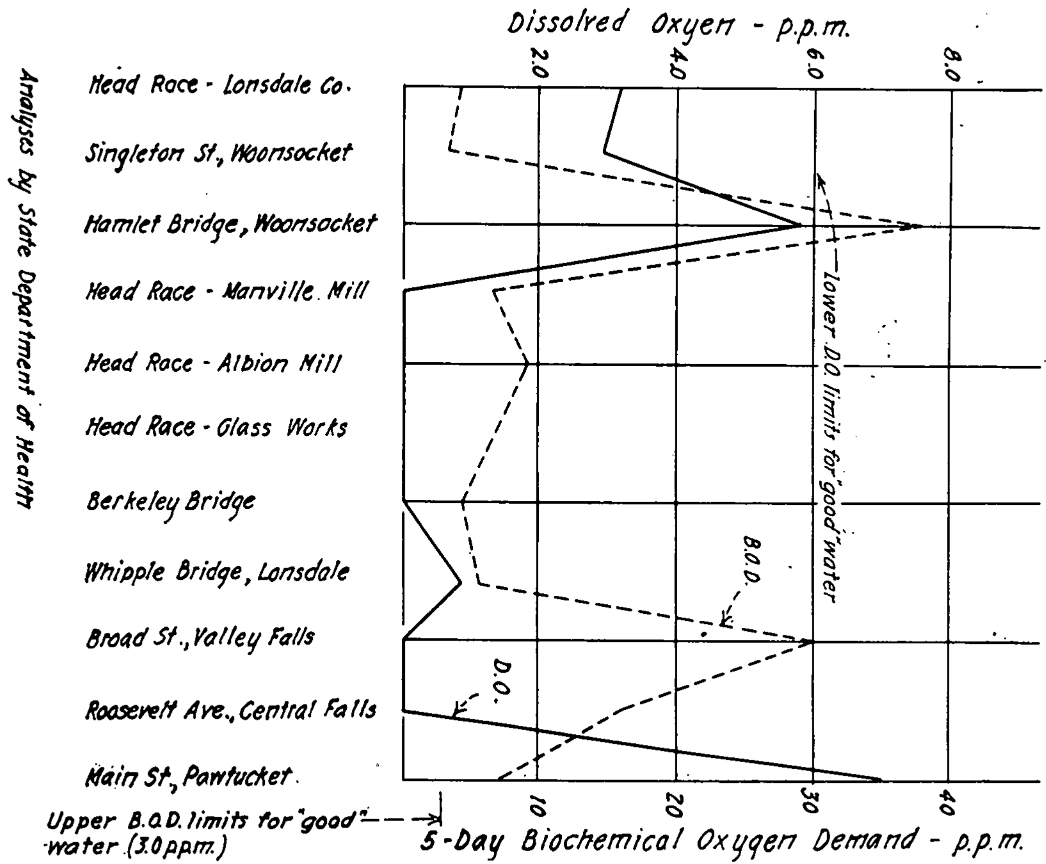
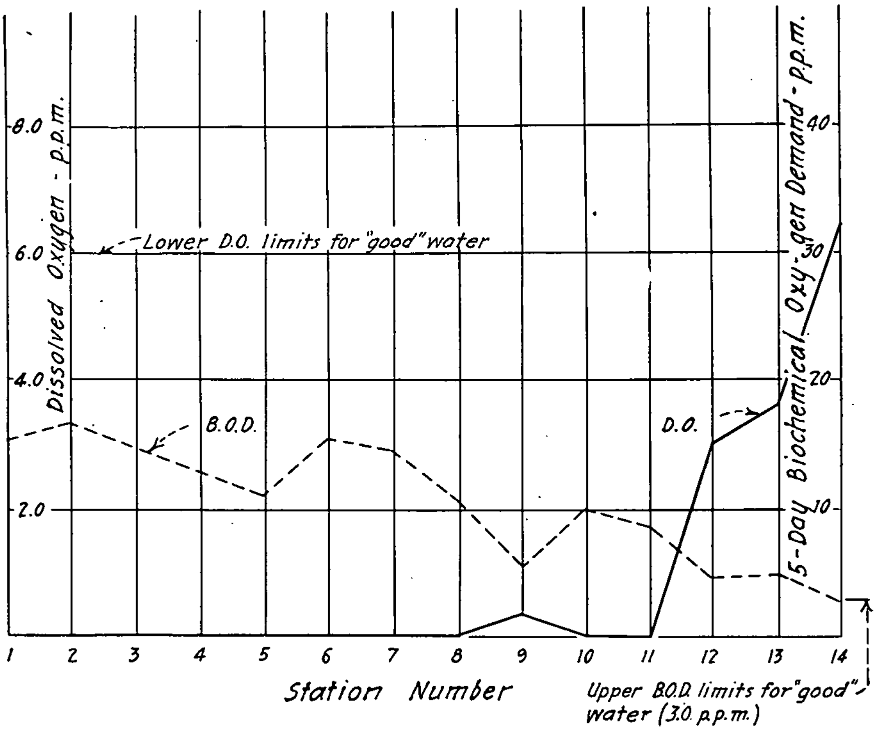


FIG. 2.—CONDITION OF BLACKSTONE RIVER ON AUGUST 21, 1947.



Station 1, near Division St., Pawtucket.

Station 14, just north of Washington St. bridge, Providence.

Analyses by State Department of Health.

FIG. 3.—CONDITION OF SEEKONK RIVER ON AUGUST 13, 1947, TOP SAMPLES.

sion Street, Pawtucket, and Washington Street, Providence, during the midsummer in 1947. From this diagram it will be seen that nearly the entire stretch of the river was devoid of oxygen, a condition indicating extreme pollution.

Although it has been estimated that the minimum daily volume of diluting water at Bucklin Point approximates 800 mil. gal., this diluting water wholly fails to adequately care for current pollution and the decomposition products of sludge deposits.

The Moshassuck River, which originates in the town of Lincoln, flows through several mill ponds and past a number of industries on its way to join the Woonasquatucket River in Providence. The area drained comprises parts of Lincoln, Central Falls, Pawtucket, and Providence. Surface drainage, sewage overflows, treatment plant effluents, and industrial wastes which discharge into the stream or its tributaries create an almost intolerable condition of pollution. During the summer months the stream in its course receives so much pollution that it becomes a foul, odorous liquid, offensive to sight and smell.

Part of the sewage from Pawtucket is diverted to the Providence sewers but overflows occur from time to time which add to the total pollution of the Moshassuck River. The sewage of Central Falls is treated in a plant consisting of an Imhoff tank, trickling filter, and final settling tank. This plant was built about 1913 and is now in a bad state of repair. The character of effluent as disclosed by analyses of the State Health Department and by tests of Metcalf & Eddy is extremely unsatisfactory. The trunk sewer leading to the plant is inadequate and substantial quantities of raw sewage flow directly to the stream at times.

Part of the industrial wastes produced by the Sayles Finishing Plants, Inc., is treated by sedimentation, filtration through a trickling filter, and final filtration through cinder beds. The filters are of insufficient capacity to treat the entire flow of effluent from the primary subsiding basins with the result that a large portion of the settled wastes by-passes these filters. In addition, considerable volumes of wastes are discharged directly into the Moshassuck River without any treatment. The wastes which are so discharged without treatment are of such character as to create an acid condition in the river, which has been found to adversely affect the water for its use by industries downstream.

At the time of the Metcalf & Eddy investigations approximately 6 mil. gal. of polluting liquids, sewage and industrial wastes, were reaching the Moshassuck River daily. In dry seasons the entire flow of the stream consists of these liquids. Analyses indicate that in dry weather the pollution introduced will demand each day many times the oxygen available. Analyses of the river water in warm weather show the complete absence of oxygen. The diagram constituting Fig. 4, based upon analyses by the State Department of Health, shows graphically a typical summer condition of the Moshassuck.

The Ten Mile River, which rises in Massachusetts, enters the

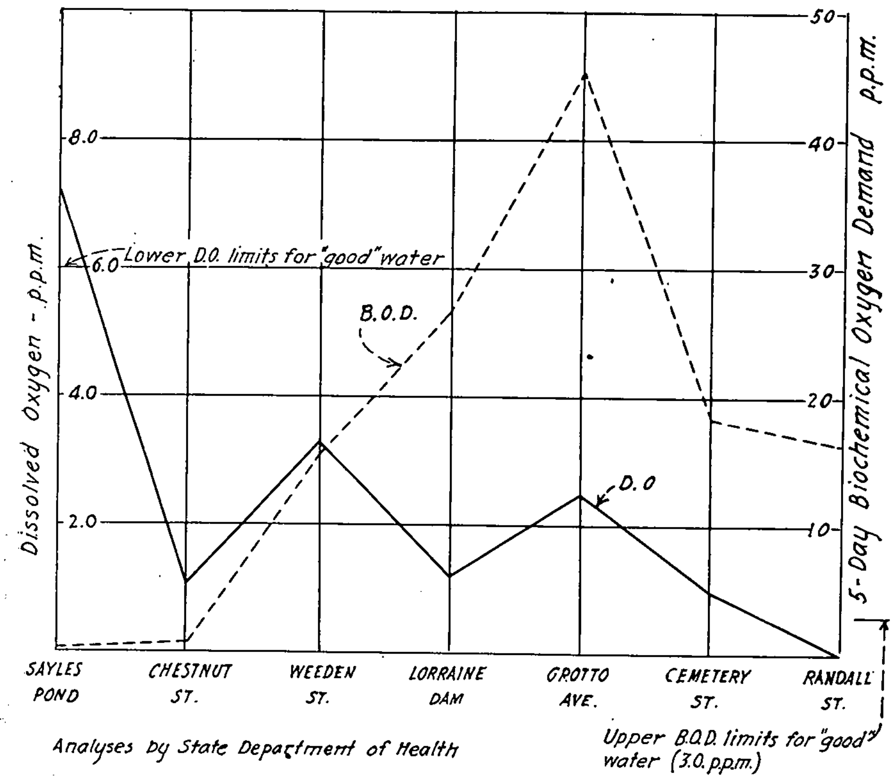


FIG. 4.—CONDITION OF MOSHASSUCK RIVER ON AUGUST 14, 1947.

Seekonk through Omega Pond in the northerly portion of East Providence. After filtration, water from this river is used as the source of water supply of that town. The water works reservoir and intake are located about two miles from the outlet of Omega Pond into the Seekonk River. Below the water works reservoir the principal pollution of the stream consists of the wastes from the Rumford Chemical Works. However, analyses at the time of the Metcalf & Eddy investigation indicated a moderate degree of pollution only.

POPULATION OF DISTRICT

It is necessary to design sewerage works to serve adequately for a number of years in the future. Consequently it is essential to estimate as closely as possible the populations to be served in the future. Such population estimates permit forecasts to be made of the volumes of domestic sewage and, to a limited degree, the volumes of industrial wastes and ground water infiltration.

As the future is the projection of the past, the census data from 1890 to 1940 have been studied for each of the municipalities in the District and predictions made as to future populations. These data are shown graphically on Fig. 5. On the basis of the population study it is estimated that the District now has a population of 170,700 which will increase to 221,800 in 1975 and to 260,000 in the year 2000.

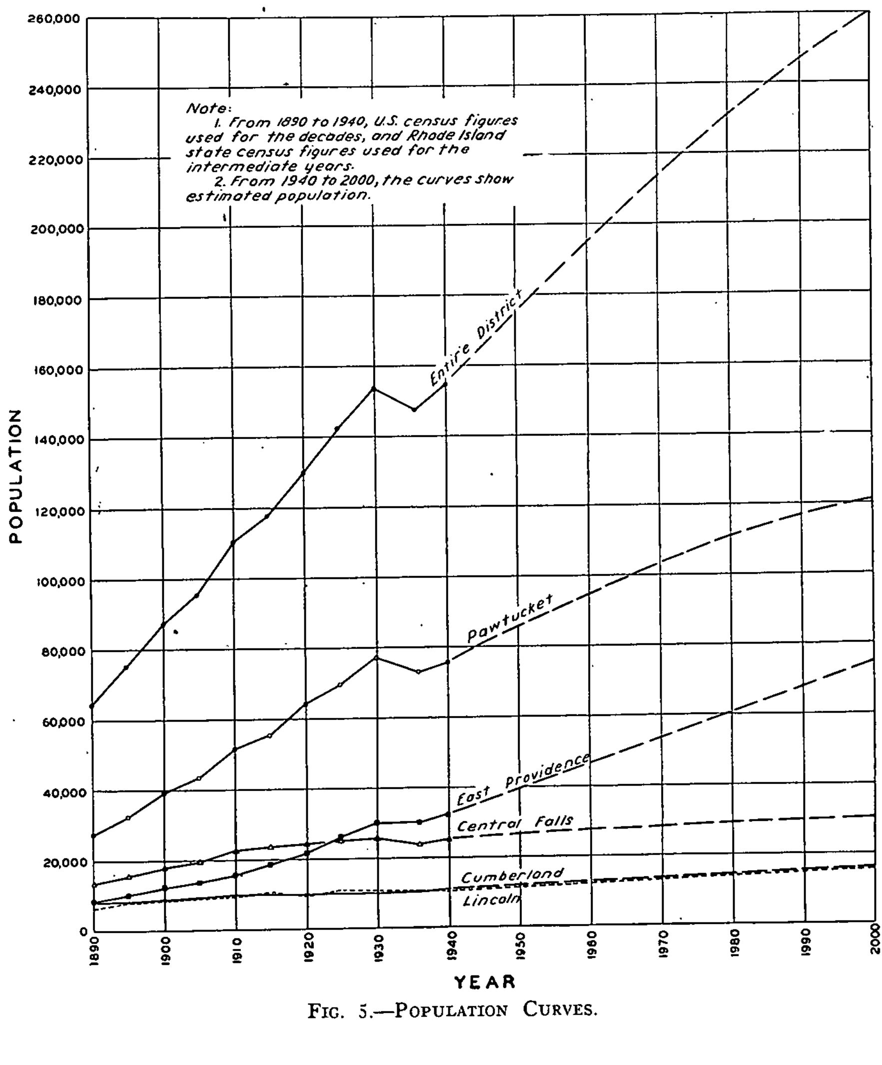


FIG. 5.—POPULATION CURVES.

Water Consumption

As a further aid to the design of the intercepting sewers and sewage treatment plants detailed studies were made of the volumes and rates of consumption of water used for domestic and industrial purposes, from both public and private sources. Broadly speaking there are two sources of water supply used within the District, namely the public supplies taken from ponds and impounding reservoirs for distribution to the general public, and the private supplies taken from wells, ponds, reservoirs and streams for use by individual industries.

The public supplies are owned and operated by the city of Pawtucket, the town of East Providence, the city of Woonsocket, and the town of Cumberland. The total public water consumption in the District from these sources in 1947 averaged about sixteen million gallons daily or about one hundred gallons per capita. The private supplies in the District are estimated to provide nearly thirty-five million gallons per day, equivalent to slightly more than two hundred gallons per capita. The use of water from the private supplies, is therefore twice that from the public sources. The total daily use of water is about fifty-one million gallons of which about 85 per cent is used by industry.

Sewage

Gagings were made and samples were collected of the sewage discharged from fourteen of the principal sewers of the District. These gagings and samples represented about 85 per cent of the total amount of sewage from public sewers, including domestic, commercial, and industrial wastes and ground water infiltration. The total volume of sewage thus gaged was slightly over thirteen million gallons daily, and the total estimated daily discharge was nearly sixteen million gallons. Table 1 summarizes the estimated discharges from sewers in the five municipalities of the District.

The computed analysis of the combined sewage flows, based upon the analyses of 24-hr. composite samples from each of the fourteen sewer outlets, shows a comparatively weak sewage.

Industrial Wastes

The large number of industries in the District necessitated a careful study of their liquid wastes. Replies to a questionnaire sent to practically all industries in the District regarding the types and

TABLE 1.—SUMMARY OF DATA RE SEWAGE FLOWS

Municipality	Maximum rate of discharge	Average rate of discharge
	Million gallons daily	
Cumberland	0.37	0.16
Lincoln	0.94	0.42
Central Falls	5.29	2.63
Pawtucket	18.02	12.29
East Providence	0.90	0.52
Total	25.52	16.02

quantities of their wastes are believed to represent conditions typical of the more important establishments. Personal inspections were made by our engineers of some sixty-seven industries and eighteen of the larger and more significant plants were selected for detailed study. Hundreds of measurements and analyses were made.

Industrial wastes discharged directly to the streams, as gaged and sampled, amounted to slightly more than twenty-two million gallons daily. These flows represented industrial production as of the days of measurement and not necessarily the flows which would be obtained under capacity production. On the basis of information regarding the capacity of those industries at which gagings were made and of those industries at which gagings were not made, it is estimated that with the wastes from all industries included, there were about thirty-four million gallons daily of wastes discharged directly to the streams of the District and about three and one-half million gallons daily to the sewers.

COMBINED FLOWS

A summary showing estimated quantities of sewage and industrial wastes produced in the District at the time of the investigation is given in Table 2, which also shows the general points of discharge of the flows of sewage and wastes.

From this table it will be seen that the total combined volumes of sewage and wastes amounted to about fifty million gallons per day or practically the same volume as the combined volumes of water supplies used within the District.

The probable composition of the combined flows of sewage and

TABLE 2.—SUMMARY OF TOTAL QUANTITY OF SEWAGE AND INDUSTRIAL WASTES, AS OF 1948, AND GENERAL POINTS OF DISCHARGE*

General point of discharge	Maximum rate mgd.	Average rate, mgd.
Blackstone-Seekonk Watershed		
Public sewers	17.01	12.15
Semi-public sewers	1.11	0.49
Industrial wastes directly to streams	68.35	28.59
Total	86.47	41.23
Moshassuck Watershed		
Public sewers	7.19	3.29
Semi-public sewers	0.20	0.09
Industrial wastes directly to streams	13.20	5.80
Total	20.68	9.18
Total for Blackstone-Seekonk and Moshassuck Watersheds	107.15	50.41

*Mills operating with production as of dates of measurement, including "clean wastes."

wastes has been computed on the basis of the actual analyses and on the assumption that the industries were operating at maximum capacity. It was also assumed that about thirteen million gallons of "weak" industrial wastes could be discharged to the streams without interception for treatment with the sewage and "strong" wastes. The computed analysis shows a relatively weak sewage but one distinctly caustic in reaction and therefore one difficult to treat by biological methods. A study of the individual industrial wastes shows that the causticity is due largely to the strongly alkaline wastes of the Sayles Finishing Plants, Inc. The Sayles wastes were also low in suspended solids and high in dissolved organic matter.

DEGREE OF TREATMENT REQUIRED FOR SEWAGE AND WASTES

In considering the degree of treatment required for the sewage and wastes it is essential to understand clearly what it is desired to accomplish. Sewage pollution of waters used for shellfish culture and for recreation results in a distinct hazard to health, particularly in the case of oysters and quahogs produced in Narragansett Bay. Such pollution in the Bay received directly or brought into it by tributary streams has resulted in the closing of certain areas for shellfish harvesting. This results in an economic loss which is accompanied by a health hazard if shellfish are taken unlawfully from polluted areas.

The health hazard due to recreational activities on and in polluted waters is difficult to evaluate but the thought of bathing, boating, and fishing in polluted waters is highly repugnant to most people. This prevents the fullest use of otherwise attractive waters and the elimination of the hazard would be a boon to many people.

Secondary in importance to the health hazard is the occurrence of such gross pollution as to render the waters offensive to sight and smell. Discoloration, floating masses of decomposing sludge, and foul odors are all conditions the abatement of which will add to the enjoyment of life by those who live, work, and travel by and over the streams with which this report deals.

Fully as important and perhaps even more important from the economic standpoint is the adverse effect pollution produces upon process waters for industries. Ill-smelling dirty water is unft for textile processing and acid waters have harmful effects upon boilers, condensers, and hydro-machinery. Clean and abundant water is a fundamental need for industry.

The sections of this report dealing with the sanitary conditions of the rivers show how far they fall short of meeting the requirements set forth above.

In a report to Governor Pastore under date December 23, 1946, Mr. Walter J. Shea, Chief, Division of Sanitary Engineering, makes an excellent statement as to the goal to be sought in the pollution abatement program. This statement is well worth quoting:

> "In planning the future condition of the waters of the state, ideal conditions must in some cases give way to conditions which are less ideal but which are practically obtainable in a state such as Rhode Island which is highly industrialized, densely populated, and enjoys considerable sea commerce. Present day essential uses will not permit the return of all the waters of the state to the condition they were in when Rhode Island was an undeveloped area."

With the above expressed viewpoint in mind, a schedule of planned conditions for the waters of the State was set up by Mr. Shea. That part of the schedule which applies to the waters affected by pollution from the District, is summarized as follows:

Class A, waters suitable for water supply or for cultivation of market shellfish:

Moshassuck River above Barney Pond
Narragansett Bay
Providence River below Bullock and Gaspee points

Class B, waters suitable for bathing:

Blackstone River from Woonsocket to Lonsdale
Moshassuck River between Lincoln Woods Reservation and Saylesville
Providence River from Sabin Point to Bullock Point Ten Mile River

Class C, waters suitable for recreational boating, fishing, culture of seed oysters, or industrial supply after treatment:

Blackstone River from Lonsdale to Main Street, Pawtucket
Providence River from Fields Point to Sabin Point

Class D, waters suitable for commercial navigation or transportation of wastes without nuisance:

Seekonk River
Moshassuck River from Saylesville to the Providence River
Providence River above Fields Point

It is possible that as time goes on, changes in classification of these waters may be found practicable and desirable.

In addition to needed pollution abatement measures outside the District which should be carried out by other agencies, the following broad anti-pollution measures are required within the District if the objectives set forth above are to be met:

1. Interception of sewage and wastes now discharged untreated into the Blackstone and Seekonk Rivers, either directly or through tributaries, between Lonsdale and the mouth of the Seekonk.

2. Treatment of the combined flows of sewage and wastes to the extent of the removal of settleable, sludge-forming solids by a plant located on the east bank of the Seekonk River at Bucklin Point.

3. Interception and treatment at the Bucklin Point plant of the sewage and wastes originating in the Moshassuck valley, with the exception of the wastes from the Sayles Finishing Plants, Inc.

4. Separate treatment of the Sayles wastes by an adequate treatment plant, the wastes to be treated being those now discharged to the Blackstone combined with those going to the Moshassuck.

5. Modernization of the treatment facilities at the Fort Hill plant of East Providence.

6. Installation of a small complete treatment plant for the

southern part of East Providence, when that area becomes sewered, with chlorination of the effluent and its discharge into the Providence River.

7. Installation of short intercepting sewers and individual treatment plants at Manville, Albion, and Berkeley for the small communities upstream from Lonsdale.

SEPARATE OR COMBINED TREATMENT OF SEWAGE AND INDUSTRIAL WASTES

The data which have been collected show that of the total volumes of polluting liquids now produced within the District, industrial wastes constitute about 75 per cent. This fact raises the question as to what policy should be adopted with respect to the acceptance of these wastes for treatment with the sewage of the District.

There are two viewpoints as to this policy. Broadly speaking, one viewpoint is that the wastes should be accepted and the other is that they should not be. These two viewpoints have been given much consideration and a reasonable reconciliation arrived at.

There are many small industries whose wastes are now going into the sewers of the municipalities in which they are located. Obviously it would be difficult if not impossible to avoid acceptance of such wastes. Furthermore, there are a number of industries so situated that individual treatment plants could not be built by them due to limited site area or to location in built-up sections. Where these industries now discharged untreated wastes to the rivers it will be necessary for the Commission to accept the wastes if pollution abatement is to be brought about.

At a few industrial plants the wastes are such that they could continue to be discharged to the river without treatment. At the Lincoln Bleachery and Dye Works there are large volumes of practically clean wash waters which can and should be separated from the more concentrated wastes. Such separation would involve certain changes in piping and the definite allocation of equipment for specific processes so that the operations producing the "clean" wastes would always be kept separate from those producing "strong" wastes. This would permit separate discharge of the two kinds of wastes with the "clean" wash waters discharged to the Blackstone without treatment. The industries from which substantial flows of wastes can be discharged without treatment or with limited treatment are given in Table 3, together with volumes involved.

TABLE 3.—"CLEAN" WASTES WHICH MAY BE DISCHARGED DIRECTLY TO STREAMS

Industry	Location	Present daily flow to be discharged directly without treatment, mg.
American Bitumels Co.	E. Providence	(included with Socony Vacuum Oil Co., below)
Collyer Insulated Wire Co.	Pawtucket	1.20
Kennecott Wire & Cable Co.	E. Providence	0.50
Lincoln Bleachery & Dye Co.	Lonsdale	5.00
Owens-Corning Fiberglass Corp.	Ashton	0.86
Socony Vacuum Oil Co.	E. Providence	4.00
Washburn Wire Co.	E. Providence	1.00
U. S. Oil Co.	E. Providence	0.09
Total		12.65

In the case of Bird & Son, Glenlyon Print Works and Rumford Chemical Works, which produce relatively large volumes of wastes, their sites are either so limited in area or are so located as to render somewhat impracticable the construction of adequate individual treatment plants. Furthermore, their wastes are not such in volume or quality as to affect adversely normal sewage treatment processes if and when secondary biological treatment of sewage and wastes is required.

Of all the industries in the District the Sayles Finishing Plants, Inc., produces the largest volume of objectionable wastes, the inclusion of which with the sewage and other wastes would materially affect the capacity and type of treatment plant required. Inasmuch as the Sayles Company has already undertaken treatment of part of its wastes, it appears logical for that company to continue its policy of separate treatment. At present acid wastes are discharged by the Sayles Company into the Moshassuck and caustic wastes into the Blackstone. These acid and caustic wastes should be combined for as much neutralization as possible and then adequately treated before discharge to the Moshassuck.

Diversion of the Sayles wastes to a treatment plant at Bucklin Point would affect adversely the riparian rights of downstream industries in the Moshassuck Valley due to the fact that the Sayles wastes are derived largely from a water supply obtained from the Moshassuck. Furthermore, in summer the entire flow of the river consists of wastes or effluents and to divert them would be to dry up the river.

Inasmuch as the domestic sewage produced in the Moshassuck Valley is derived from water supplies originating outside the valley, its diversion would not involve riparian rights. As a matter of fact, such diversion has been going on through the Providence sewer system for many years.

Consideration has been given to treatment of the Sayles wastes with the sewage of the Moshassuck Valley. This would involve a joint treatment plant near the present Sayles treatment plant and a sewage pumping station and force main near the Pawtucket-Providence line. The combining of large volumes of wastes with relatively small volumes of sewage would necessitate the installation of an industrial wastes treatment plant rather than a sewage treatment plant. At present it is difficult to foresee exactly what kind of treatment would produce an effluent which would meet the requirements for discharge to the Moshassuck. Although it would be possible to install settling tanks as an initial first step, the combined sewage and wastes would be low in settleable solids and high in oxygen demand. Consequently, settling tanks would bring about comparatively little improvement in the condition of the river.

It was the considered opinion that the Sayles wastes should be treated independently of the sewage and that the Sayles Company should undertake at once studies to determine the type of treatment for their wastes which will produce a satisfactory effluent for discharge to the Moshassuck River. As soon as such studies are completed and conclusions reached as to treatment methods then the company should proceed to install the indicated remedial works. It was also the opinion that the other wastes produced in the District, except those which can be discharged without treatment, can and should be combined and treated with the sewage.

Periods of Design for Intercepters and Treatment Works

It is essential, in considering the problem of pollution abatement, to establish the economical periods of design for the remedial works required. Due to the impossibility of closely determining the future development and growth of the District, as well as construction and operating costs, the economical periods of design are not subject to exact computation but must be based upon judgment and experience.

Because of the large expense of building sewers, the inconvenience to the public during construction, and the fact that the larger the capacity of a sewer the lower its construction cost per unit of capacity, it is advisable to provide for the anticipated needs for a relatively long period in the future. On the other hand, interest charges on the original cost of the structures during the early years of their use, before the territory they serve has reached the anticipated development and the uncertainties involved in the prediction of their development, tend to shorten the time for which it is economical to build. In the case of the intercepting sewers, it is our considered judgment that their design should be based upon conditions expected to prevail about 50 years hence, or the year 2000.

The construction of sewage and industrial wastes treatment works, however, does not inconvenience the public as does sewer construction. Furthermore, changes and improvements in the art and science of sewage and industrial wastes treatment occur from time to time. Large treatment works can be designed and built with multiple units so that additions and enlargements can be made readily as need arises. For these reasons it is uneconomical to design and build treatment plants for as long a period of time in the future as in the case of intercepting sewers. It was, therefore, decided that the design of treatment works should be based upon conditions anticipated for the year 1975, or about 25 years hence.

Interception of Sewage and Wastes

Several different plans have been studied for intercepting the sewage and wastes of the various municipalities and industries of the District. The recommended plan is outlined on Fig. 1 and described in the following paragraphs.

A main intercepter will be located along and in the vicinity of the Blackstone River from a point in the town of Cumberland north of the village of Lonsdale to Bucklin Point in East Providence. This sewer is designated the "Blackstone Valley Intercepter." The proposed sewer will intercept the dry-weather flow plus a portion of the wet-weather flow from all the existing sewers of Central Falls and Pawtucket which discharge into the Blackstone River.

This sewer will have sufficient capacity to enable it to carry off all the wastes from the industries located within its tributary area, except for certain "clean" wastes which can be discharged to the streams. The intercepter will also have capacity to carry the flows discharged into it from tributary branch intercepters. The total length of the Blackstone Valley Intercepter will be about seven miles and its greatest diameter will be 7 ft. 9 in.

Another main intercepter will be required in the Moshassuck Valley extending from a point in Central Falls near the intersection of Lonsdale Avenue and Emmett Street to the intersection of Esten Avenue and Moshassuck Street in Pawtucket. From this point the sewage will be conveyed to the Blackstone Valley Intercepter by means of a sewer running in an easterly direction, in tunnel beneath the divide separating the Moshassuck and Blackstone Valleys and in an inverted siphon under the Seekonk River.

The Moshassuck Valley Intercepter will take the flow from the existing separate sewers of Central Falls tributary to the existing sewage treatment plant of that city. In addition it will take the dry-weather flow plus a portion of the wet-weather flow from an existing "storm drain" of Central Falls which now discharges domestic and other dry-weather sewage into the Moshassuck River. Various existing sewers of Pawtucket in the Moshassuck watershed will also be intercepted. The intercepter will have sufficient capacity to carry all the wastes from industries in the tributary area with the exception of the wastes from the Sayles Finishing Plants, Inc., and certain clean wastes from other plants. The total length of this intercepter within the Moshassuck Valley will be about three miles and its maximum diameter will be 4 ft.

The Blackstone Valley Intercepter will serve an area of about 10,250 acres and an estimated population of about 172,000 in the year 2000. Of the total area and population about 3,500 acres and 56,000 persons will be served by the Moshassuck Valley Intercepter.

Another intercepter will be needed to serve the northern section of East Providence and the now unsewered portion of the central section of that town. This sewer, called the "East Providence Intercepter," will extend from the intersection of North Broadway and Roger Williams Avenue to Bucklin Point. In general it will be located along the southerly side of Ten Mile River and Omega Pond and the easterly side of Seekonk River. A pumping station will be needed near the outlet of the pond to lift the sewage through a force main to the high point in Bourne Avenue. From this point the sewage will flow by gravity to Bucklin Point.

At present the area to be served by this intercepter is unsewered. The proposed sewer will have sufficient capacity to carry off the sewage eventually produced in a tributary area of about 3,000 acres and from an estimated population of about 27,000 (year 2000). It will also carry the industrial wastes of the area except for those clean wastes which can be discharged directly to the streams. The total length of this intercepter will be about a mile and a half. Its maximum diameter will be 4 ft. 9 in.

In addition to these three main intercepters three smaller intercepters will be required eventually for portions of Cumberland and Lincoln located along the Blackstone Valley northerly of New Pond. These sewers will discharge to treatment plants located near the villages of Manville, Albion, and Berkeley, respectively. The population to be served by these sewers in the year 2000 is estimated as about 11,000 and the areas tributary thereto about 1600 acres. The total length of the three intercepters will be approximately four miles.

The sewage from central East Providence is now brought to a treatment plant at Fort Hill. Eventually the southern part of East Providence, now unsewered, will be provided with sewers, at which time the sewage should be brought to a plant at Riverdale for treatment before discharge to the Providence River. The populations expected to be tributary to these two plants in East Providence in the year 2000 are about 25,000 and 23,000 respectively. The corresponding areas served will approximate 1600 and 2100 acres.

Treatment Plants

Under the recommended plan for the interception and treatment of the sewage and wastes of the District the bulk of sewage and wastes will be brought to a treatment plant located at Bucklin Point. Sewage from the central part of East Providence will be treated in a modernized plant at Fort Hill. Eventually small treatment plants will be required at Manville, Albion and Berkeley on the Blackstone River and for the southerly part of East Providence at Riverside. These six plants will treat all the sewage and wastes produced in the District except for certain "clean" wastes which can be discharged directly to streams and except for the wastes of the Sayles Finishing Plants, Inc., which should be treated by an adequate plant constructed and operated by that company.

The Bucklin Point treatment plant will involve Parshall flume measuring devices, mechanically cleaned bar screens, screenings, grinders, grit channels, pre-aeration tanks, mechanically cleaned sedimentation tanks, chlorination facilities, sludge digestion tanks, sludge elutriation tanks, vacuum filters for sludge dewatering and incinerator for disposal of sludge cake. The buildings at the plant will consist of a screen house, administration building, chlorination house, sludge dewatering building and incinerator building.

The general plan of the plant is shown on Fig. 6. From this plan it will be seen that provision is made for obtaining more com-

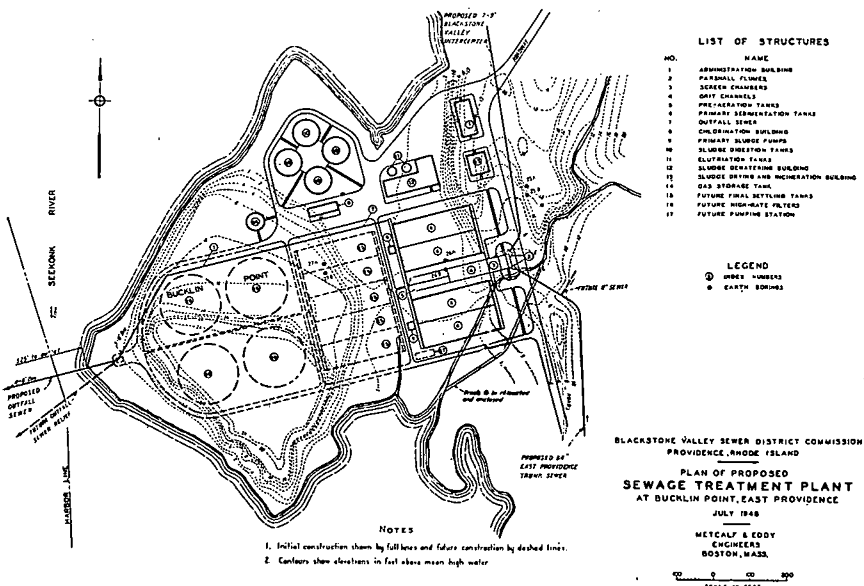


FIG. 6.—PLAN OF PROPOSED SEWAGE TREATMENT PLANT.

plete treatment by means of high-rate trickling filters when and if such additional treatment is required.

The plant is designed to be adequate for conditions anticipated to prevail in the year 1975. As of that date the population to be seved is estimated at 158,000 and the total average flow to be treated is expected to reach 47 mgd. The industrial wastes will materially increase the flows, suspended solids and oxygen demand of the sewage reaching the plant as compared to normally expected contributions from the sewered population. This is shown by the following table:

TABLE 4.—ESTIMATED CONTRIBUTIONS PER CAPITA PER DAY—SEWERED POPULATION

Item	Normal sewage	Sewage at Bucklin Point
Average flow, gal.	100	336
Suspended solids, lb.	0.20	0.35
5-day B.O.D., lb.	0.17	0.44

As a result of these contributions by industry the size of the plant with its various facilities is much greater than that which would be required for the treatment of sewage alone. In this connection it is important to note that, due to the industrial wastes, the total organic load for which the treatment plant would be designed is equivalent to that from a human population of about 400,000.

The effluent from the treatment plant will be discharged into the Seekonk River at a point near the navigation channel, about 525 ft. from the shore line.

The modernized treatment plant at Fort Hill for the sewage of the central part of East Providence would consist of grit chambers, measuring flume, comminutors, a two-compartment, mechanically-cleaned settling tank, heated separate sludge digestion tanks, and open sludge beds. The present tanks would be used partly for storage of sludge and partly for chlorine contact. The present building would be utilized insofar as possible. Eventually a parallel trunk sewer will be needed to bring anticipated increased sewage flows to the plant and an additional outfall sewer to the Seekonk River will be required.

The treatment plant required some time in the indefinite future at Riverside in the southern part of East Providence will involve complete treatment with chlorination of the effluent and its discharge through a submerged outlet into the Providence River.

The three small treatment plants which may eventually be required between Woonsocket and Lonsdale would involve treatment by Imhoff tanks, high-rate trickling filters, and chlorination.

COST ESTIMATES

The estimates of cost for structures, sewers, and appurtenances have been based upon unit costs as determined from proposals for similar work for construction during 1948. The costs of tunnel work and special structures and equipment have been estimated from data for many projects built over a comparatively long period, with adjustments to allow for the higher costs in 1948.

Construction costs have increased rapidly since the end of the last war and it is probable that they will continue to increase. It is not possible to determine closely the cost of work to be done in the future. At the present time general contractors have plenty of work and there is a tendency toward a scarcity of labor and delay in delivery of materials.

Designs have been worked out to determine the required capacities and sizes of structures and sewers and preliminary locations have been selected and profiles prepared. When detailed designs shall have been prepared based upon additional detailed surveys and more extensive underground explorations it will be possible to prepare relatively close estimates of cost for work to be done in the near future.

The estimates of cost herein are intended to be applicable to conditions and the cost of labor and materials as of April 1948. Allowances have been made for contingencies, engineering for detailed designs, engineering and inspection during construction, and land and rights-of-way. No allowance, however, has been made for the overhead expense of the Sewer Commission or for financing and legal expenses.

Summarized Construction Costs

The estimated construction cost for the entire pollution abatement program is $15,662,000, divided into the major items as given in Table 5.

Table 5.—Total Estimated Construction Cost of Entire Sewerage Project

Blackstone Valley Intercepter and Branches		$ 3,875,000
Moshassuck Valley Intercepter and Branches		1,348,000
East Providence Intercepter and Pump. Sta.		537,000
Bucklin Point Sewage Treatment Plant*		
a. Primary Treatment	$4,460,000	
b. Secondary Treatment, additional	2,895,000	
		7,355,000
Manville, Albion and Berkeley Intercepters and Treatment Plants		1,087,000
East Providence, Fort Hill Treatment Plant*		440,000
East Providence, Riverside Treatment Plant*		1,020,000
Total		$15,662,000

*Including outfall.

Summarized Annual Charges

The estimated fixed, operating and total annual charges for the entire program are given in Table 6, based on 20-yr. serial bonds and 2 per cent interest.

Table 6.—Total Annual Charges for Entire Sewerage Project

Item	Fixed charges	Operating costs	Total charges
Sewers and Pumping Stations	$448,100	$ 47,300	$ 495,400
Treatment Plants	491,700	346,000	847,700
Total	$939,800	$393,300	$1,333,100

This table shows the heavy fixed charges resulting in part from the fact that retirement of bonds takes place in 20 years although a large portion of the construction costs are for structures useful for at least 50 years.

Progressive Development of Project

In view of the large expenditure required to carry out the entire project and in view of the limited funds now available, it was recommended that the program of pollution abatement be carried out progressively in successive steps as funds can be obtained.

As the first step of the entire program, it was suggested that the following sewerage works be built:

1.	Bucklin Point primary treatment plant of one-half 1975 capacity	$2,440,000
2.	East Providence intercepter, Sections A and B	247,000
3.	Omega Pond pumping station	175,000
4.	Blackstone Valley intercepter, Sections A and B	1,764,000
5.	Taft St.-Pleasant St. branch intercepter, Sections A and B	685,000
6.	Bucklin Brook branch intercepter	13,000
		$5,324,000

The cost estimate for this first step is somewhat in excess of available funds, but it is probable that final designs could be worked out in such a way as to keep actual costs below the preliminary estimates.

This first step would permit the interception and treatment of about 23 mgd. or 72 per cent of the 32 mgd. of industrial wastes and sewage now produced within the District which requires treatment, exclusive of wastes from the Sayles Finishing Plants, Inc.

The estimated annual charges, based on an estimated average sewage flow of 20 mgd. intercepted and treated, are shown in Table 7.

TABLE 7.—SUMMARY OF ESTIMATED TOTAL ANNUAL CHARGES FOR FIRST STAGE OF SEWERAGE WORKS

Item	Fixed charges	Operating costs	Total
Sewers	$162,000	$ 8,000	$170,000
Omega Pond Pumping Station	10,500	31,500	42,000
Bucklin Pt. Treatment Plant	146,400	112,100	258,500
Total	$318,900	$151,600	$470,500
Per million gallons (Average = 20 mgd.)	$43.80	$20.70	$64.50

*Based on 20-year serial bonds at 2 per cent interest.

The second step of the entire Project would involve the following items and costs:

1.	Bucklin Point, primary treatment completed to 1975 design capacity	$1,720,000
2.	Bucklin Point outfall sewer	300,000
3.	Blackstone Valley Intercepter, Sections C and D	1,134,000
4.	East Street and Roosevelt Avenue branch intercepters	70,000
5.	Moshassuck Valley Intercepter	1,151,000
6.	Central Falls branch intercepter	42,000
	Total	$4,417,000

The third and final step would involve the following items:

1.	Bucklin Point secondary treatment plant	$2,780,000
2.	Bucklin Point outfall sewer relief	115,000
3.	Blackstone Valley Intercepter, Section E	153,000
4.	Mill St. and Concord St. branch intercepters	127,000
5.	Moshassuck Valley Intercepter, Section C	84,000
6.	East Providence Intercepter, Section C	115,000
7.	Manville, Albion and Berkeley intercepters and treatment plant	1,087,000
8.	East Providence, Fort Hill treatment plant	440,000
9.	East Providence, Riverside treatment plant and outfall	1,020,000
	Total	$5,921,000

It should be appreciated that modifications in these three steps in the whole program can and probably will be made as conditions warrant.

Apportionment of Costs

Under Sections 24 and 25 of the 1947 Act (Chapter 1837) the Commission can assess the municipalities and industries within the District reasonable charges for use of the sewerage project. After considering the volumes and character of the sewage and wastes produced in the District, it was decided that the simplest and most easily applied method of assessment would be based upon volumes of sewage and wastes delivered by each municipality and by each industry connected directly with the District's sewers. Provision should be made in any schedule of rates for penalty charges in case of extraordinary concentrations of impurities, such as suspended solids or oxygen demand, in the sewage and wastes received by the District's intercepters.

Charges would be computed by dividing the total annual costs, fixed and operating, by the total volume intercepted and treated in order to obtain the total cost per unit volume. Table 7, based on an estimated average volume of 20 mgd. cared for by the proposed first stage of the project, indicates that a charge of about six and one half cents per thousand gallons of wastes and sewage would provide the necessary funds. During the first years of operation, the charge would need be somewhat greater as the volumes to be received by the works would be less than the average assumed and the total costs would be about the same irrespective of volumes handled due to higher interest charges and fixed bond retirement payments offsetting lower operating costs. Obviously the actual established charges must be deferred until costs are known more accurately than at present.

It is also possible that industries upon finding it necessary to pay for the treatment of wastes now being discharged to streams or sewers without direct cost will economize in the use of process water and thus reduce the volume of industrial wastes. This, however, is by no means an assured fact but must be considered as a possibility only.

Recommendations

The recommendations for the abatement of water pollution in the Blackstone Valley Sewer District may be summarized as follows:

1. The interception of the bulk of the sewage and industrial wastes originating within the District for treatment at Bucklin Point. The intercepting sewers should be designed for conditions expected to prevail in the year 2000.
2. The construction of a modern treatment plant of 47-mgd. capacity at Bucklin Point for the removal of sludge-forming solids and the chlorination of the effluent before discharge through a submerged outlet into the Seekonk River.
3. The construction of a new modern primary treatment plant of 1.5-mgd. capacity at Fort Hill for the treatment of the sewage and wastes from the central part of East Providence.
4. The separate and adequate treatment by the Sayles Finishing Plants, Inc., for their industrial wastes before discharge into the Moshassuck River.
5. The construction of a complete treatment plant at Riverside when a substantial number of sewers are about to be built in the now unsewered southerly part of East Providence.
6. The construction of complete treatment plants at Manville, Albion, and Berkeley for sewered areas below Woonsocket and above Lonsdale.
7. The construction of secondary treatment features at Bucklin Point when conditions warrant.
8. The installation of measuring devices at points where the intercepters cross municipal lines and where industries are directly connected to sewers built by the Commission.
9. The establishment of service charges computed by dividing the total annual costs, fixed and operating, by the total volume of sewage and wastes intercepted and treated to obtain a charge per unit volume.
10. The taking of steps at an early date to acquire the Bucklin Point treatment plant site and the other sites at indicated locations.
11. The construction of the total project in three successive steps involving approximately equal expenditure, now estimated at \$5,324,000, \$4,417,000, and \$5,921,000 respectively, the exact extent of each step to be determined by funds available and coincident construction costs.

Acknowledgments

In conclusion the writer wishes to acknowledge the interest and cooperation of the Commission, of its Chief Engineer, Mr. Hammann, of its Executive Secretary, Mr. Bonvicin, of Mr. Shea, Chief Sanitary Engineering Division, State Department of Health, and of the staff of Metcalf & Eddy.

SOIL MECHANICS IN THE DESIGN AND CONSTRUCTION OF THE LOGAN AIRPORT

By A. Casagrande, Member*

(Presented at the Fall Meeting of the American Society of Civil Engineers, held in Boston.)

General

During the past four years a new airport was built in the Boston Harbor on man-made land which required about 40 million cubic yards of fill, most of it hydraulic clay fill. Many challenging problems in applied soil mechanics presented themselves during the design and construction of this project.

The air traveler who sees now the long, wide runways on which transatlantic planes are operating daily (see the aerial photograph Fig. 1b), may find it difficult to visualize the former appearance of this part of the harbor when most of the area was under water (see Fig. 1a), Governor's Island with its fortification works was still standing out as a prominent landmark, and the existing airport covered only the small area which now forms the southwest corner of the new airport.

Most of the new fill consists of Boston Blue Clay which was dredged hydraulically from adjacent harbor areas and pumped through pipe lines to build up 500 foot wide embankments for the runways. Fig. 2 shows a typical view of this fill during construction. In the background of this photograph can be seen an observation platform which was built prior to filling for the purpose of installing piezometers to measure the pore pressures in the foundation clay and in the clay fill, and underground settlement observation points to measure the compression of the clay fill and the various foundation strata. Analysis and interpretation of these measurements forms a principal topic of this paper.

A hydraulic clay fill differs in character from a natural clay deposit inasmuch as it consists of balls of clay varying from pebble to head size, which are laid down in a matrix of semi-fluid clay. The

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close-up in Fig. 3 conveys a good picture of the appearance of the freshly deposited clay fill. The pebbles resembled so closely the shape of coarse, well-rounded river gravel that some visitors had to convince themselves, by cutting some up, that they really contained plastic clay. Even though these pebbles and balls consisted of medium-plastic clay, the mass as a whole was rather unstable due to the matrix of semi-fluid clay, and even a light caterpillar tractor would often sink and mire down.

At the start of this project, in the summer of 1943, I was confronted with questions which were in essence: Can one build a hydraulic fill of Boston Blue Clay on top of a thick foundation of the same clay, such that this fill will form a satisfactory foundation for runways which are to carry the heaviest planes?

At that time the impact of the news about the continuous sinking of the LaGuardia airport caused others to question seriously whether one should risk that something similar might happen to this aiprort. However, the advantages of an airport so close to the center of the city were obviously most desirable and Governor Saltonstall and the State engineers concluded that the airport should be built at this site, if at all feasible from an engineering standpoint.

Evidence on the behavior of hydraulic clay fills, particularly on the speed with which the upper layer would consolidate to a sufficient extent to become suitable for runway construction, was extremely meagre. The existing Boston airport, which now forms but a small corner of the new airport, had also been built as a hydraulic clay fill, but it had been standing for many years before it was finally developed as an airport for light planes.

Since there was no reliable, quantitative information on the consolidation characteristics of hydraulic clay fills, the desirable procedure would have been to build first a full-size test section and keep it under observation for whatever period might be needed. However, the construction of Idlewild airport was then well under way and the State authorities were fearful that Boston would lose out in the race for transatlantic traffic. Therefore, all possible speed was urged upon the State engineers and, in the absence of empirical data, the soil mechanics consultant had to find the answers by raking his brains in sleepless nights. Under these circumstances some important decisions on the design of the runways were deferred until construction was well under way and until large-scale tests on the clay fill were

FIG. 1A.—SITE OF LOGAN AIRPORT BEFORE CONSTRUCTION OF NEW FILL.

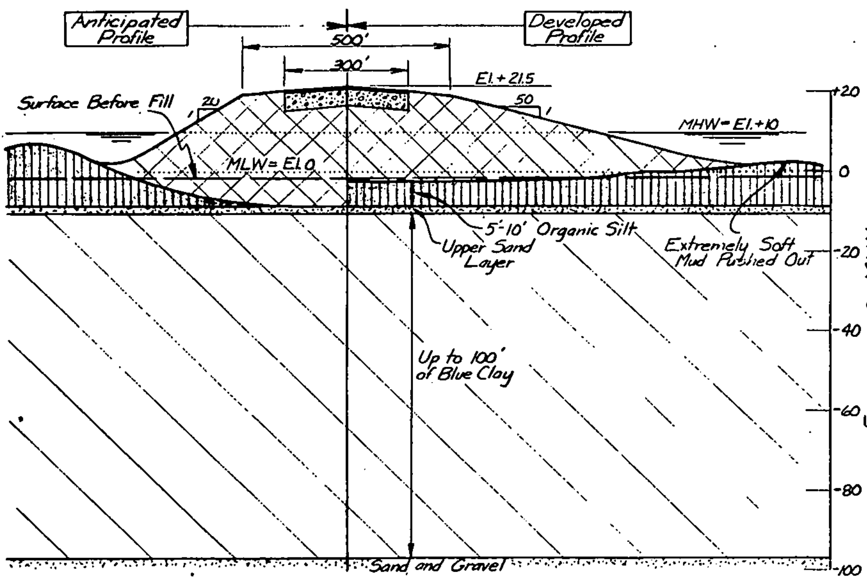
FIG. 1B.—AERIAL PHOTOGRAPH OF LOGAN AIRPORT IN EAST BOSTON.

FIG. 2.—TYPICAL VIEW OF HYDRAULIC FILL; OBSERVATION PLATFORM NO. 1 IN BACKGROUND. (Photograph by E. N. Hutchins.)

FIG. 3.—CLOSE-UP OF FRESHLY DEPOSITED CLAY FILL. (Photograph by E. N. Hutchins.)

made and analyzed. It cannot be denied that this initial reliance on judgment involved a certain amount of risk that the fill would not develop sufficient strength in a reasonable length of time, or that the differential settlements would be excessive for the satisfactory operation of runways.

Fig. 4 shows the subsoil profile with the original ground surface



COMPARISON OF ANTICIPATED AND DEVELOPED PROFILES THROUGH RUNWAY EMBANKMENTS

FIG. 4

slightly below mean low tide. Superimposed in this drawing are, on the left half, the profile of a runway embankment as it was anticipated and, on the right half, as it actually developed.

Most of the area was covered with a surface layer of very soft organic silt-clay averaging 5 to 10 ft in thickness. Directly beneath this harbor silt was found in many borings a thin layer of sand, and then followed the typical Boston clay profile, starting with stiff yellow or medium blue clay and changing with depth to the well known soft blue clay. The site was crossed by former ship channels which had been dredged to about 15 feet below mean low tide and which complicated the problems. The most encouraging characteristic of this profile was the depth to which the clay had been consolidated by partial drying due to geological conditions which existed several thousand years ago when the surface of the clay was above sea level. From detailed tests on boring samples it was established that to the depth of dredging the clay was preconsolidated to a considerably greater stress than the stress to which it would be exposed in the fill. Therefore, I concluded that compression of the clay fill would be largely due to plastic deformation of the clay pebbles and balls rather than to consolidation of the clay itself, and that this adjustment would develop at a considerably faster rate than the primary consolidation of a natural clay deposit because of the relatively pervious matrix.

The question arose whether to remove and waste the soft organic silt-clay, generally referred to as silt, which covered the borrow areas and the site of the fill. I was convinced that the silt from the borrow areas would be liquified to such an extent on its way through the pipe lines that it would drain off the fill and not form any harmful intrusions. Experience proved this assumption correct and therefore all dredging was carried out without prior removal of the silt in the borrow areas.

More difficult was the question of what to do with the organic silt-clay covering the site to be filled. The dredging contractors who were consulted were certain that this silt would be largely displaced by the weight of the fill. I anticipated difficulties and recommended removal. It was finally decided not to excavate the silt. Check borings after completion of the fill showed that for practical purposes the silt remained beneath the fill, with only local displacements. In retrospect I believe that on this project it was just as well to leave the silt in place, even though it is responsible for a substantial amount of the differential settlements which developed, because during construction radical changes in the layout of the runways were made. If the silt had been removed according to the original layout, it would have resulted in an embarrassing situation. However, I believe that for similar conditions, where the layout of paved areas is definitely fixed, the cost of removel of such organic silt would be warranted, since it would reduce considerably the differential settlements of the pavements.

To counteract the settlements which were expected to develop prior to the completion of the pavements, it was decided to fix the placement elevation for the clay fill two feet higher than the desired elevation. This was a lucky guess, since that was just about the average settlement of the surface of the clay fill prior to completion of the pavement. In contrast, the estimates made about the slope on which the clay fill would stand were not correct. Based on their extensive experience, the contractors thought that the fill should stand on slopes of about 1 on 20. Actually, the average slope was about 1 on 50. This resulted in a considerably larger volume of fill placed than was included in the estimates.

Pore Pressure Measurements and Analysis of Compression of Foundation Clay

Since the settlements of the fill surface are the combined result of the compression of the clay fill, of the underlying trapped organic silt-clay and of the foundation clay, a rational analysis of the settlements would not have been possible on the basis of settlement observations on the surface only. Necessary observations for analyzing the progress of consolidation were measurements of the pore pressures and vertical displacements at various elevations in the foundation clay and clay fill.

The difficulty with pore pressure measurements had heretofore been that ordinary standpipes were not sensitive enough for measurements in clay. On the other hand, various electrical instruments which had been invented had until then proved unsatisfactory within a short time after installation. Another objectionable feature in some of the pore pressure installations was the development of gas in the pore pressure points which seriously affected the measurements. In my opinion, this is due to galvanic action of the metals of which such piezometers are usually made.

After extensive and sometimes disheartening experimentation, both in the field and in the laboratory, we developed a piezometer which proved satisfactory for measurements in clay and which is simple and inexpensive. It combines the principle of a standpipe with the following requirements: (1) use of a piezometer point and tubing which contain no metal; (2) use of a piezometer point which has sufficient contact area with the clay to ensure rapid response of the water level in the standpipe with changing pore pressures; (3) use of a standpipe which is as thin as possible, but still permits accurate measurements of the water level; (4) use of a positive seal between the plastic standpipe tubing and the casing inside of which the piezometer is installed.

The Appendix contains a description of such a piezometer and instructions for its installation. Successful operation of such a piezometer depends to a large extent on careful attention to all the details enumerated in the Appendix.

For the purpose of measuring the water level inside the tubing, a flexible electric wire, which is weighted with lead, is lowered until the contact points touch the water. At that moment the contact is registered sharply on the sensitive ohmmeter to which the wires are attached. In contrast to other electrical water level observations inside of wells in which earphones, lights or ammeters are used, we have found that the use of an ohmmeter has the important advantage that the measurement of the water level can be repeated any number of times without diminishing accuracy, since measuring the electrical resistance produces only a negligible current between the points which will not cause formation of gas bubbles at the contact points.

A number of these pore pressure observation points have been installed in the clay fill and clay foundation on this project and have now been operating successfully for several years. I might mention in passing that the success we have had with this installation has led to the use of this type of piezometer on several earth dam and building foundation projects.

A typical set of pore pressure observations is shown in Fig. 5. It is interesting to note that the rise in pore pressures developed not only when fill was placed at the location of the points, but also during filling operations at a considerable distance. Note, for example, the rapid rise in pore pressures during the "4th Fill", the bulk of which was about 300 ft from the piezometers. The thin sand layer, which more or less continuously underlies the silt, rapidly transmits any increases in pore pressures that develop in some other areas. The increase in pressure is transmitted into the adjacent clay, thereby resulting not only in a rise in pore pressures but actually in a temporary swelling of the clay as evidenced by a slight rise in the settlement observation points.

The principle of the use of the pore pressure observations is illustrated in Fig. 6. When a sudden increase in load is applied on a clay layer, this load is at first carried by the water in the pores of the clay. Gradually, as consolidation takes place, the pore pressure drops until it finally reaches again the normal hydrostatic pressure. For example, if we know that a given elevation the pore pressure has

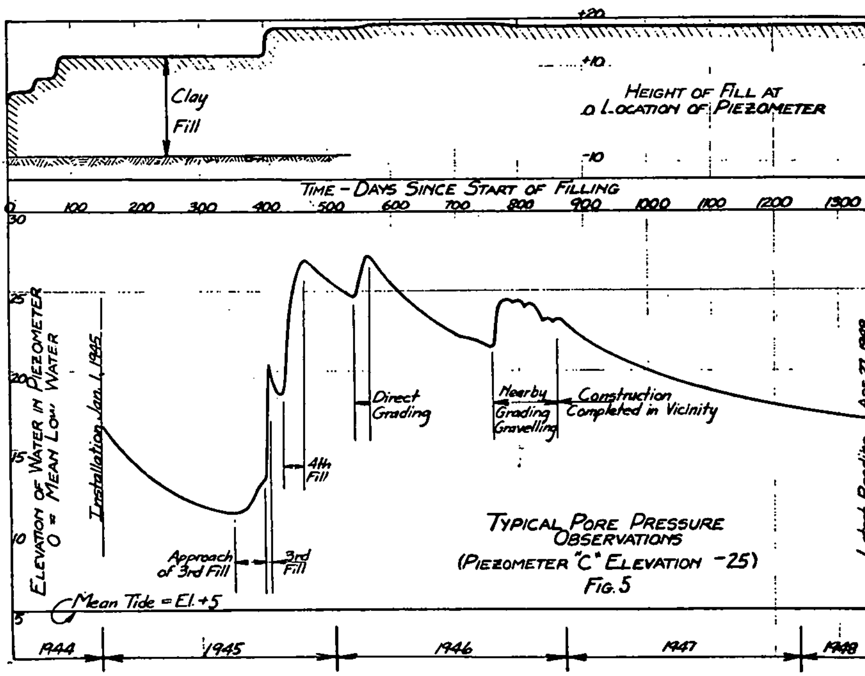


Fig. 5.

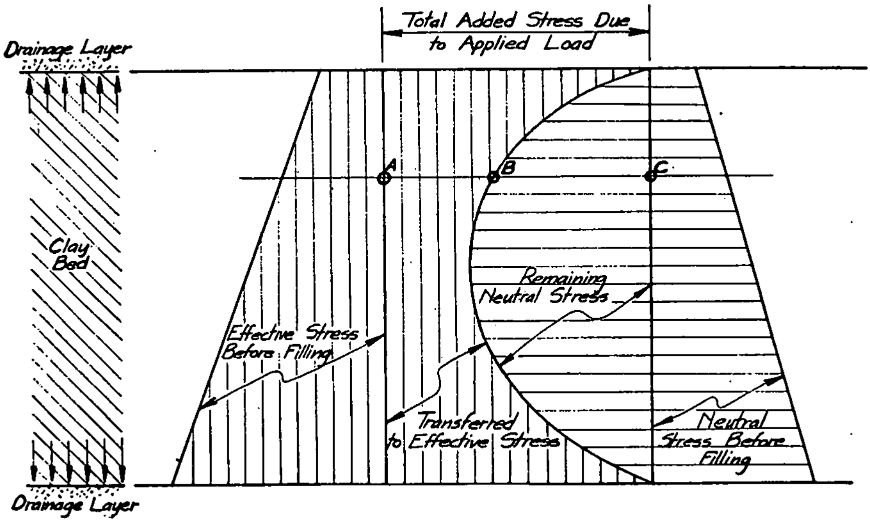


Fig. 6.—Diagram Illustrating Analysis of Pore Pressure Observations.

dropped from its maximum value AC to the value BC, we can estimate how much of the compression due to primary consolidation has developed at that elevation.

An example of the analysis of the observed pore pressures is shown in Fig. 7. The left boundary of the area indicates the total

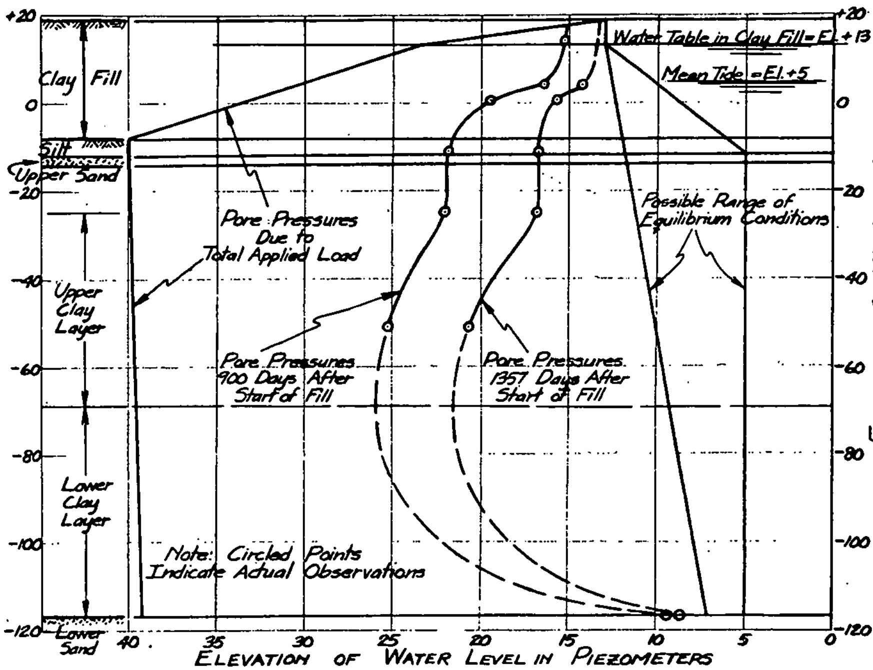


FIG. 7.—TYPICAL ANALYSIS OF OBSERVED PORE PRESSURES.

applied pressures due to the load of the fill. The left curve shows the observed pore pressures 900 days after the start of filling. The right curve shows the pore pressure distribution 1,357 days after the start of filling. The final condition of equilibrium in pore pressures is shown by the right side boundary lines. They represent two possible extremes. It can be seen that at about half of the depth of the clay fill the pore pressures have almost reached a condition of equilibrium. On the other hand, in the middle of the foundation clay stratum, about one-half of the load of the clay fill is still being carried by the pore water.

The observations of the settlement points installed in the foundation clay were correlated with the pore pressure measurements and utilized to derive curves, as for example in Fig. 8, correlating com-

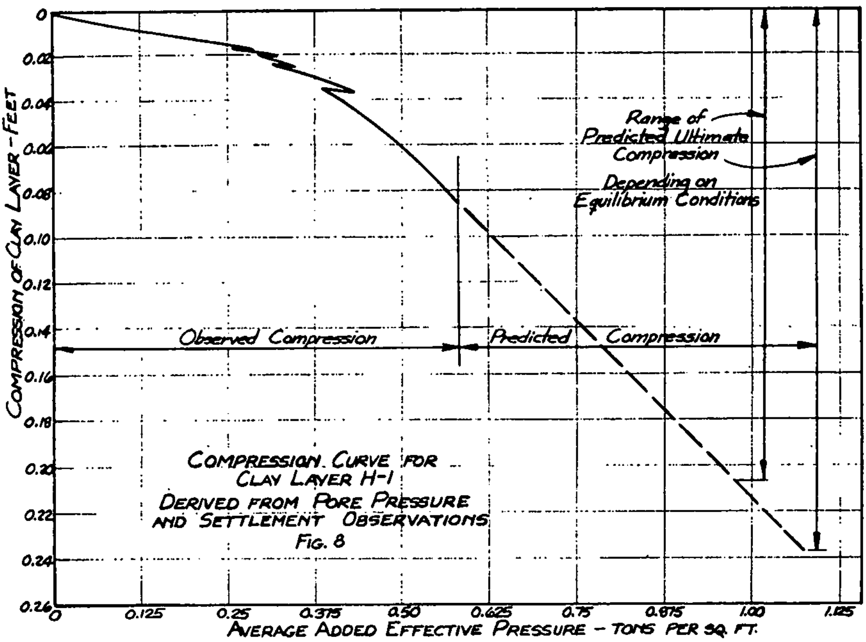


FIG. 8.

pression (decrease in thickness) with effective pressure (stress carried by soil grains). Such a curve is inherently the same as a pressure-void ratio curve obtained from a consolidation test. The small zig-zags which interrupt the curve in Fig. 8 correspond to a cycle of loading at some distance from the observation points. Such remote loading causes a temporary increase in pore pressure without change in total stress at the piezometer location, and therefore a corresponding decrease in the pressure carried by the grain skeleton. The zig-zags in Fig. 8 are similar to what would happen in a laboratory consolidation test as a result of a small unloading and reloading cycle.

Without going into further detail, the final results of this analysis for the foundation clay are shown in Fig. 9. The total compression of the foundation clay to date is about 0.2 ft. About one-half of this value occurred since completion of the pavement. In the next five years the additional compression will be about 0.1 ft. Thus it can

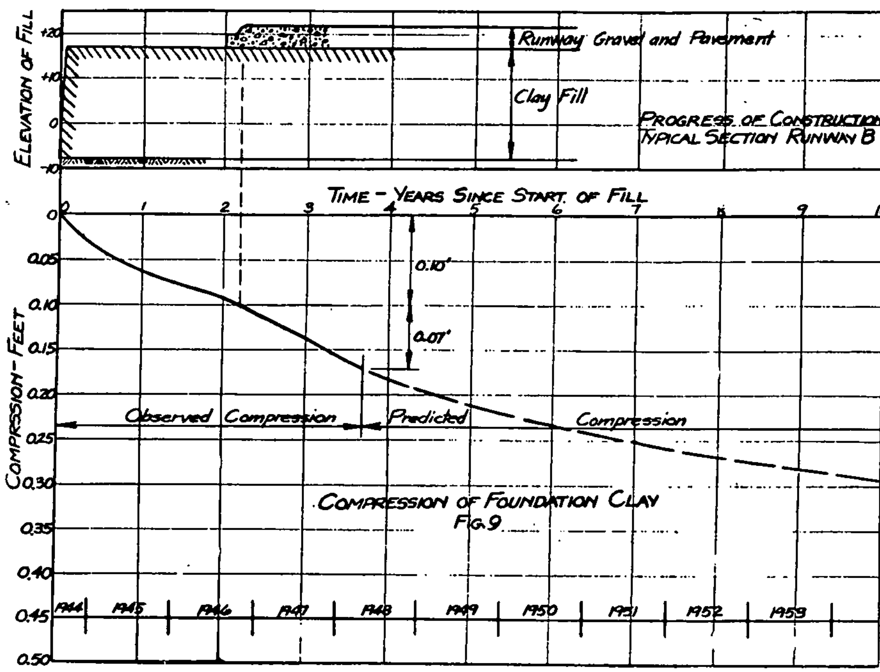


Fig. 9.

be seen that the contribution of the clay layer to the surface settlements is small. Furthermore, these settlements are very regular, so that their contribution to the differential settlements of the pavements is negligible.

Compression of Hydraulic Clay Fill and Underlying Silt

It was not difficult to predict that the major amount of the settlements would be due to the compression of the hydraulic clay fill. However, because of complete lack of empirical data about the compressibility of hydraulic clay fills, the estimate that this fill would settle 2 to 4 ft was largely a guess. The prediction that the organic silt would compress about 10% of the thickness of any trapped silt was based on test results and experience with similar materials on other projects. I also assumed that most of the compression of the clay fill and of the organic silt would develop during the first year after placement of the fill.

Because of these uncertainties extensive settlement observations were made on the fill surface in the hope that it would be possible to extrapolate from such observations the probable future development of the settlements, to serve as a guide for maintenance operations and for any future construction.

Typical settlement observations are plotted in Fig. 10. It can

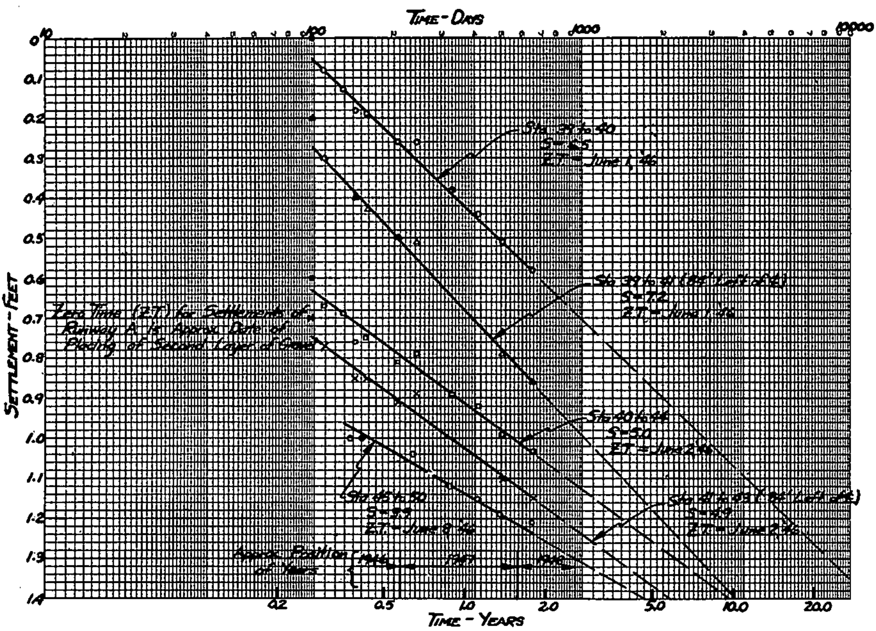


FIG. 10.—TYPICAL OBSERVED SETTLEMENTS FOR RUNWAY "A".

be seen that the settlement vs. time curves are practically straight lines on a semi-log plot. So far as the compression of the clay fill is concerned, this result was not unexpected, since this is characteristic of settlements due to plastic deformation. Furthermore, the compression of the organic silt has a very large secondary time effect which also follows a straight line on a semi-log plot. As shown before, the compression of the foundation clay is so small that it would not materially affect the shape of the settlement vs. time curves.

The observations in Fig. 10 are those after completion of the pavement. Similar observations, made for the period before construction of the base and pavement, showed the same pattern.

Without going into a detailed presentation, I should like to sum-

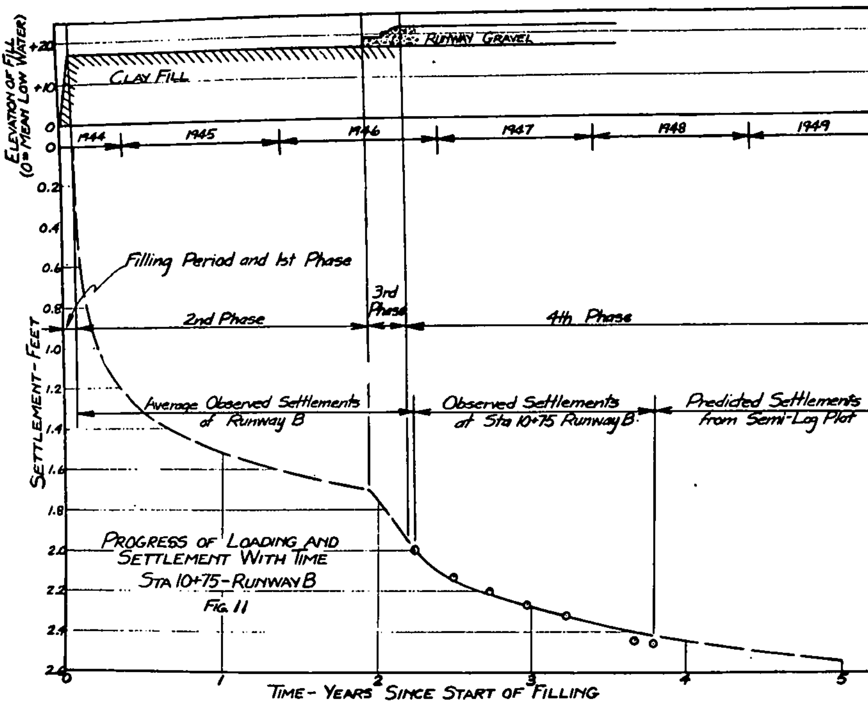


FIG. 11.

marize the final results of all these settlement studies, using Fig. 11 as an illustration for the average conditions. Four phases of settlement can be distinguished:

The first phase lasted about a day, or less, after hydraulic filling was completed. In this period some irregular subsidence due to shear deformation in the fill took place. Although generally not of a serious nature, it was occasionally large enough to require refilling of an area which had slumped. The irregular settlements which occurred during this first phase are not shown in Fig. 11.

The second phase is the period during which the fill rested, that is until the start of placement of the gravel base. On an average this period lasted about 2.5 years and the average subsidence of the surface was about 1.7 ft.

The third phase is the period in which the gravel base and pavement were built; a period which lasted only a few months and in which the settlements averaged 0.3 ft.

The combined settlements of the first three phases influence the general elevation of the fill surface, and must be taken into account when deciding how much higher the clay fill is to be placed, so that the surface will be at about the desired elevation after most of the settlements have taken place.

The fourth phase is the period after completion of the pavement. Since the pavement is, of course, so built that the subsidence due to the preceding three phases is practically eliminated, only the settlements of the fourth phase are of consequence so far as the pavement is concerned. For the average conditions illustrated in Fig. 11, the settlement after completion of the pavement to the present date is about 0.5 ft.

Along the center lines of runways A and B accurate settlement observations were made. The progress of these settlements, since completion of the pavement, is illustrated in Figs. 12 and 13. In these figures the square points are the observed maximum settlements, the triangular points the observed minimum settlements, and the round points are observed settlements for a station for which the settlements

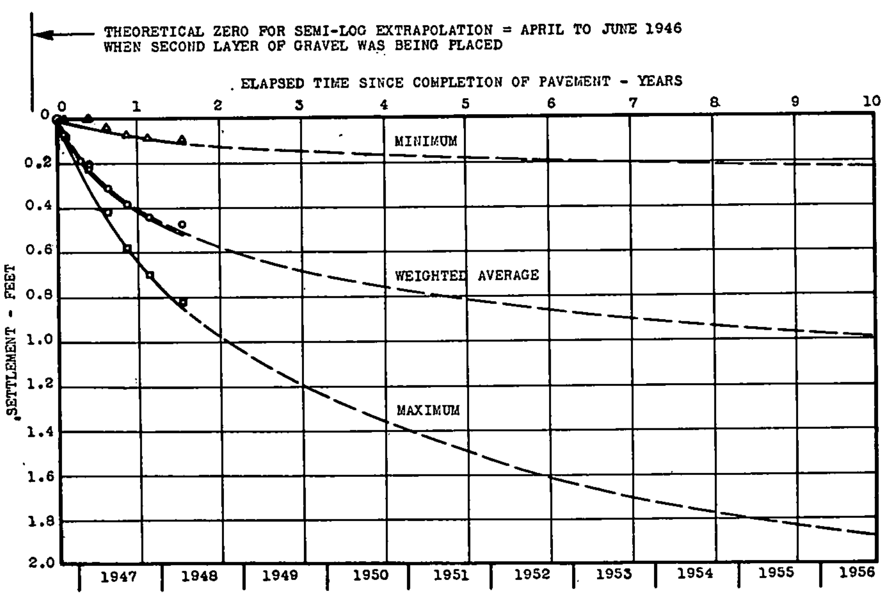


Fig. 12.—Range of Observed and Predicted Settlements for Runway "A" (Extrapolation from Straight-Line Semi-Log Plots).

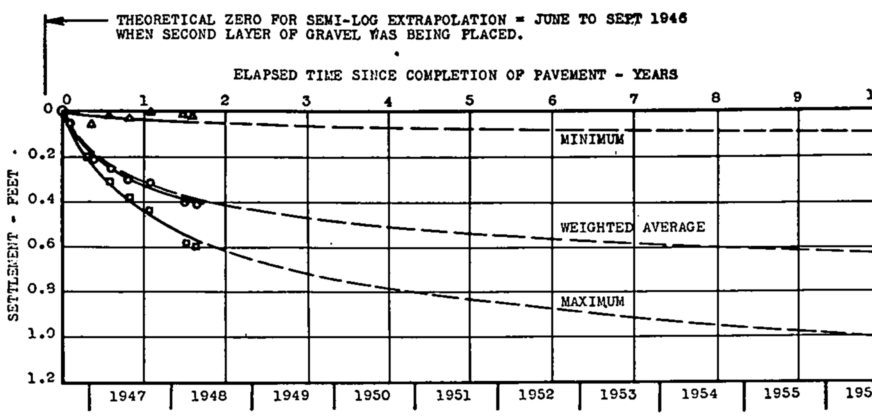


FIG. 13.—RANGE OF OBSERVED AND PREDICTED SETTLEMENTS FOR RUNWAY "B" (EXTRAPOLATION FROM OBSERVED STRAIGHT-LINE SEMI-LOG PLOTS.)

are about equal to the weighted average for the entire runway. The dashed portions of this time-settlement curves are extrapolated with the assistance of semi-log plots.

In Fig. 12 the weighted average settlement on runway A has reached a value of about 0.6 ft. The minimum is only about 0.1 ft and developed in an area which was subjected to a temporary excess fill of sand which was found in an adjacent borrow area and stockpiled here for later use as base material. The maximum settlement occurred along a stretch where the following four unfavorable factors combined: (1) A thickness of about 10 ft of silt underlying the clay fill, found in borings made after completion of this fill; (2) the fill itself consisted of exceptionally soft clay; (3) the clay was filled to an elevation which was one to three feet too low, which deficiency was made up by a greater thickness of gravel; (4) the interval between completion of the placement of gravel and of the pavement was exceptionally short, namely only three weeks as compared with three months which had been recommended as a minimum. Extrapolating the observed curves in Fig. 12 with the assistance of a semi-log plot, one finds that 10 years after completion of the pavement the weighted average settlement of this runway will be 0.9 ft, the maximum 1.9 ft, and the minimum slightly more than 0.2 ft.

In Fig. 13 one can see that for runway B, at present, i.e. about two years after completion of the pavement, the weighted average settlement of the pavement is about 0.4 ft; the maximum about 0.6 ft; and the minimum is almost zero, which corresponds to a section of the runway at Apple Island where there was only a few feet of clay fill on an incompressible foundation of hardpan. The extrapolated values for 10 years after completion of the pavement are a weighted average of about 0.6 ft, a maximum of about 1.0 ft, and a minimum of 0.1 ft.

There was also a section along runway B where sand was stockpiled to a maximum height of 7 ft above the clay grade; that means a load greater than the load of gravel and pavement which was subsequently placed. Although this excess load was active for only a few months, it precompressed the clay fill to such an extent that the settlements of the pavement in that area were very small. These observations are a convincing proof that preconsolidation by overloading would have been on this project a practical and effective method of reducing settlements to a nominal amount, provided sufficient time would have been available. An important application of such preconsolidation suggests itself in connection with future construction where new pavements will adjoin existing pavements; for example, where a second set of runways will intersect existing runways and taxiways. At such crossings, objectionable differential settlements will develop unless special precautions are taken. I believe that the most satisfactory method would be the judicious use of a temporary surcharge with variable thickness to form an appropriate transition.

A more detailed study of the runway settlements is contained in my report to the Department of Public Works on "Investigation of Settlements at Logan Airport", dated July 1948. Therein it was concluded that a few limited sections of the runways may develop objectionable differential settlements during the first ten years and that filling of the depressed areas may be required. After such corrective measures have once been carried out, it is believed that no further corrections will become necessary.

Because differential settlements of the order of magnitude shown in Figs. 12 and 13 were anticipated, it had been decided at an early stage that a concrete pavement would not be suitable for the runways and taxiways on the new fill. The question was later raised whether by means of appropriate measures construction of concrete pavements on the new fill would have been possible. On the basis of the available settlement observations and studies one can now conclude that by preconsolidating the fill for a period of about two years under a ballast equal in weight to that of the base and pavement one could obtain reasonable assurance that the magnitude of the subsequent differential settlements would not be objectionable for a rigid pavement. The same result would be obtained by preloading with a heavier ballasf for a shorter period.[1]

Design of Base and Pavement Thickness for Runways

Observations on the completed clay fill soon showed that the bearing capacity of the upper few feet of the clay did not develop as anticipated. For example, in constructing a temporary runway it was found that after placing two feet of gravel fill, trucks and tractors moving on the surface of the gravel would still cause serious weaving of the surface. Tests on the clay below the dried crust showed a CBR value of less than one per cent. It was also found that the drying effect could not be counted on to increase the bearing capacity. Even after a year, or more, the dried crust was only a few inches thick and the consistency of the underlying clay was practically unchanged.

It became clear that in designing the thickness of base and pavement it would be necessary to count on the increase in strength of the clay due to consolidation under the weight of the base and pavement. I should emphasize that the word "consolidation" in this case refers principally to the firming-up of the soft matrix between the clay balls. To study this increase in strength and to develop data on which to base the design of base and pavement thickness, a full-scale test section was built consisting of six panels with combined thickness of base and pavement ranging from 48 to 66 in. On the surface of these panels, load tests were made at frequent intervals, on 30 in. diameter plates, to determine the increase in strength of the subgrade with time. Typical results are shown in Fig. 14a and b.

In Fig. 14a the base consisted of 30 in. of sand, 12 in. of crusher run stone and 6 in. of bituminous pavement. The first set of load tests was made without pavement and all others after the pavement was completed. As can be seen in this figure, the settlement under the load decreased rapidly with time. For example, under 20 lb per sq. in. stress the deflection was 0.2 in. soon after the base was placed. After 180 days a stress of 70 lb per sq in. had to be applied to cause

[1]These recommendations do not apply to the terminal area which is located on the old airport fill. For the construction of the concrete pavements in this area I had recommended the use of preloading for a period of three months with a gravel ballast weighing about as much as the pavement.

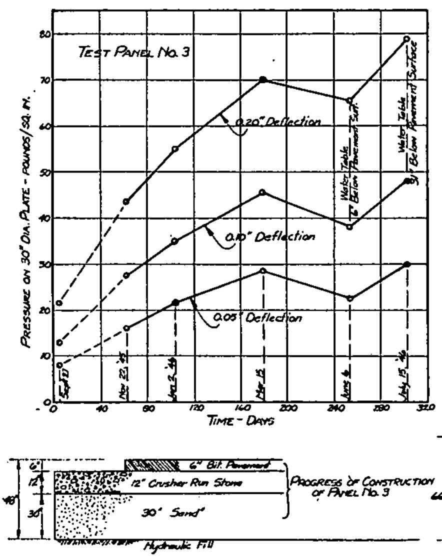


FIG. 14A. — INCREASE OF DEFLECTION RESISTANCE WITH TIME FOR TEST PANEL No. 3.

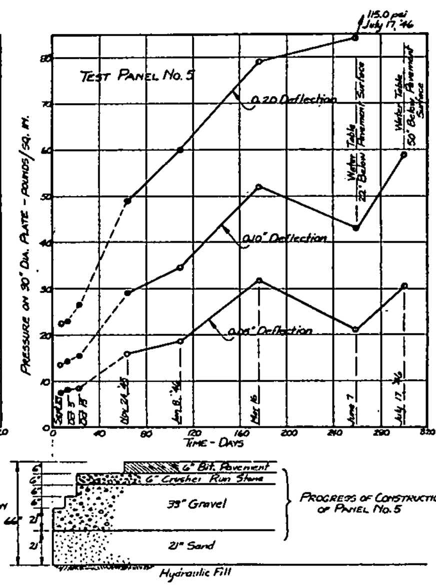


FIG. 14B. — INCREASE OF DEFLECTION RESISTANCE WITH TIME FOR TEST PANEL No. 5.

the same deflection. The temporary drop in resistance between 180 and 250 elapsed days was due to the fact that in this period the water table in the test section rose to the bottom of the pavement as a result of improper drainage provisions in the test area. The resulting buoyancy caused the effective load in the clay to be substantially reduced, which explains the drop in the bearing capacity. Subsequently, the water table was lowered and the bearing capacity started to rise again.

In Fig. 14b the increase in bearing capacity with time is shown for another test panel with a combined thickness of base and pavement of 66 in.

From this full-scale investigation it was possible to extrapolate the relationship shown in Fig. 15 between elapsed time and load for various combined thicknesses of pavement and base. This diagram is based on the conservative assumption that 0.2 in. constitutes the maximum allowable deflection of the pavement for load tests on 30 in. diameter areas. According to this diagram, a 5 ft combined thickness of pavement and base would develop for a single wheel load a bearing capacity of 65,000 lb for one year, and 125,000 lb for five years after completion of the pavement. Based on this information, the engi-

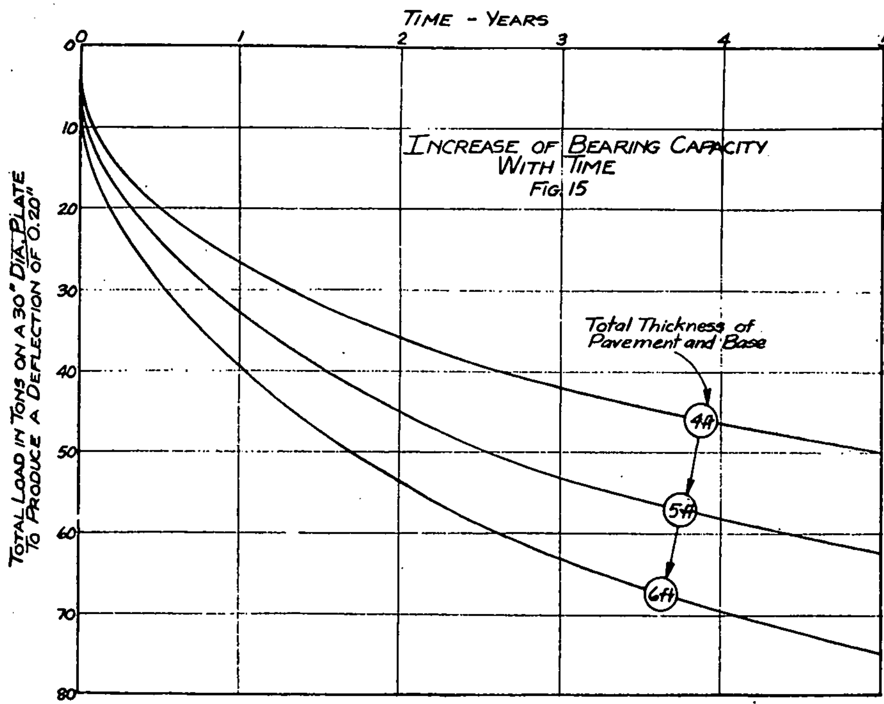


FIG. 15.

neers of the Public Works Department decided to adopt a design thickness for pavement and base of 5 ft for the runways and 5½ ft for the ends of the runways and the taxiways.

A series of similar loading test which were later conducted on the completed pavements for check purposes revealed that the resistance shown by Fig. 15 was generally reached and in most instances exceeded.

To my knowledge this is the first time that the design of runways was based on the increase in bearing capacity of the underlying clay subgrade produced by consolidation under the weight of the base and pavement. Considering that before placement of the gravel base the clay fill had a California Bearing Ratio of less than unity, it would not have been possible to design and build the base and pavement utilizing any of the accepted design methods.

Keeping in mind the increase in strength of the clay which has developed since completion of the pavements, I believe that I am correct in stating that these runways have a higher bearing capacity for single wheel loads than any other runways now in use for commercial traffic along the entire Eastern Seaboard.

This project has proven that the increase in strength of a very soft clay subgrade due to consolidation under the weight of a substantial thickness of base and pavement should be taken into account to enable more economical designs of airfield pavements for heavy wheel loads. In fact, in some cases, such as on this project, one could not design the runways by any of the accepted methods and one is forced to base the design on the anticipated consolidation of the clay under the weight of the base and pavement. The design procedure which was used on this project is probably of greater significance for hydraulic clay fills than for natural deposits of soft clay.

ACKNOWLEDGMENT

The Logan Airport was designed and built under the direction of the Waterways Division of the Massachusetts Department of Public Works, with General Richard K. Hale as Director and Mr. Everett N. Hutchins as Engineer. The design and construction of the base, pavements and drainage of the runways and taxiways was under the direction of Mr. George H. Delano, Project Engineer. Numerous consultants assisted on various phases of the project.

Mr. W. L. Shannon was associated with the writer as soil mechanics consultant in connection with the bearing tests and other investigations for the design of the runways.

The following former members of the staff of the Harvard Soil Mechanics Laboratory assisted in the development of the piezometers: Mr. J. A. Delaney, Dr. M. J. Hvorslev, Mr. R. J. Marsal and Dr. A. A. Warlam. Mr. R. W. Brandley assisted in the preparation of the Appendix.

Mr. R. H. Lamb, of the Massachusetts Department of Public Works, supervised the field installations for pore pressure and settlement observations and rendered valuable service in plotting and analyzing these observations. The success of these measurements is due largely to his untiring efforts.

Mr. L. Tuttle, while a graduate student, analyzed the results of the field bearing tests.

Mr. J. P. Gould, graduate student at the Harvard Engineering School, performed a detailed analytical study of all factors which influenced the runway settlements and prepared the diagrams used in this paper.

APPENDIX

A Non-Metallic Piezometer for Measuring Pore Pressures in Clay

1. *Design and Construction of Porous Point*

The porous point consists of a 2 foot length of fine grade Norton Porous Tube, O.D.-1.5", I.D.-1.0". One end of this tube is plugged with a #5 rubber stopper, and into the other end is installed an appropriate length of ½" O.D. Saran tubing, held in place by a soft rubber bushing. A cross section of this point is shown in Fig. 16. The length of Saran tubing is determined by the depth of the point below the elevation where the observer will stand.

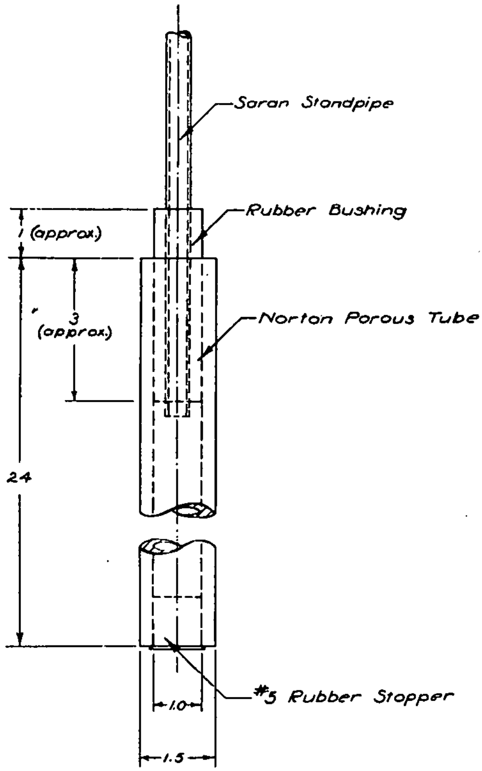


Fig. 16.—Porous Point Assembled.
(Dimensions in inches)

To install the length of Saran tubing, proceed as follows:

(a) A 4 inch length of ⅜" I.D. by $\frac{5}{16}$" wall Neoprene or rubber bushing is cut from a length of tubing of this size.

(b) One end of the ½" O.D. Saran standpipe is bevelled on the outside with a knife and lubricated with water.

(c) This lubricated end of the Saran tube is inserted about 1" into the Neoprene bushing.

(d) The Neoprene bushing is next inserted into one end of the porous tube as far as possible (approximately 3 inches).

(e) With a twisting motion the Saran tubing is forced 3" further into the Neoprene bushing. This distance can be determined by placing a mark 4" from the end of the Saran tube prior to assembly.

If properly installed, it is impossible for one man to pull this joint apart with his hands.

It requires considerable effort to force the Saran tube into the Neoprene bushing and it is advisable to make a strap wrench for gripping the Saran tube. An improvised strap wrench can be made with strong twine and a piece of wood as shown in Fig. 17. If the ends of the twine are held tightly and the wooden

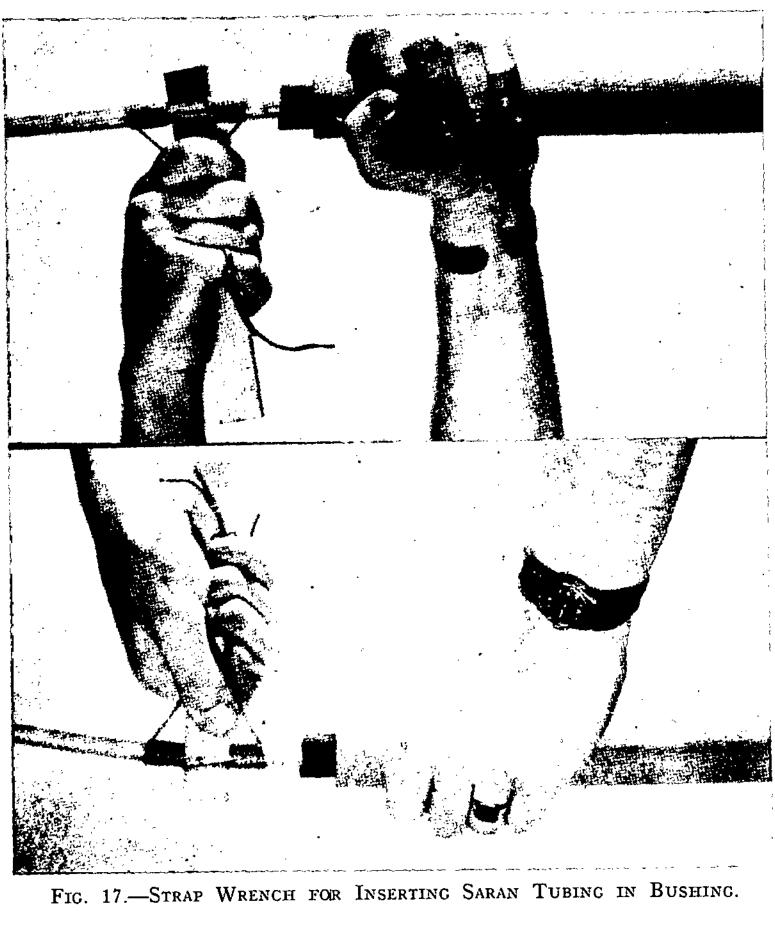


FIG. 17.—STRAP WRENCH FOR INSERTING SARAN TUBING IN BUSHING.

handle is turned in the right direction, the twine tightens on the Saran tubing and acts as a wrench.

2. *Design and Construction of Tamping Hammer*

The hammer is made of a 3 foot length of tubular steel, 1⅝″ O.D. and ⅝″ I.D. to fit the 2″ casing and the ½″ O.D. Saran tubing. The hammer is provided with a flat tamping face $\frac{3}{16}$″ larger in diameter than the hammer body. The hammer is rigged with a ⅛″ 7×19 Galvanized Preformed Airplane Cable through a snatch block over the casing. The cable is operated by a hand reel. The detailed design of the hammer is given in Fig. 18.

The purpose of the hammer is twofold: (a) To tamp the bentonite seal

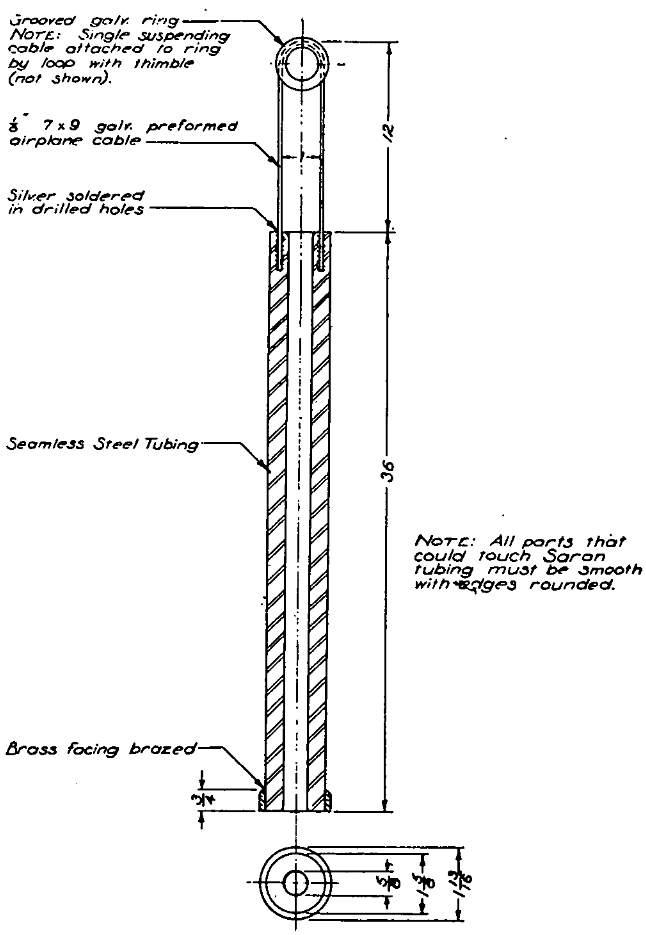


FIG. 18.—TAMPING HAMMER (Dimensions in inches).

into place. (b) To center the Saran standpipe while the seal is being tamped into place.

It is important that all parts of the hammer that may touch the Saran tubing be smooth and the edges rounded.

3. *Installation of Porous Point*

(a) A cased hole is advanced to the elevation planned for the bottom of the permeable space. Two-inch diameter pipe is recommended for the casing. The first section of casing should be at least 10 feet long, and it should *not* be provided with a coupling (or drive shoe) at its lower end. No washing should be done in advance of the bottom of the casing as the last 10 feet are driven. This will assure a tight contact between the bottom 10 feet of casing and the surrounding soil because the lack of a drive shoe or coupling will not leave any space outside the lower casing when it is pulled back.

(b) The inside of the casing is washed clean to the bottom, then the wash water is entirely replaced by clear water. This is done by reversing the flow of the jetting pump and using the jet pipe, with its lower end a few inches above the bottom, as the intake. The casing is kept filled by pouring the clear water in until all the cloudy water is pumped out.

(c) The casing is then pulled up 2 feet. This should be done preferably by jacking, and just prior to backfilling with sand. A slip weight may be used for raising the casing if necessary. After the casing is raised, saturated sand is poured in, to fill the bottom 2 feet of the open hole, that is to the bottom elevation of the porous point as shown in Fig. 19. Any washed and screened sand between #20 and #35 mesh may be used for this purpose. Care should be taken that the sand is not poured in faster than the casing is raised to prevent the sand from arching within the casing. This would render it impossible to get the porous point down below the bottom of the casing. The sand must be thoroughly saturated before pouring it into the hole, and the casing kept filled with water. The volume of sand needed should be computed and closely controlled. It is recommended that the hammer be used to check the height of the sand in the hole.

(d) The standpipe of the assembled piezometer is attached to a small tank. The point is immersed a few feet below the top of the casing and a vacuum is applied to the tank. The piezometer is filled in this way and a small reservoir of water is obtained in the tank. While lowering the point to the bottom a small excess head is maintained in the point to assure that a small amount of water will flow out of the point while it is being lowered into place.

The elevation of the top of each point must be accurately determined during its installation by lowering the hammer and measuring the length of cable when the hammer is just resting on top of the point. For this purpose it is convenient to have the cable marked off in 5 foot intervals on strips of tape, starting with zero feet at the working face of the hammer.

(e) With the point resting on the sand in the bottom of the hole, the casing is pulled up an additional 2 feet, corresponding to the desired elevation of the top of the point. Then, saturated sand is poured in to fill the space around the point. The volume of sand needed for this operation can easily be computed and controlled.

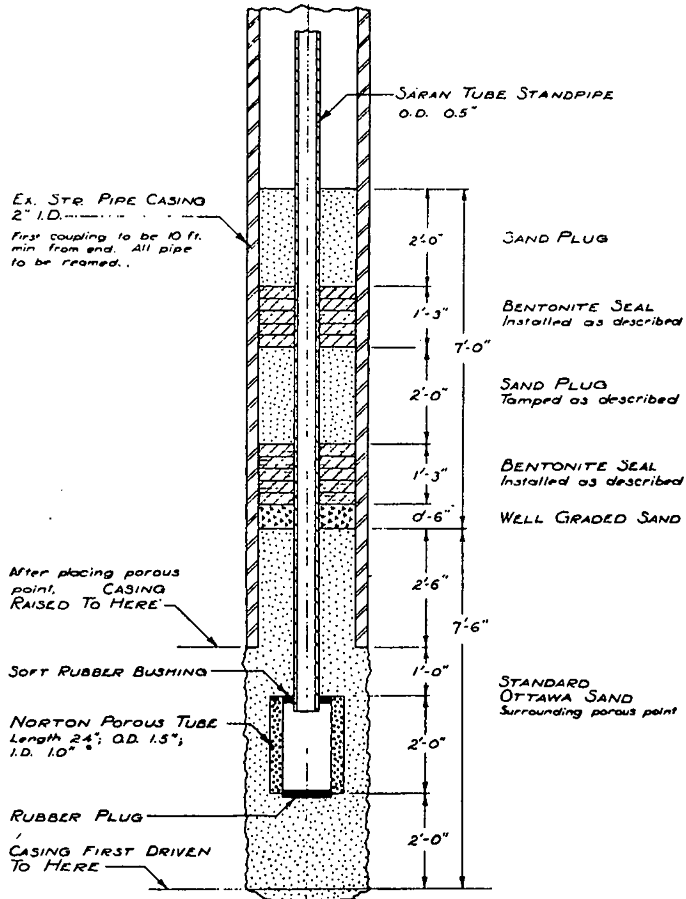


FIG. 19.—DIAGRAM, IN DISTORTED SCALE, SHOWING DETAILS OF PIEZOMETER INSTALLATION.

(f) While pulling the casing up to its final position, that is one foot above the top of the porous point, the open hole is backfilled with more saturated sand.

(g) Enough sand is next poured into the casing to fill approximately the bottom 3 feet of the casing. It is then tamped by means of 10 blows of the hammer dropped 6 inches. The purpose of this sand plug is to minimize the effect of swelling pressures of the overlying bentonite.

(h) Then, bentonite which has been prepared in a putty like state (slightly above the plastic limit but below the sticky limit) and formed into balls about 3/8 inches in diameter, is placed. The bentonite balls are dropped through the water to the bottom of the casing. Five 3" layers, each one well tamped, provides an effective seal.

To determine how many bentonite balls should be dropped into the casing to make one 3″ layer, the water level in the casing is lowered until it is 3″ from the top, and then enough balls are dropped down to raise the water level again to the top. Then a ¾″ thick layer of approximately ⅜″ diameter rounded pebbles is dropped on top of the bentonite layer, to prevent the hammer from sticking. The hammer is next lowered onto the pebbles and 20 blows are applied by raising the hammer about 6 inches and allowing it to drop freely. This process is repeated until the 5 layers of bentonite are in place.

(i) A plug of graded sand, about 2 feet in length, is then added and tamped into place.

(j) As an additional precaution against leaking, another seal of five 3 inch layers of tamped bentonite should be placed on top of the sand plug. This second bentonite seal is placed as described above. See Fig. 19.

(k) The second bentonite seal should be capped with about 3 feet of sand, and the remainder of the hole may be left open if desired.

4. *Equipment for Measurements*

The water level in the piezometer standpipe, when below elevation permitting direct readings, is found by a simple electrical sounding device. A drawing of a contact tip and a sketch showing the circuit, is shown in Fig. 20. A pair of #22 B and S Neoprene insulated 7/#32 stranded wires are taped together and marked off for measuring.

The lower end of the sounding wire is weighted by wrapping with sheet lead (about 1/32 inch thick) in short sections. The sections should be about 1 inch long and spaced 1 inch apart. Each weight should not be more than ¼ inch in diameter and the sharp corners on it should be filed smooth. Care must be exercised in putting the weights on the wire that the insulation is not damaged. Fifteen such weights will keep the wire taut.

The contact point is made by baring the ends for about ¼ inch and leaving a little less than ¼ inch space between them. Sealing wax applied above the bared ends keeps the contact points in shape.

This sounding wire is connected to a suitable ohmmeter and lowered to the water surface. When the bare ends touch the water, the hand on the ohmmeter kicks out, indicating that the circuit is closed. A Weston model #564, type 3-C, Volt-Ohmmeter has sufficient range to give distinct readings.

The operator of this sounding device should note that as the points approach the water surface (say within 2 feet of it)) often there is a slight movement of the hand on the ohmmeter. This is probably due to a thin film of water on the inside of the standpipe. However, if the points are gradually lowered further, there is a distinct jump of the hand as the points make contact with the water surface.

The entire tip (sealing wax and bared wires) should be covered with a film of grease at all times. If this is not done, the water will not always break free on withdrawing the contact point, and it may not be possible to check a reading by making and breaking the water contact several times in succession.

This device has been developed empirically and is known to give satisfactory

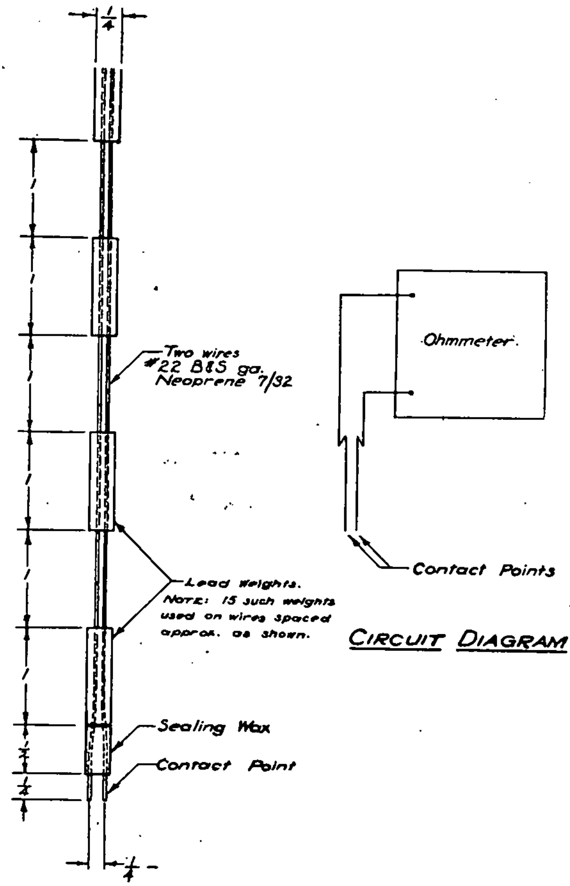


FIG. 20.—CONTACT POINT ASSEMBLY.
(Dimensions in inches)

results. Deviations in construction may cause difficulties and should be tried before they are used.

In all cases where the elevation of the water in the Saran standpipe can be obtained by a direct reading this should be done.

If the pore pressure becomes so great that the water is forced out of the top of the standpipe, the excess pressure can be measured by means of a Bourdon Gage or a Mercury Manometer.

5. *List of Firms from Which Parts Were Procured*

Norton Porous Tube—made of an abrasive refractory material; Norton Co., Worcester, Mass.

Rubber Stoppers—Central Scientific Co., 79 Amherst St., Cambridge, Mass.
Saran Tubing—Brown Wales, 493-C St., Boston, Mass.
Soft Rubber Tubing—Central Scientific Co., 79 Amherst St., Cambridge, Mass.
Seamless Steel Tubing—Austin Hastings Co., Inc., 226 Binney, Cambridge, Mass.
Galvanized Preformed Airplane Cable—Roebling & Sons, 51 Sleeper St., Boston, Mass.
#22 B & S Neoprene Insulated 7/#32 Stranded Wire—Boston Insulator Wire Co., 65 Bay St., Dorchester, Mass.
Weston Model #564 Type 3-C Volt-Ohmmeter—Weston Electric Instrument Corp., Newark, New Jersey
Bentonite—Commercial Name Volclay KWK #33—American Colloid Co., 363 West Superior St., Chicago, Ill.
Neoprene Tubing—Greene Rubber Co., Broadway & 6th, Cambridge, Mass.
Pliobond—United States Plywood Corp., 57 Mystic Ave., Medford, Mass.

GROUND VIBRATION DUE TO BLASTING AND ITS EFFECT UPON STRUCTURES

By F. J. Crandell, Member*

Introduction

Building vibrations and the determination as to whether these vibrations cause damage to the structures has been for some time a controversial subject.

As far as earthquake vibrations are concerned, there still is a difference of engineering opinion regarding the design of structure to withstand them.

Certain phases of the construction industry, by their very nature, produce earth vibrations that are imposed upon structures and buildings. One of the most common operations of this nature is the use of explosives which assist in the excavation for foundations, tunnels, sewers, etc.

When the property owners feel the vibration and hear the detonation of the explosives, they immediately question, "Will the ground movement injure my building?" By this same token, when a contractor sets up his procedure for the use of explosives, he is likely to think, "How many pounds can I use without injury to structures? Will I have to reduce the number of pounds because of the nearness of these structures?"

This research was instigated in order to establish information that will assist the contractor in determining a safe charge of explosives to use in his excavation procedure and to insure no damage to adjacent structures.

Other organizations have investigated building vibrations and produced very informative data. However, most all the building vibration determinations have been made in the structure itself. Forced vibrations have been imposed upon the buildings and structures to determine their natural frequency. Measurements have been made upon the slabs and walls of structures to determine how much vibration they could withstand, and certain threshold determinations have been established.

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When a contractor is confronted with 500 to 1,000 buildings adjacent to his operation, a situation that occurs in a long tunnel job, it is not practical to measure the vibration within each structure itself.

If the intensity of ground vibrations could be used as the indicator of structural damage, it would not be necessary to measure the vibration in each individual structure.

Therefore it became our objective to try and determine the amount of ground vibration that would be needed before injuring the structure, and a method of predetermining the amount of charge of dynamite that could be used safely without damaging or rupturing any part of the structure.

With this in mind we investigated the ground vibrations produced by the use of explosives.

Instrument

In 1936, it was decided to design an instrument that would assist in measuring vibrations that exist in the ground during construction operations.

Because we were interested in measuring the magnitude and direction of forces transmitted through the ground, our present recording accelerometer (or accelerogaph) was designed to measure the accelerations in three planes.

Figure 1 shows the three reeds in the vertical, transverse, and longitudinal planes which are activated when vibration occurs. To these three reeds are attached mirrors which pick up a light source in the box and transmit the light beam on to a 35 mm. motion picture film.

The natural frequencies of the vibrating reeds are between 90 and 100 cycles per second. As a rule, the elastic frequencies in the earth and rock have been below the natural frequency of these reeds.

The 35 mm. film travels at a speed of 3¾ inches per second.

It will be noted in Figure 1 that there are dampening pots containing stiff grease attached to each reed to give a fairly sharp dampening after the external vibration ceases.

With this accelerograph we have measured vibrations produced by dynamite in the course of blasting rock and earth, tremors produced by the driving of piles, and vibrations produced by machine rotation in the flooring system of manufacturing buildings.

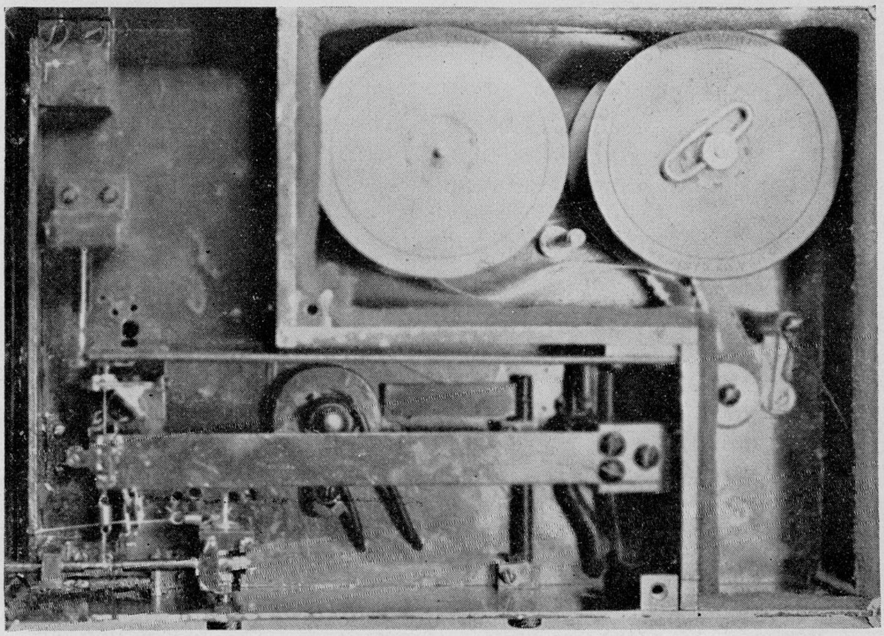


Fig. 1.—Accelerograph.

This study is primarily concerned with vibrations due to blasting that are relatively close to the source of energy that produces them. Most of our measurements were taken at a distance of from 20 to 200 feet from the energy source.

Conclusions

1. This investigation indicates that Energy Ratio (E. R.) can be used as a measure of damaging force that may be imposed upon structures due to ground vibrations.
2. (a) The E. R. is proportional to the square of the weight of explosive detonated at any instant.

   (b) The E. R. decreases as the distance from the center of the explosion increases.
3. Since the E. R. is proportional to the square of the weight of explosive, at any instant, the use of delays decreases the E. R.
4. The equation $\left(\frac{50}{D}\right)^2 C^2 K = E. R.$ is a fair approximation of energy ratio to expect from a specified number of pounds of dynamite.
5. Generally, if the charge of dynamite to be used is proportioned so that the number of pounds per delay develops an E. R. in the equation of less than 3

no damage is likely to occur in structures composed of standard engineering building materials that have not been prestressed.

6. By preliminary vibration tests of the ground strata in question, it is possible to determine the ground constants and thus determine the amount of explosives that can safely and practically be used without causing damage to adjacent structures.

Objectives

We wish to determine:

1. The manner in which the transmitted energy varies with the amount of explosive detonated at any instant.
2. The manner in which the transmitted energy varies with distance from the source.
3. The total amount of energy imposed upon a structure by ground vibration that will damage the structure.
4. Whether or not it is possible to predetermine the safe amount of explosive that can be used in a specific location.

Procedure

Our basic assumption during these investigations was that the vibration in the ground was simple harmonic in nature.

Starting with the equation that $K.E. = \frac{W V^2}{2g}$ it is possible to show that this equation can be reduced to give a value in terms of acceleration, i.e.—

$$K. E. = \left(\frac{W}{2g4\pi^2}\right)\left(\frac{a^2}{n^2}\right)$$

The first fraction of the equation is a constant at any location depending on the mass set into vibration. Therefore, if the mass is constant, the K. E. may be considered to be proportional to $\frac{a^2}{n^2}$ hereafter termed the E. R.

Throughout this analysis most of the curves pertaining to blasting are comparisons of E. R. with distance or pounds of dynamite. This was necessary because we were not able to evaluate the mass of ground that was in motion.

In all cases, the instrument was placed upon the surface of the earth, rock, or the soil overburden.

Measurements were taken when:

(a) The explosive was detonated in the rock with the instrument located on the rock.
(b) The explosive was detonated in the rock with the instrument located on the surface of the soil overburden.
(c) The explosive was detonated in the soil below the surface with the instrument located at the surface.
(d) The explosive was detonated in the rock and/or soil with the instrument located in buildings.

Among the variables we had to deal with were: distance from the energy source; weight of explosive being used, either in the total shot or in the individual delays; variations in the density and consistency of the soil; and variation in the rock strata due to faults and dips, etc.

We endeavored to overcome these variables (1) by locating the instrument at a constant location and varying the number of pounds of dynamite or (2) by keeping the number of pounds of dynamite as constant as possible and locating the instrument at different distances from the source.

Findings

It is evident from the equation used that the E. R. is always a ratio involving acceleration and frequency. Therefore, in all cases we were interested in both these factors. The frequency of vibrations in the ground is a potent factor in the amount of energy that is transmitted to structures. This was found to be evident when we made a comparison of the vibration in rock with the vibration in earth overburden.

Frequencies

The rock, being a uniform elastic medium, transmitted the vibrations readily but always at a frequency of vibration that was relatively high. We have measured frequencies of vibration in rock as low as 40 cycles per second, and as high as 80 to 90 cycles per second. Because the E. R. is inversely proportional to the square of the frequency, the resulting energy imposed upon the structure was always small, even though in many cases the instrument was so close to the blast that accelerations greater than gravity were measured. The frequencies in rock were always so high that the resulting E. R. transferred to the structures was low.

The frequencies of vibration through the earth medium were always found to be much lower than those through rock. For the same acceleration, the resulting E. R. imposed upon the structures built upon earth was many times greater than that imposed upon the structures founded on rock. Our experimental data have indicated that frequencies in the earth range from a low of 7 to 10 cycles per second to a high of 40 cycles per second.

It is evident that the safety of structures located in or on the overburden is the critical problem.

The frequencies of vibration in the three planes were not always the same, varying both in magnitude and in phase. Because of these variations, it did not seem advisable to try to plot the resultant E. R.'s but to record the E. R. separately in all three planes.

Earthquake

Examples of the destructive force caused by vibrations in the ground are found in the seismological data established by the Coast and Geodetic Survey. One of the most destructive earthquakes recorded in the past few years in the United States was the 1933 earthquake in Long Beach. The records as established by the Coast and Geodetic Survey show an average acceleration of approximately 3 feet per second per second, and a frequency of about one cycle per second measured in the basement of buildings. This would establish an E. R. equal to $\frac{9}{1} = 9$.

The force associated with this E. R. demolished wall bearing brick buildings, destroyed chimneys and porches on frame structures, produced damage to and in some cases collapse of the water tanks, and caused some but not excessive breakage in underground utilities.

Although the effects of a large E. R., as indicated above, are known we did not know at what lower E. R. the damage would be likely to start.

Experience to date has shown that if the E. R. is kept below 3 no damage is done to buildings of sound construction and material.

Dampening Due to Distance

Measurements were taken of the ground vibrations during the excavation of rock for a water tunnel shaft. Because this was a practical operation and not an experimental one, it was impossible to control accurately the number of pounds of dynamite used in

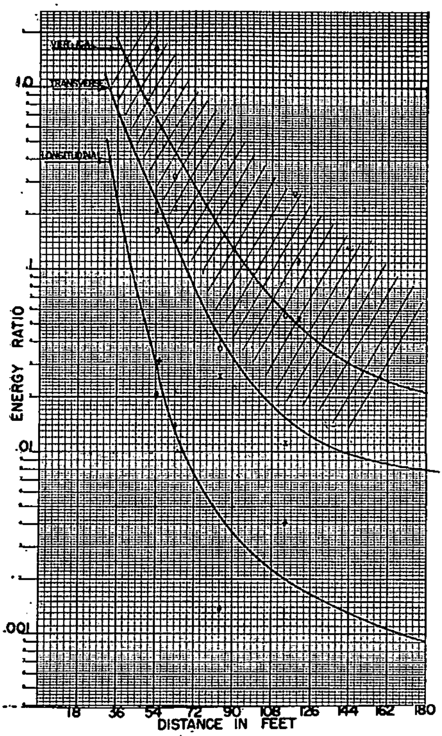


FIG. 2.—DAMPENING DUE TO DISTANCE. (Shaft Sinking)

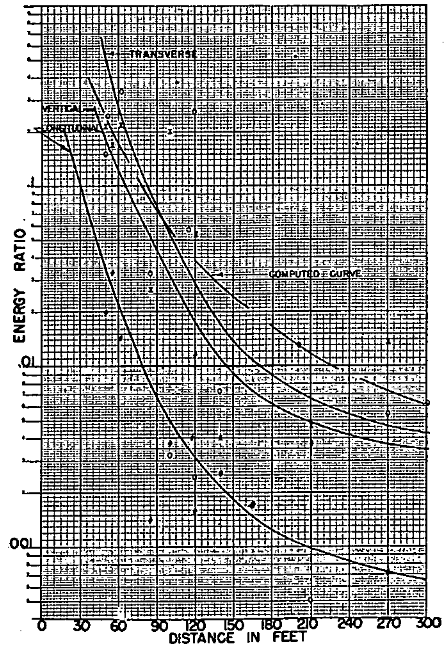


FIG. 3.—DAMPENING DUE TO DISTANCE. (Sewer Tunnel)

each delay or in each shot. Since the variation was not great, the instrument was located at varying distances from the source, and an endeavor was made to coordinate the E. R. at each distance with the number of pounds of dynamite used per delay. Figure 2 is a plot of the vibration measured in the vertical, longitudinal, and transverse planes as the instrument position varied from approximately 50 feet to 125 feet from the source of vibration.

The material being excavated was a very hard rock and had an overburden averaging approximately 20 feet of clay and silt. The instrument in all tests was located on the surface of the overburden. Figure 2 shows the dampening effect at various distances from the source of the vibration.

Because of the tremendous variation in the location of the points on the graph, the curves at best can be taken only as averages. Most of this scatter was caused by the fact that it was impossible to have exactly the same number of pounds of dynamite in each individual delay.

Further tests taken on the same shaft sinking job showed that as the depth increased the dampening effect continued with distance. The distance increased up to approximately 270 feet from the source of vibration. After approximately 200 feet from the source, the dampening with distance tapered off and was not so rapid.

As in Figure 2, the curves on Figure 3 can be used as general indications of the dampening that occurred with distance.

As will be noted in this graph, an attempt was made to establish a relation between the number of pounds of dynamite in the vibration source and the E. R. At approximately 50 feet from the source of vibration the equation reads as follows: $C^2\,(0.001) = \text{E. R.}$ We established empirically the following equation for distance:

$$\left(\frac{50}{\text{Distance}}\right)^2 C^2\,(0.001) = \text{E. R.}$$

The curves in Figure 3 can be used as a general indication of the dampening that occurs with distance but should be considered only general because of the variation of the points on the curve. The variation of these points were again primarily caused by the fact that we were unable to standardize the number of pounds of dynamite used per delay.

Delays

In all cases, the establishment and use of delays materially decreased the total vibration. Each individual delay was characteristically recorded on the film, and at no time was there an overlapping or continuous vibration between delays when the delays acted as they were designed. Whenever two or more delays went off at once, we obtained a cumulative effect of the vibration equal to the square of the number of pounds of dynamite in those delays.

The equations shown were established for instantaneous shot, and therefore the C^2 in the equation is the square of the number of pounds of dynamite used in each delay.

Dampening Due to Reduction in Charge of Explosive

Further tests were made in the excavation of tunnel work in which a very stiff clay had to be excavated. In order to operate economically, the contractor drilled and shot this clay similar to the operation performed in blasting rock.

In all the measurements taken at this time, the direct transmission of the vibration through the clay were measured.

In determining the effect in the clay caused by the variation in pounds of explosive, the instrument at all times was located at a distance of 50 feet horizontal from the source of vibration. The overburden above the excavation in all cases was approximately 20 feet, but the total depth of overburden was approximately 90 feet. Figure 4 is a plot of E. R. in relation to the pounds of explosives per delay.

Again it was found that vibrations from all shots were separated, that the vibrations were not cumulative. Therefore the number of pounds of explosive in each delay was proportional to the amount of vibration produced. A variation will be noted on the chart, but in general the curves indicate an increased amplitude of vibration with an increased number of pounds per delay.

In this case, since the energy source was in the clay and the measurements were taken in the same medium, the transverse vibration appeared at all times to be the maximum.

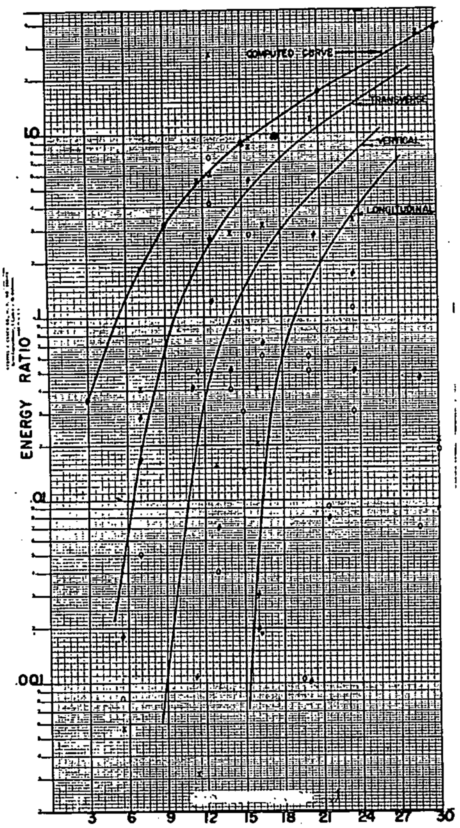
An investigation of the curve in the transverse plane shown in Figure 4 allows us to establish the equation that $C^2 (0.004) = E. R.$

Damage

During this operation, an endeavor was made to control the vibration by limiting the charges per delay. The maximum charge recommended in this case was 32 pounds of explosive per delay, which by calculation should produce an E. R. of three. Experience throughout this area, which is almost completely built up with residential homes, schools, and churches, indicated that when the structure had not been pre-stressed and the materials of construction were average, no damage occurred.

However, where there were structures that had been pre-stressed because of settlement or movement of any sort, and some showed actual breaks, an E. R. of three had sufficient force to increase the width of the cracks in some of the foundation walls.

As can be expected, the number of charges per shot could not always be controlled because at some times, either because of the inaccuracy of the delays or because of the propagation of one delay



Pounds of Dynamite
FIG. 4.—DAMPENING DUE TO REDUCTION IN CHARGE OF EXPLOSIVE.

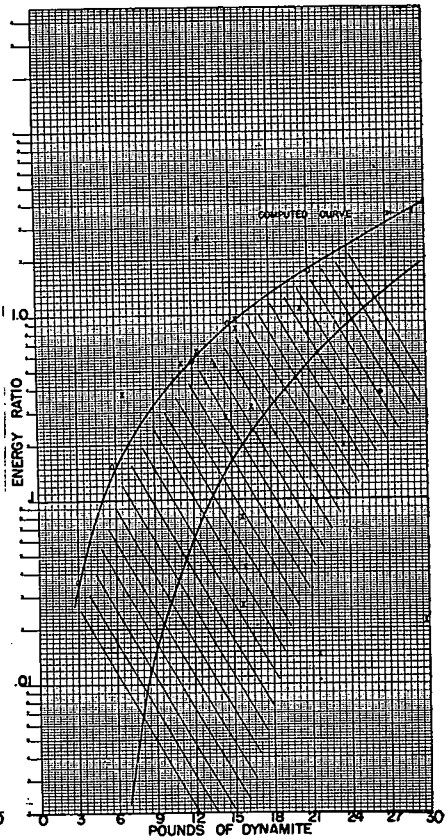


FIG. 5.—DAMPENING DUE TO REDUCTION IN CHARGE OF EXPLOSIVE.
(Transverse plane)

to another due to impact, there was more than one delay going off at once. When this occurred, the force of vibration increased with the square of the number of pounds of dynamite exploded at any given instant. In one case, a sufficient number of delays went off at once to explode approximately 70 pounds of dynamite. This produced an E. R. of approximately 10, and actually canned goods were knocked off the shelves of a grocery store within 75 to 100 feet of the shot.

Damage was done to a building that was constructed as a wall bearing structure and that had a terra cotta face built against the brick. This terra cotta face at certain locations was knocked off, but it was found upon examination that there were no ties between the terra cotta and the brickwork. This would not be considered good average building construction. However, this facing only came off when the E. R. was greater than three.

By separately plotting the longitudinal, transverse, and vertical planes, a better picture was obtained of the scattering of the points plotted, Figures 5, 6, and 7. The cross-hatched area of these charts can be called the Scatter Zone.

A study of the transverse plane data shows that with 16 to 17 pounds of explosive the measured E. R. varies from 0.02 to 0.9. However, this is still less than the threshold limit of 3.

Available tests using the equation:

$$\left(\frac{50}{D}\right)^2 C^2 (K) = E.R. = 3$$

allow a sufficient charge of explosive to effectively break the material being excavated. In fact, generally efficient and practicable charges of explosives have produced E. R.'s less than 3.

Frequencies in Separate Planes

The frequencies of vibration through the ground were found to be different in each individual plane.

Figure 8 is a magnified photograph of an instantaneous shot showing a comparison of the frequencies in each of the longitudinal, vertical, and transverse planes.

In order to give a better evaluation of the shape and frequency of the wave, the record on all three planes has been traced off separately. The first tracing is that of the record in the longitudinal plane or compression wave. This may also be called the push-pull wave.

This record is typical of the type of wave found in the longitudinal plane, and it will be noted that low frequencies predominate.

The second tracing is that of a typical record in the vertical plane. In this record, you will note that as the vibration comes into existence, there are large amplitudes, and the slope of the line indicates high frequencies. In all cases, high frequencies predominated in the vertical plane.

Although it can be seen that there are low frequencies and small amplitudes at the end of the wave, this is primarily caused by the dampening effect in the earth and does not indicate the maximum amount of energy in this plane.

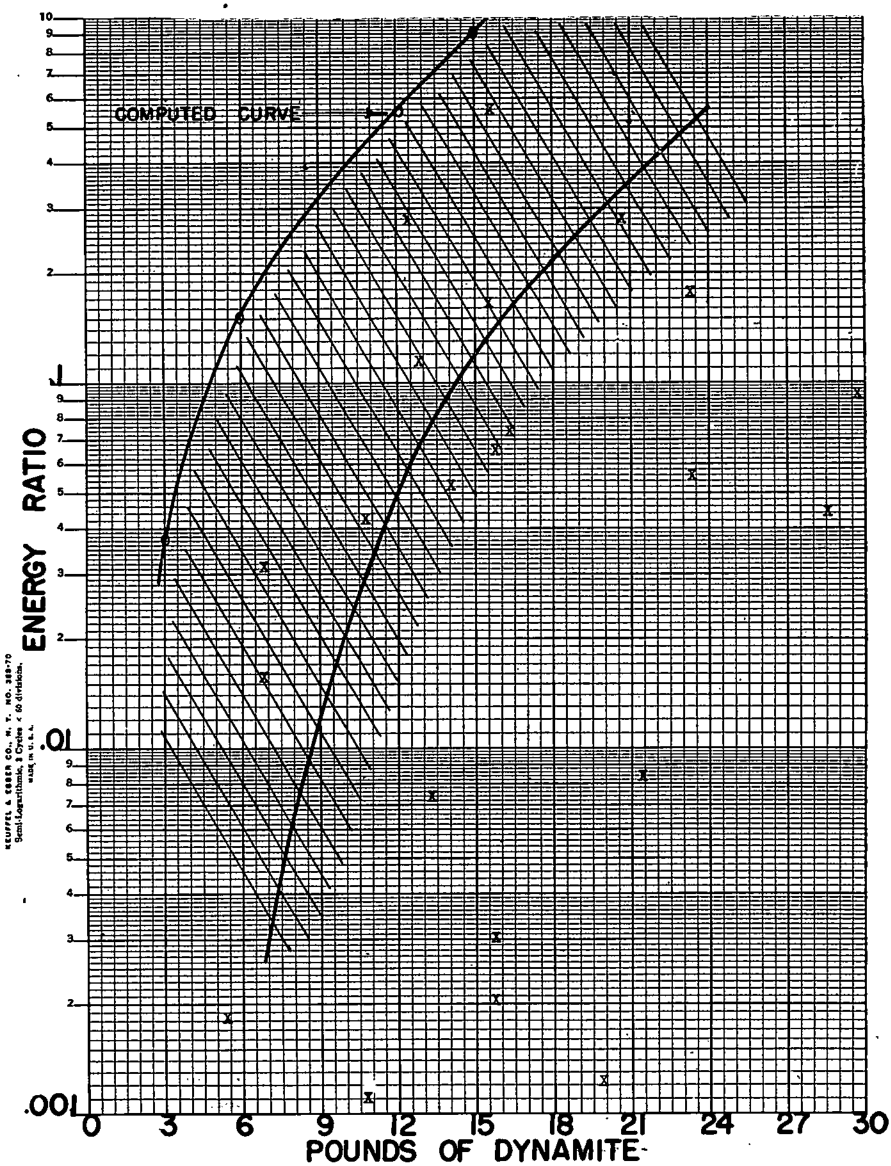


FIG. 6.—DAMPENING DUE TO REDUCTION IN CHARGE OF EXPLOSIVE.
(Longitudinal plane)

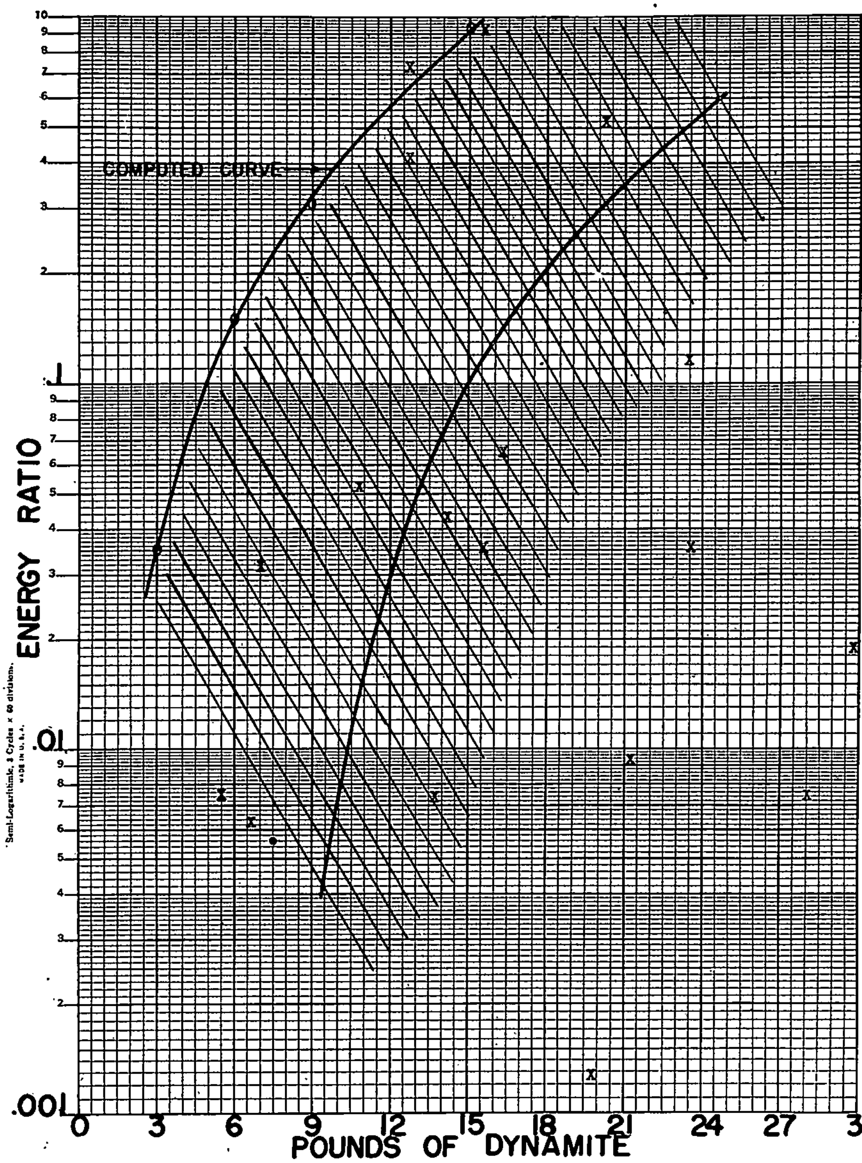


FIG. 7.—DAMPENING DUE TO REDUCTION IN CHARGE OF EXPLOSIVE.
(Vertical plane)

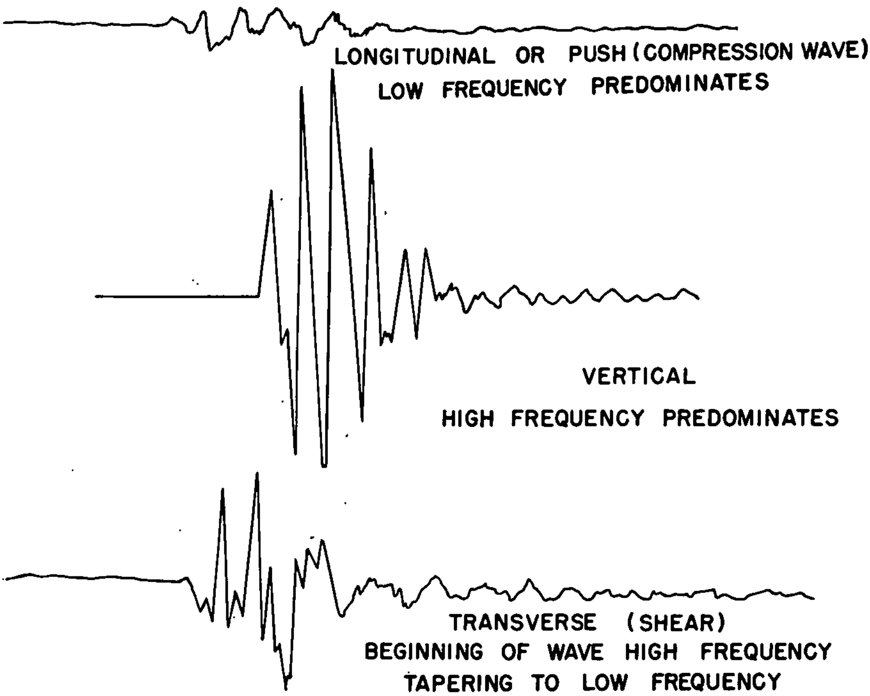


FIG. 8.—COMPARISON OF FREQUENCIES.

The final tracing of the original record is in the transverse plane, which can be called the "shear" wave in the earth.

At the beginning of the vibration, as it is first picked up by the instrument, the high frequencies definitely predominate, but they are rapidly wiped out by the low frequencies which give us the maximum energy in this plane.

Because the vibrations in all three planes are not in phase, and the frequencies in each plane differ, a very complicated movement of the ground due to the vibrations set up by the blast is indicated. No doubt each individual vibration from any instantaneous shot contains what might be called a spectrum of frequency, the high frequencies being predominant at the beginning of the vibrations in both the vertical and transverse planes, and the low frequencies in the longitudinal plane.

In measuring the record, the maximum combination of high amplitude and low frequency was always determined because this gave the highest E. R. The maximum E. R. was obtained in the transverse plane a short time after the vibration started in this plane. The high frequency vibration was overcome by the more forceful low frequency ones.

The maximum combination of amplitude and frequency was measured in each separate plane, even though they did not come at the same instant. This procedure gives a maximum E. R. in each plane. The vector sum of these maximum E. R.'s would give a resultant greater than expected. However, this resultant would be definitely on the safe side.

ENERGY RATIO

Because of our previous assumption that the mass of earth in vibration was unknown and that therefore in like locations the mass was the same, we made the basic assumption that E. R. $= \frac{a^2}{n^2}$.

Our investigation and test has shown that when the E. R. in the ground becomes three or more, old pre-stressed structures are likely to be damaged. Further, when the E. R. in the ground becomes six, damage will occur to residential types of buildings or brick wall bearing structures.

Because the E. R. is made up of the square of the acceleration divided by the square of the frequency, it is readily seen that if the frequency is high, the acceleration can also be high and still be within a safe range.

A visual indication of this ratio of acceleration and frequency is shown in Figure 9. A study of this chart indicates that when the frequency of vibration in the earth strata is approximately 2, an acceleration of 3½ feet per second per second will reach the damage threshold and one of 4.8 feet per second per second is likely to cause damage. On the other hand, if the frequency within the earth strata itself is 15, an acceleration of 24 feet per second per second would have to be induced in order to reach the threshold limit.

Thus, in a stiff clay medium, whose frequency may be in the vicinity of 20 cycles per second, a relatively large acceleration will not bring us into the damage zone. On the other hand, if there is a sedimentary deposit completely saturated with water, the frequency will be low, probably in the vicinity of 5 to 7 cycles per second, and the damage range of the structure will be reached very quickly.

Because the E. R. varies inversely with the square of the fre-

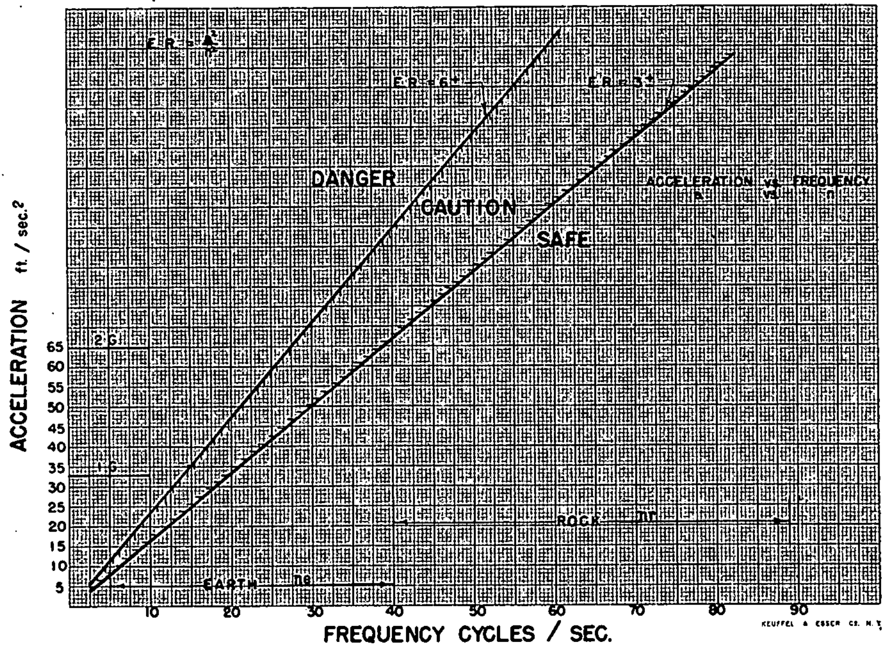


FIG. 9.—ENERGY RATIO LIMIT CHART.

quency and the constant in the equation $C^2 K = E. R.$ only fluctuates from approximately 0.001 to 0.004 in our findings, it appears safe to say that as the frequency of the strata increases, more pounds of dynamite can be used per instant or per delay and still stay within the safe range.

The ground constant K can readily be found in preliminary tests and the average frequency of the strata involved can be indicated as well. By use of the Energy Ratio Chart at a 50 foot distance, the number of pounds of dynamite allowable to keep within the safe range as shown by the lower threshold mark on the Energy Ratio Chart can be found with relative ease.

As our research has shown, for the distances involved, i.e. 50 to 250 feet the E. R. decreases inversely with the distance. Therefore by inserting the distance D in the equation the fluctuation in pounds allowable as distance increases can be determined with relative ease.

It is interesting to note that as the frequency of the strata decreases, the range between the lower threshold limit and actual danger points becomes very small. The lowest frequencies that we have been able to measure were in swampy ground that had a frequency of 7 cycles per second. The earthquake frequencies as measured by the Coast and Geodetic Survey instruments show frequencies as low as 1 to 3 cycles per second. At these frequencies, even 10 per cent of gravity brings the E. R. into the damage zone.

By the same token, as frequencies such as those found in the rock strata increase, accelerations equal to 1-g or even 2-g can be obtained without exceeding the safe E. R. limit.

Displacement vs. Frequency

Ground energy ratio can be used with displacement and frequency measurements as well as acceleration frequency measurements. The Kinetic Energy equation can be set up in relation to displacement and frequency as well as acceleration and frequency as follows:

$$\text{K.E.} = \frac{W V^2}{2g} = \frac{W}{2g4\pi^2}\frac{a^2}{n^2} = \frac{W}{2g4\pi^2}\,16\pi^4n^2D_1^{\,2}$$

or

$$\frac{a^2}{n^2} = 16\pi^4n^2D_1^{\,2}$$

Because $\frac{a^2}{n^2}$ is designated as Energy Ratio, in regard to acceleration then $16\pi^4n^2D_1^{\,2}$ can be designated as Energy Ratio in regard to displacement.

$$\text{E. R.} = 16\pi^4n^2D_1^{\,2}$$

Taking lower threshold as 3

$$\text{Then E. R.} = 3 = 16\pi^4n^2D_1^{\,2}$$

$$\text{or } D_1 = \frac{.0439}{n}$$

Taking upper threshold as 6

$$\text{Then E. R.} = 6 = 16\pi^4n^2D_1^{\,2}$$

$$\text{or } D_1 = \frac{.0621}{n}$$

By use of a displacement instrument similar to the Leet Portable Seismograph ground measurements can be taken to determine the

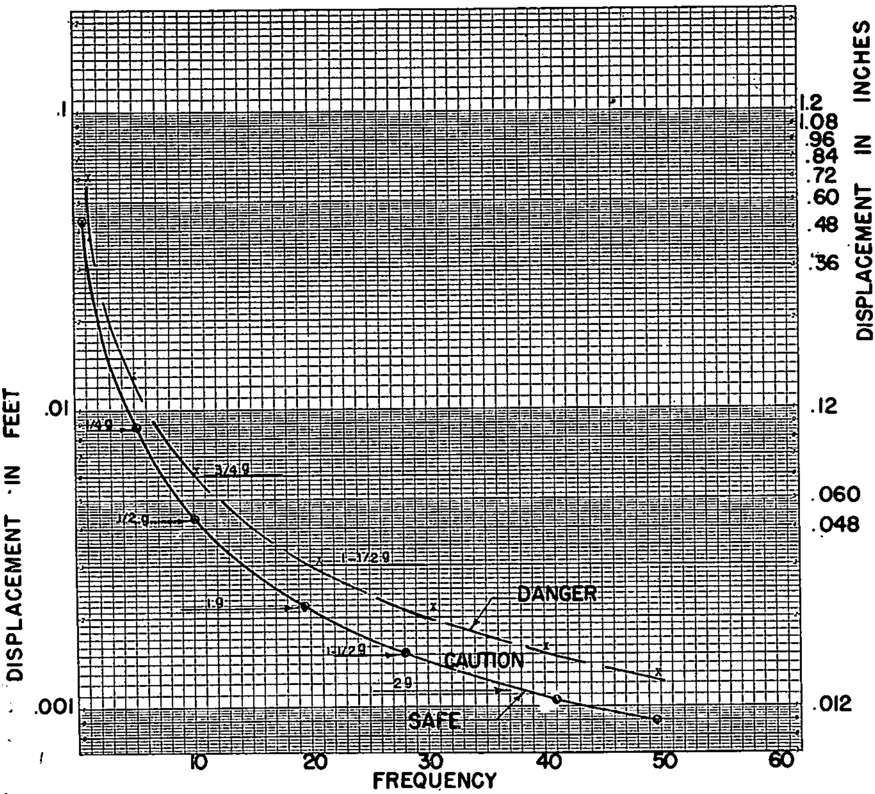


FIG. 10.—ENERGY RATIO LIMIT CHART, DISPLACEMENT VS. FREQUENCY.

amount of vibration transmitted in relation to the weight of dynamite used in the detonation. Figure 10 indicates the safe range and the danger range of ground vibration for an energy ratio of 3 as the lower threshold limit and an energy ratio of 6 as the upper threshold limit. The equation for the number of pounds of dynamite allowable in any instant will be the same or $\left(\frac{50}{D}\right)^2 C^2 K$ will be the same except for the right-hand side of the equation. The fraction $\frac{a^2}{n^2}$ would change to $16\pi^4 n^2 D_1^2$. Therefore the equation in using displacement and frequency measurements will be asfollows:

$$\left(\frac{50}{D}\right)^2 C^2 K = 16\pi^4 n^2 D_1^2$$

In this equation D_1 is equal to the ground displacement and n is equal to the frequency of the ground vibration.

With control tests and equating the right-hand side of the equation = 3 as the lower limit, knowing the number of pounds of dynamite used in the control tests, K can be found, thus allowing one to determine the safe number of pounds of dynamite that can be used per delay, determined from displacement and frequency measurements of the ground.

Owners' Reaction to Vibration

The limits of ground vibration needed to cause damage to structures having been determined by these experiments, the question arises as to the Level of Human response to vibration. Physiological tests have established the lower limit of human response to vibration to be 1/100 of the vibration that causes damage to ordinary structures.

Our experiments showed that it is possible to predetermine the amount of explosive that can safely be used in any blast without injuring the adjacent structure, but claims as to damage were still being made by the owners. Investigation of these claims showed that no structural damage had occurred. From this we can conclude that because the vibration was felt by the people living in the homes adjacent to the blasting, that they assumed damage occurred. This in all probability is because of the ability of people to feel vibration long before any structural damage could occur.

A very thorough study of human response to vibrations was made by Reiher and Meister at the Technical University of Stuttgard in Germany about 1931.

This experiment was performed by vibrating a freely suspended platform, $6' \times 4'$, by means of an unbalanced electric motor between 3 to 70 cycles per second and .00004" amplitude.

The results are shown in Figure 11 in accelerations and in Figure 12 in displacements in inches vs. frequency.

When the subjects were standing, they were much more sensitive to vertical than horizontal vibrations. When lying down the vertical vibrations were not as noticeable as the horizontal and the motion perpendicular to the long axis of the body in the horizontal plane were most disturbing.

All subjects easily noticed the vibrations at a level 1/100 of that which was the caution range for structures. In addition, severe vibra-

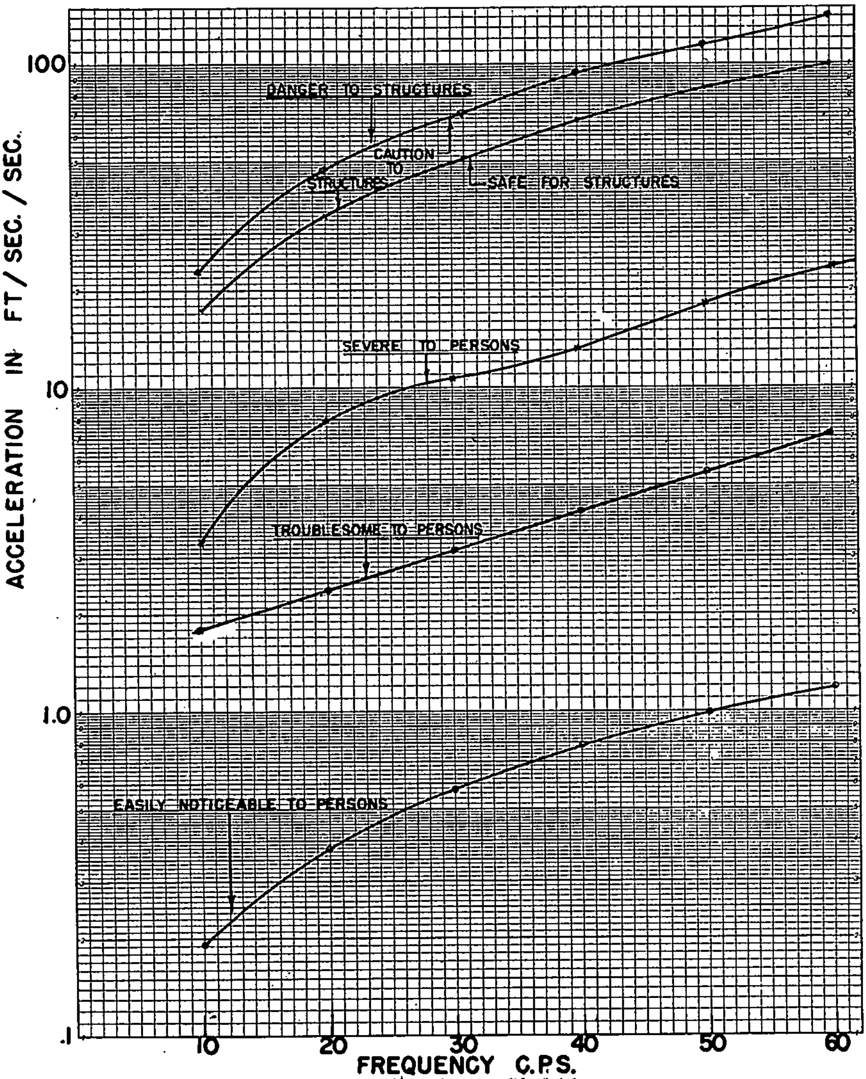


FIG. 11.—LEVEL OF HUMAN RESPONSE TO VIBRATION.

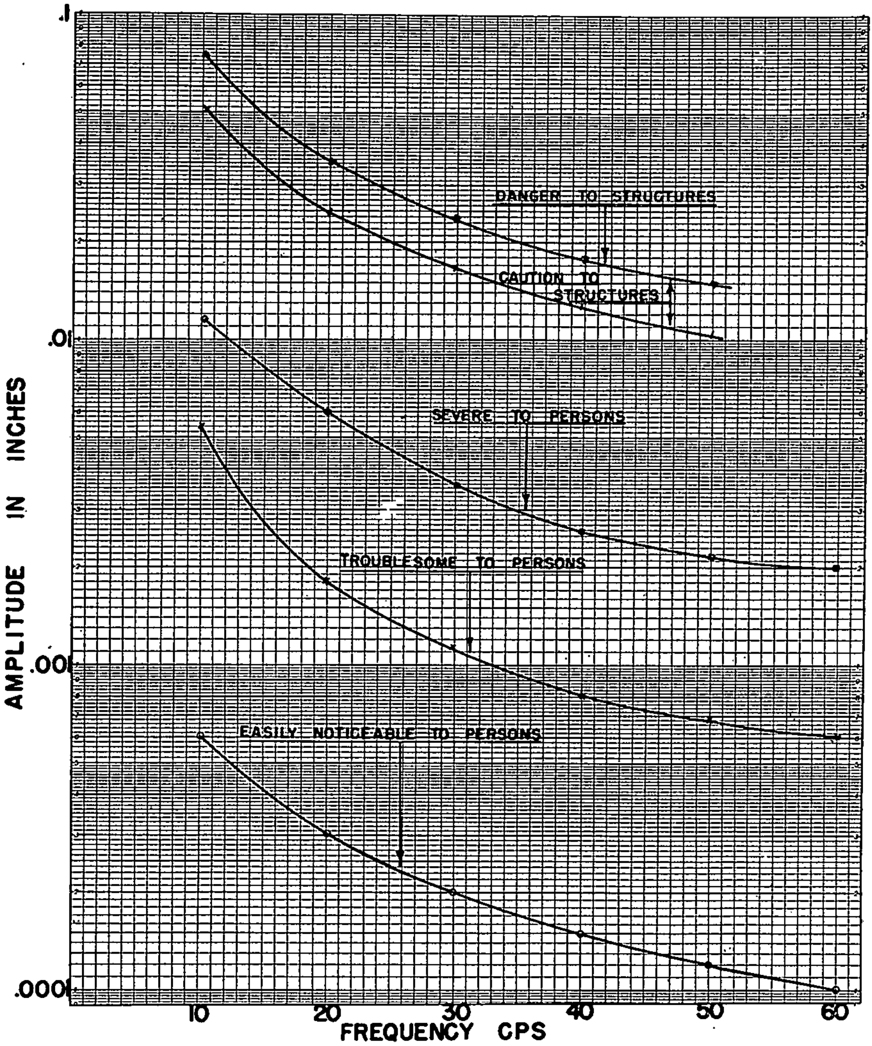


FIG. 12.—LEVEL OF HUMAN RESPONSE TO VIBRATION.

tions to persons, which comes about 1/5 of that needed to damage structures, would be very startling to a house owner, although it would not cause damage to his structures.

Figure 12 plotting amplitudes vs. frequency shows that at 20 cycles per second, a displacement of no more than the thickness of a piece of paper is easily felt by persons and a vibration of only 1/64″ would be a severe vibration to persons.

The noise accompanying a blast and the fact that persons feel very small vibrations lead many owners to assume that the vibration is enough to injure their structure based upon their own response to the movement.

It might be advisable for the contractor performing the operation to warn the property owners as to the time of each blast, thus eliminating the element of surprise that comes from the noise and vibration.

At the same time informing the owners that tests have been made to determine the ground constants so that the explosives are limited to insure no damage to ordinary structures will more likely obtain better owner cooperation.

General Summary

1. It is practical to use the measured ground accelerations and frequencies to indicate the ratio of energy being transmitted through the ground.

2. An investigation of over 1,000 residential homes, stores of 2 stories high, some schools, churches, and hospitals that were investigated before the disturbance or vibration occurred and after the vibration occurred, shows that if the energy ratio as measured by acceleration and frequency is kept below 3 there will not be damage to buildings of average workmanship and good materials.

3. A study of the records shows that the energy ratio is proportionate to the amount of explosives detonated at any instant. Therefore the use of delays in any general shot allows a control of the amount of energy ratio transmitted through the ground.

4. A study of the wave train records shows that the instantaneous shot invariably produced a greater energy ratio in proportion to the pounds used, than any other delay. Bureau of Mines averaged this instantaneous wave to be 11 times the following waves due to delays.

5. Equation $\left(\frac{50}{D}\right)^2 C^2 K = E. R.$ is practical in predetermining

the amount of vibration for energy ratios that will be transmitted through the ground.

It is only necessary to measure the transmission and determine K by use of known amounts of dynamite, determine the distance of the nearest structures to be able to determine the number of pounds of dynamite that safely can be used, without injury to the structures.

6. When using ordinary delays, the wave trains from each separate delay invariably died out before the next delay was detonated.

7. At no time did we find the wave trains overlapping from one delay to the next.

8. The range of this investigation covered the distance from 25 to 250 ft. and charges of explosives from one pound to one-hundred pounds. It is conceivable that for greater distance and greater poundage, this equation may not hold wholly true.

9. Displacement and frequency can be used as well as acceleration and frequency in determining E. R. as shown in Chart 2.

10. Experimentation shows that people are very sensitive to vibration, and the lower level of human response to vibration, which is easily felt by all persons, is 1/100 of the vibration that approaches the caution range for structures.

ACKNOWLEDGMENTS

The measurements and investigations that made available the date for this report were performed by Engineers Arthur Gordon, R. T. Halbert and James A. Shaw of the Liberty Mutual Insurance Company. Preliminary and post examinations of structures made by Kuehl and Heavey, Consulting Engineers, Chicago, allowed sufficient data to establish the limits of damage.

The writer wishes to express his gratitude to Dr. L. Don Leet for his help and suggestions during the course of this study and for his assistance and suggestions during the writing of this report. Dr. L. Don Leet is the seismologist for the Harvard Seismological Station, Harvard, Massachusetts and a Professor at the Harvard University.

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Abbreviations

a = Acceleration in feet per second.
n = Frequency of vibration in cycles per second.
K.E. = Kinetic Energy in foot pounds.
E.R. = Energy Ratio $= \dfrac{a^2}{n^2}$.
W = Weight in pounds.
g = Acceleration of gravity in feet per second per second.
K = Constant (depending upon local ground conditions).
C = Pounds of explosive detonated at any instant.
D = Distance from the center of detonation in feet.
D_1 = Ground displacement.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

JANUARY 26, 1949.—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.

President Frederic N. Weaver presided at the meeting and announced the following deaths:

Frank H. Mason who was elected a member March 20, 1929 and who died December 21, 1948.

William J. Sullivan who was elected a member December 20, 1922 and who died December 31, 1948.

President Weaver stated that at the last meeting of the Society held on December 15, 1948, the Treasurer presented a recommendation of the Board of Government which was acted upon favorably at that meeting and that final action on the recommendation would now be taken.

The Treasurer read the following recommendation:

MOTION "That the Board of Government be authorized to transfer an amount not to exceed $5,000 from the Principal of the Permanent Fund to cover unusual expenditures of the Centennial Year and to balance the Secretary's Budget at the end of the year."

On motion duly made and seconded it was unanimously VOTED "that the recommendation of the Board of Government be adopted." President Weaver stated that this was the final action to be taken on this matter.

President Weaver introduced the speaker of the evening, Mr. Henry N. Halberg, Associate Engineer, U. S. Geological Survey, Water Resources Branch, Boston, Mass., who gave a most interesting illustrated talk on "Ground Water Resources in Greater Boston". A discussion period followed in which many of the members and guests took part.

At the close of the meeting President Weaver announced that a collation would be served in the lounge on the floor above.

Eighty-one member and guests attended the meeting.

The meeting was adjourned at 9:20 P.M.

ROBERT W. MOIR, *Secretary*

FEBRUARY 16, 1949.—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. This was a joint meeting with the Structural Section, B.S.C.E.

President Frederic N. Weaver presided at the meeting and announced that the Annual Meeting would be held on March 16, 1949, at the Hotel Vendome, 160 Commonwealth Avenue, Boston, Mass. Mr. Bradford Washburn, Director of Boston Museum of Science, will be the speaker.

President Weaver called upon Mr. Oliver G. Julian, Chairman, Structural Section to conduct any business matters necessary for the Structural Section.

The Secretary announced the following had been elected to membership on February 14, 1949:

Grade of Member: Harry L. Kinsel, Donald W. Scully, Howard Simpson, Joseph W. Lavin*, Alland L. Levy*, Daniel J. Conlin*, Paul D. Killam*, Ernest P. Demers†.

President Weaver called upon Mr. O. G. Julian to introduce the speaker of the evening:

Mr. Edward B. Oberly, Construction Superintendent, Portland Cement Association, New York, who gave an interesting illustrated talk on "Architectural Concrete".

At the close of the meeting the members gathered in the lounge where a collation was served.

There were sixty-eight members and guests present.

The meeting was adjourned at 9:00 P.M.

Respectfully submitted,
ROBERT W. MOIR, *Secretary*

MARCH 16, 1949.—The one hundred first 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 the President, Frederic N. Weaver.

The minutes of all previous meetings of the current fiscal year which have been printed in the various issues of the Journal were approved as printed.

The Annual Reports of the Board of Government, Treasurer, Secretary and Auditors were presented. Reports were also made by the following committees: Hospitality, Relations of Sections to Main Society, Library, John R. Freeman Fund, Subsoils of Boston, Membership, Competitive Bidding for Engineering Services, Quarters, Building Laws and Building Construction, Publicity, Safety, Insurance and By-Laws.

It was VOTED "That the reports be accepted with thanks and placed on file and that they be published as a part of the Annual Report in the April, 1949, issue of the Journal."

The Annual Reports of the various sections were read.

It was VOTED "That the Annual Reports of the various Sections be accepted and that they be published as part of the Annual Report in the April, 1949, issue of the Journal."

The report of the Tellers of Election, George W. Coffin and Charles T. Main, 2nd, was presented and in accordance therewith the President declared the following had been elected officers for the ensuing year:

President—Harrison P. Eddy, Jr.
Vice-Pres. (for one year)—Thomas R. Camp
Vice-Pres. (for two years)—John B. Wilbur
Secretary (for one year)—Robert W. Moir
Treasurer (for one year)—Herman G. Dresser
Directors (for two years)—Edwin B. Cobb, Frank L. Lincoln
Nominating Committee (for two years) — Charles M. Anderson, George G. Bogren, Murray H. Mellish

The retiring President, Frederic N. Weaver, then gave his address on "The Second Hundred Years".

Eighty 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.

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

Following general remarks and the introduction of the newly elected President, Mr. Harrison P. Eddy, Jr., and other guests at the head table the various prize awards were made.

The Secretary read the various prize awards and asked the recipient to come forward and President Weaver presented the following awards:

Award	*Recipient*	*Paper*
Desmond FitzGerald Medal	Thomas R. Camp	"The Merrimac River Valley Sewerage District Project"
Clements Herschel Award	John B. Babcock, 3rd	"Centennial History of the Boston Society of Civil Engineers, 1848-1948"
Structural Section Award	Robert J. Hansen	"Long Duration Impulsive Loading of Simple Beams"
Hydraulics Section Award	George E. Rich	"Basic Hydraulic Transients"
Surveying & Mapping Section Award	Charles M. Anderson	"The Engineering Aspects of the Land Court"

The President stated that special recognition was being made to Everett N. Hutchins, for his long service to the Society.

President Weaver presented Mr. Hutchins with Engrossed Resolutions which read as follows:

BOSTON SOCIETY OF CIVIL ENGINEERS

MR. EVERETT N. HUTCHINS

WHEREAS, Mr. Everett N. Hutchins has served the Boston Society of Civil Engineers as Secretary and Editor of the Journal for eighteen years, and Vice-President for the past two years;

WHEREAS, Mr. Hutchins was asked by the Nominating Committee to be its nominee for President for the coming year but was unable to accept;

WHEREAS, it is recognized that Mr. Hutchins' long and steadfast service, covering nearly a fifth of the life of the Society, has in large part been responsible for the continued high ethical and professional standards of the Boston Society of Civil Engineers and the excellence of its publications;

Be It Therefore Resolved that, in recognition of his great service, the Boston Society of Civil Engineers, through its Board of Government, hereby expresses to Mr. Hutchins its appreciation and gratitude for his untiring efforts in its behalf over the many years of his services; and

Be It Further Resolved that the Society cause these resolutions to be engrossed and presented to Mr. Hutchins at the Annual Meeting, March 16, 1949.

President Weaver then introduced the speaker of the evening, Mr. Bradford Washburn, Director of Boston Museum of Science, who gave a most interesting talk on "The Conquest of Mr. McKinley". The talk was illustrated with colored movies and slides.

At the conclusion of the address President Weaver on behalf of the Society thanked Mr. Washburn for a most enjoyable talk and then turned the meeting over to President elect, Harrison P. Eddy, Jr., who adjourned the meeting at 9:30 P.M.

Two hundred twenty-three members and guests attended the meeting.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

OCTOBER 6, 1948.—A meeting of the Sanitary Section was held at 7:00 P.M. in the Society Rooms. A dinner at Patten's Restaurant preceding the meeting was attended by 35 members and guests. There were 54 present at the meeting.

The speaker of the evening was Mr. Stuart Coburn, Chief Chemist for Metcalf and Eddy. He was presented by Chairman Allen J. Burdoin. Mr. Coburn's subject was "Headaches from Combined Treatment of Industrial Wastes and Sewage."

Mr. Coburn with some 35 years of experience in this field was exceptionally well qualified to discuss this subject. He described the various substances discharged by industries into the sewers and the effect of these substances on sewage treatment works. He told of the methods that had been developed for reducing these compounds to a less harmful state. He recommended the exclusion of others by suitable ordinances. Some treatment plants which had failed to perform their function because of the uncontrolled dumping of certain industrial wastes were named and the trouble in each instance was described.

Prepared discussion was given by Mr. Joseph McCarthy of the Lawrence Experiment Station and Professor C. N. Sawyer of Massachusetts Institute of Technology.

The discussion period continued with Messrs. Thomas Camp and Sherman Chase contributing additional information on the subject.

The meeting was adjourned at 9:00 P.M.

FRANK L. HEANEY, *Clerk*

OCTOBER 14, 1948.—A joint meeting of the Sanitary Engineering Division of the American Society of Civil Engineers and the Sanitary Section of the Boston Society of Civil Engineers was held at 10:00 A.M. in the Ballroom of the Hotel Statler. The occasion was the Fall Meeting of the American Society of Civil Engineers which was held in Boston this year to help commemorate the Centennial of the Boston Society of Civil Engineers.

Dean Fair, representing the American Society of Civil Engineers, presided over the first half of the session. Mr. Allen J. Burdoin, Chairman of the Sanitary Section of the Boston Society of Civil Engineers, presided over the second half.

Four paper were presented:

1. "Pollution Abatement Policy" by Thomas R. Camp of Camp, Dresser and McKee.
2. "Pollution of the Androscoggin River by Industrial Wastes and Control Measures Thereof" by E. Sherman Chase of Metcalf and Eddy.
3. "Utilidors for Water, Sewer, and Other Underground Utilities in Arctic Climates" by William L. Hyland and Murray H. Mellish of Fay, Spofford and Thorndike.
4. "The New Water Supply Tunnel of the Boston Metropolitan District Commission" by Karl R. Kennison, Chief Engineer, Construction Division, Metropolitan District Commission.

One hundred fifty-one members and guests of the two Societies were present.

FRANK L. HEANEY, *Clerk*

DECEMBER 1, 1948.—A meeting of the Sanitary Section was held at 7:15 P.M., in the Society Rooms. A dinner at Patten's Restaurant preceding the meeting was attended by 55 members and guests. There were 96 present at the meeting.

A short business meeting, conducted by Chairman, Allen J. Burdoin, preceded the presentation of the speakers. A Nominating Committee was elected consisting of:

Frederick S. Gibbs, Chairman
Murray H. Mellish
Walter E. Merrill

Mr. Ralph Soule moved that the Executive Committee be instructed to either find a new place to eat or else start the dinner earlier. After discussion it was voted to instruct the Executive Committee to get in touch with Patten's Restaurant to see if faster service could be obtained.

The Chairman then presented the speakers of the evening: Messrs. E. Sherman Chase and Charles G. Hammann. The subject of their paper was "The Proposed Blackstone Valley Sewage and Wastes Collection System and Treatment Works". Mr. Hammann described the series of events leading up to the forming of the Blackstone Valley Sewer District Commission. Mr. Chase described the methods proposed for the abatement of this pollution. After considerable discussion the meeting was adjourned.

FRANK L. HEANEY, *Clerk*

STRUCTURAL SECTION

DECEMBER 8, 1948.—The meeting was called to order at 7:30 P.M., by Chairman, Oliver G. Julian, at the Society Rooms, after a dinner held at the Smorgasbord at which 24 attended.

The minutes of the previous meeting were read and accepted.

Dr. Charles H. Norris, Professor of Structural Engineering, M.I.T., was introduced as the speaker. His subject was "Localized Buckling of Structural Members".

He warned that the talk would be somewhat mathematical as he was developing the ultimate strength of steel due to the buckling resistance of parts. He showed how he had developed formulae and also showed tests that were made in a special machine developed for that purpose. The gap width on the edge of the plates was varied by means of yokes in supporting channels. His talk was illustrated with many lantern slides and concluded at 8:15 P.M.

At the extended discussion that followed the following participated, Prof. Reissnor, Prof. Haertlein, Prof. Wilbur, Prof. Peabody and Messrs. Lasker, Torrence, Thompson, Cundari, Probst and Fullerton.

A motion was made and voted that plans be made to have this paper published in the Journal.

The meeting adjourned at 9:05 P.M.

Fifty-three members and guests attended the meeting.

ARTHUR E. HARDING, *Clerk*

JANUARY 12, 1949.—The meeting was called to order at 7:20 P.M., by Chairman, Oliver G. Julian, at the Society Rooms after a dinner held at the Smorgasbord at which 29 attended.

The minutes of the previous meeting were read and accepted.

The minutes of the Structural Section Executive Committee meeting of December 18, 1948, were read. A motion was made and voted that the Section be recorded as in accord with the thoughts contained in the report.

A motion was made and voted that the Chairman appoint a Nominating Committee of three to bring in nominating for officers for the coming year.

Mr. James H. Carr, Jr., of the Timber Engineering Company was introduced as the speaker. His subject was "Wood Trusses".

Mr. Carr said that wood trusses were used quite extensively during the last war because of the shortage of steel and a great deal was learned as to the possibilities. The largest Timber building constructed was the Chicago Aircraft Assembly Plant which was 800 ft. wide and 300 ft. long. The largest clear span was the Minneapolis-St. Paul Airplane Hangar with a 260 ft. span. The Blimp Hangars were 137 ft. span, 153 ft. height and 1000 ft. long.

The three major faults that developed during the war were shrinkage caused by having to use green lumber, eccentric joints and deflection. Other faults were poor design due to incompetence, poor fabrication, poor tools, poor erection and poor maintenance due to agencies not tightening bolts.

Items mentioned were use cement line load, allowable stresses for timber grade used, plastic flow, joint shippage, and edge distance of connectors.

Results of tests made on a Pratt Truss and a Lank-Teco Truss both of 48 ft. span were shown.

The talk was illustrated by lantern slides and a short movie.

An extended discussion period followed. A vote of thanks was extended to Mr. Stuart Huckins for his efforts in arranging for the speaker.

A vote of thanks was extended to Mr. James H. Carr, Jr., for his fine talk.

The meeting adjourned at 9:35 P.M. Fifty-two members and guests were present.

ARTHUR E. HARDING, *Clerk*

FEBRUARY 16, 1949.—The joint meeting of the Boston Society of Civil Engineers and the Structural Section of the same society was held on this date at 7:00 P.M., at the American Academy of Arts and Sciences, 28 Newbury Street, Boston. Mass.

The meeting was opened by the President of the Society, Frederic N. Weaver, who then turned the meeting over to Oliver G. Julian, Chairman of the Structural Section. The reading of the Secretary's report was omitted on account of the absence of the Secretary.

The nominating committee read its report for nominating for officers for the coming year to be acted on at the next meeting of the Section.

The speaker, Mr. Edward B. Oberly, Construction Superintendent of the Portland Cement Association, was then introduced by Mr. Julian.

Mr. Oberly gave a very instructive talk on architectural concrete. He talked about the form required for this work, the type of concrete to be used, air-entrained concrete, and concrete finished after the forms are stripped. His subject was illustrated with slides. A very interesting question period followed. A collation was served following the meeting.

Sixty-eight members and guests attended this meeting.

FRANK A. CUNDARI, *Acting Clerk*

MARCH 9, 1949.—The meeting was called to order at 7:30 P.M., in the Society Rooms, by Chairman Oliver G. Julian.

The nominating committee read its report of the following names for officers for the coming year:

Chairman—Ernest L. Spencer
Vice-Chairman—Arthur E. Harding
Clerk—Edward C. Keane
Exec. Comm.—Frank A. Cundari, Charles H. Norris, Jean M. Ducharme

It was moved, seconded and voted that the nominations be closed. It was moved, seconded and voted that the clerk cast one ballot for those named above for a slate of officers for the coming year.

The minutes of the last meeting were read by Acting Clerk Frank A. Cundari and voted accepted.

The minutes of the Executive Committee report were read by Clerk Arthur E. Harding and voted accepted.

Mr. LaMotte Grover, Chairman Structural Steel Committee, Welding Research Council was introduced by the Chairman and the topic of his paper was "Theory and Practice of Welding of Bridges and Buildings".

He noted the great strides that have been made in recent years in the designing of welded connections both in the shop and in the field. He also told of the work necessary for the shops to change over equipment to do welded work when called upon to do so and also still do riveted work. He gave as an example of a large local project that is using welded construction as the new Jordan Marsh Company store. He also told of the research work being done at Massachusetts Institute of Technology and Lehigh University. His paper was completed at 8:50 P.M. and an extended discussion period followed. The paper was well illustrated with lantern slides.

The meeting adjourned at 9:30 P.M. There were 75 members and guests present.

ARTHUR E. HARDING, *Clerk*

TRANSPORTATION SECTION

MARCH 2, 1949.—The Nominating Committee of the Transportation Section, consisting of William C. Paxton, Chairman, George W. Hankinson, and Francis T. McAvoy, presented the following slate of officers for the ensuing year:

Chairman—William L. Hyland
Vice-Chairman—Albert Adelman
Clerk—George G. Hyland
Executive Committee—
R. Newton Mayall, Herman J. Shea, Ernest L. Spencer

After voting that the nominations be closed, the Section voted that the Clerk cast one ballot for the nominees.

The Section was addressed by Mr. Russell C. Chase, Engineer, Shell Oil Company, on the subject "The Practical Aspects of Design and Construction of Asphalt Pavements". Mr. Chase showed a colored film, which he himself had prepared, illustrating the use of asphalt in pavement construction and the manner in which laboratory tests of asphalt are conducted.

Coincidentally, he delivered a prepared narrative describing the film. Following the film Mr. Chase talked extemporaneously on the construction of asphalt pavements. After the talk there was a general discussion.

The attendance was thirty.

WILLIAM L. HYLAND, *Clerk*

HYDRAULICS SECTION

FEBRUARY 2, 1949.—A meeting of the Hydraulics Section was held in Tremont Temple Social Hall following a dinner at The Smorgasbord.

The meeting was called to order at 7:15 P.M. by Chairman John G. W. Thomas. During a brief business session, a nominating committee composed of Leslie J. Hooper, Chairman, Allen J. Burdoin, and Harold A. Thomas, Jr., submitted the following slate of officers for the coming year:

Chairman—James F. Brittain
Vice-Chairman—Elliot F. Childs
Clerk—Gardner K. Wood
Members of Executive Committee—
Byron O. McCoy, Lincoln W. Ryder, Arthur T. Ippen

The above slate was elected in the ensuing vote.

The speakers of the evening were Rev. Daniel Linehan, S.J., Seismologist-in-charge, Seismological Observatory, Weston College, and Mr. Scott Keith, Project Engineer, Metcalf and Eddy, Engineers. They had as their subject "Seismic Reconnaissance for Underground Conditions." Their talks illustrated that the application of seismic methods, popularly connected with the oil industry, has broadened to include the fields of Foundations and Ground Water Development.

At the conclusion of the lecture, Clyde Coppage, Worcester Airport Engineer, discussed the results of seismic investigations at the Worcester Airport. Additional discussion indicated the interest of the large group in attendance. The meeting adjourned at 9 P.M. with a rising vote of thanks to the speakers.

One hundred and seven members and guests were present at the meeting.

GARDNER K. WOOD, *Clerk*

SURVEYING AND MAPPING SECTION

JANUARY 19, 1949.—The seventh meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:15 P.M. on this date.

Approximately 60 members and guests were present.

The meeting was called to order by Chairman Charles M. Anderson. The minutes of the previous meeting were read and approved.

Chairman Anderson called for the report of the Nominating Committee which had been elected at the October meeting, which report was read by Leonard S. Hubbard and the original is attached hereto.

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

On motion duly 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—Hugh P. Duffill
Vice-Chairman—Herman J. Shea
Clerk—John H. Lowe
Executive Committee—
Louis A. Chase, William C. Moberger, John J. Vertic

Chairman Anderson appointed the following members to serve on the Committee authorized by the Section at the October meeting to keep the members informed as to any legislative measures or recommendations of other Societies pertaining to Registered Professional Engineers and Land Surveyors: Frederick C. Joy, Jr., R. Newton Mayall, L. T. Schofield, Chairman.

Chairman Anderson introduced Mr. William W. Drummey, Boston Architect, who spoke on the subject of "The Relationship Between Architect and Engineer-Surveyor", and Mr. R. Newton Mayall, Landscape Architect, who spoke on "The Relationship Between the Landscape Architect and the Engineer-Surveyor".

Following these talks a long and interesting discussion was held in which Prof. Charles O. Baird, Jr., Wesley B. Thompson and many others took an active part.

The meeting adjourned at 9:00 P.M.

HUGH P. DUFFILL, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[July 1, 1948]

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

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

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

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

For Admission

STEPHEN E. DORE, JR., Canton, Mass. (b. April 1, 1918, Providence, R. I.) Graduated from Brown University in Providence, R. I., in June, 1940, with Sc.B. degree in Civil Engineering. Experience, upon graduation worked as a Transitman with the Met. Dist. of Hartford County in Conn. until about November, 1940, when started work as a Junior Civil Engineering draftsman in the Navy Design Dept., at Quonset Point Naval Air Station, R. I. Became Engineer Draftsman March, 1941, and continued as such until May, 1942. At this time resigned to accept a commission as 2nd Lt. in the Corps of Engineers, Army of the United States. Remained in active service until October, 1945, and was then placed on terminal leave until January, 1946. Various assignments included: platoon commander, ass't operations officer, operations officer, and liaison officer in a Gen. Service Engr. Regiment and an Engr. Combat Battalion engaged in active operations in Africa and Italy. Highest rank attained—Captain. Upon release from active service worked as a Road Draftsman for the Div. of Roads and Bridges, State D.P.W. of R. I., until February, 1946, at which time accepted a job as Hydraulic Engineer in the Providence office of the U. S. Army Engineers. This office was closed in September, 1946, and was employed by E. B. Badger & Son of Boston, Mass., as structural designer. Remained with this firm until November, 1947, then left to accept a position as Civil Engineer in the office of Coffin & Richardson, Engineers, Boston, Mass. Refers to *S. M. Dore, G. W. Coffin, L. J. Fenocketti, R. W. Moir.*

C. FRANK JOHNSON, Quincy, Mass. (b. September 16, 1897, Mayfield, Kentucky). Received B.S. degree in Civil Engineering from University of Kentucky in 1938. Experience, 1919-1921, assistant engineer on land reclamation work with W. C. Kelly, Civil Engineer, Union City, Tenn.; 1922-1923, assistant engineer on municipal work, officer of city engineer, Paducah, Kentucky; 1924-1930, designing engineer and (later) project engineer on sewerage design and construction work, office of city sanitary engineer, Grand Rapids, Michigan; 1930-1944, junior engineer to Chief Engineer on sewerage design and construction work, with Commissioners of Sewerage of Louisville, Kentucky (a municipal corporation); 1944 to date, project engineer on sewerage and other work, with Metcalf & Eddy, Consulting Engineers, Boston, Mass. Refers to *E. B. Cobb, D. F. Coburn, E. S. Chase, A. J. Burdoin, H. P. Eddy, Jr.*

JOHN D. M. LUTTMAN-JOHNSON, Cambridge, Mass. (b. December 28, 1905, Grahamstown, South Africa.) 1928, awarded Certificate in Civil Engineering by Imperial College of Science & Technology, London, England. Experience, 1929-1937, with Peirce, Leslie & Co., London and India, as assistant manager in India and Ceylon, carried out design and construction of waterfront structures, factory buildings, etc.; investigated mountainous road and river transportation projects; in technical charge of a match factory and a motor vehicle assembly plant; departmental charge of port operations, etc. As Branch Manager responsible for Company administration and finance, management of subsidiary companies, negotiation of contracts, control of office and factory staff. 1938-1945, joined the South African Railways & Harbours Administration as Assistant Engineer at Port Elizabeth. Over a period of seven years engaged on planning, layout and construction of major harbor works and port facilities. Four years in responsible charge of "New Works" Department with labor force of 400/800. Responsible for general administration and port maintenance. 1945 to date with Fay, Spofford & Thorndike, Boston, Mass., as Assistant Engineer, and Structural Engineer at Boston and as Project Engineer at Port of Whittier, Alaska, supervising construction. Engaged on design and layout of waterfront works, wharves, piers, bulkheads, intransit sheds, powerplant etc. Membership in technical societies include: American Society of Civil Engineers, South African Institution Civil Engineers, Society American Military Engineers. Registered Professional Engineer, Massachusetts. Refers to *R. E. Crawford, W. L. Hylund, F. L. Lincoln, M. H. Mellish.*

THOMAS J. LOGAN, Fall River, Mass. (b. September 21, 1900, Fall River, Mass.) Graduate of Wentworth Institute, Boston, Mass. Architectural construction course, year 1920. Experience, 10½ yrs., E. M. Corbett's office, Fall River, Mass.; 1½ yrs., Louis D. Destremps' office, both architectural firms. For last fifteen years Commissioner of Public Works of Fall River, Mass. As commissioner am City Engineer, Superintendent of Streets, Superintendent of Buildings and Wire Inspector having charge of above Divisions.

ALEXANDER E. MANNING, Milton, Mass. (b. June 10, 1910, Milton, Mass.) Employed by the Town of Milton, Engineering Department, for twenty years commencing on May 16, 1929, as a rodman, then a transitman, chief-of-party and in 1942 as Assistant Town Engineer. Graduated from Northeastern University, evening division, receiving Civil Engineering Diploma in 1934. Late in 1942 enlisted in the Army of the United States. Served for three years, two years as a master sergeant as "Chief of Survey" with a Port Construction and Repair Group. This unit worked in Georgia and Florida while in the United States and in England, Wales, and France in Europe. Completed assignments in the Pacific Theatre, stationed in Manila and working on projects in Corregidor, Bataan, and other parts of the Philippines. In 1945 returned to work for the Town of Milton as Assistant Town Engineer. Duties are varied, having a survey party and a constructon party as well as a certain amount of administrative work in the office. Refers to *F. J. Maynard, C. F. Joy, G. N. Watson, J. H. Bowie.*

Transfer from Grade of Student

DAVID I. LOWELL, West LaFayette, Indiana. (b. January, 1926, Newton, Mass.) Received B.S. in Civil Engineering from Northeastern University in June 1946. Special Investigations Officer 4th CIC District, 441st CIC Group. Refers to *C. O. Baird, E. A. Gramstorff, E. L. Spencer, W. E. Nightingale.*

ADDITIONS

Members

Henry R. DiCicco, 29 Tileton Street, Boston, Mass.

Daniel J. Conlin, 19 Wycliff Avenue, W. Roxbury, Mass.

Ernest P. Demers, Boston Street, Middleton, Mass.

John R. Hartley, 21 Linden Road, Barrington, R. I.

Harry L. Kinsel, 147 Lowell Avenue, Newtonville, Mass.

Joseph W. Lavin, 210 Market Street, Newark, N. J.

Alland L. Levy, 2220 Francis Drive, Palo Alto, California

Paul D. Killam, 70 Ocean Avenue, Salem, Mass.

Salvatore G. Puccio, 2 Hastings Street, W. Roxbury, Mass.

Howard Simpson, 86 Gerry Road, Chestnut Hill, Mass.

Henry E. Wilson, 494 Summer Street, Boston, Mass.

Junior Members

Ralph S. Archibald, 752 Franklin Street, Melrose, Mass.

Stuart M. Alexander, 602 Ins. Blldg., Denver, Colorado

Richard E. Sprague, 24 Neponset Road, Quincy, Mass.

Jerome T. McCullough, 252 Albion Street, Wakefield, Mass.

James H. Brown, 5 Gerry Street, Cambridge, Mass.

Student Members

Charles D. Adams, 11 Eustis Street, Arlington, Mass.

John J. Aherne, 37 Fairview Street, Roslindale, Mass.

Jack S. Barritt, 12 South Street, Jamaica Plain 30, Mass.

Paul H. Bedrosian, 101 High Street, Haverhill, Mass.

Warren H. Bell, 29 Queensberry Street, S. 15, Boston, 15, Mass.

William A. Billings, Jr., 76 Main Street, Woburn, Mass.

Peter K. Beshara, Jr., 52 Falmouth Street, Boston, Mass.

Edwin G. Calcagni, 89 Burrier Street, Barre, Vermont

Robert E. Cameron, 98 Gainsboro Street, Boston, Mass.

Allan L. Campbell, 29 Jefferson Street, Taunton, Mass.

Daniel Collins, 18 Short Street, Brookline, Mass.
William J. Collins, 316 Huntington Avenue, Boston, Mass.
Gordon E. Cossaboom, 17 Dunlap Street, Dorchester 24, Mass.
Bruce P. Eaton, 1 Lane Parkway, Danvers, Mass.
Richard F. Ebens, 85 High Street, Winchester, Mass.
Ralph W. Emerson, 20 Harrison Street, Framingham, Mass.
Stanley F. Gesek, 8 Pleasant Street, Salem, Mass.
Robert N. Glassman, 1539 No. Shore Road, Revere, Mass.
Henry G. Grilli, 142 Stratford Street, W. Roxbury 32, Mass.
David H. Hamilton, 232 Bellevue Street, Newton 58, Mass.
Frank C. Hancock, 235 Adams Street, Holliston, Mass.
Robert B. Hicks, Burley Street, R.F.D. Danvers, Mass.
Gordon T. Hoffman, 11 Vancouver Street, Boston 15, Mass.
Robert H. Homer, 226 Elm Street, Everett, Mass.
Donald H. Horrigan, 9 Burton Street, Brighton, Mass.
Robert P. Hunt, 398 Woodward Street, Waban 68, Mass.
Milton I. Isenberg, 320 Washington Avenue, Chelsea, Mass.
Edward J. Joyce, 343 Church Street, New Britain, Conn.
John P. Kirwin, 6 Harris Avenue, Everett, Mass.
William Kerr, 55A Englewood Avenue, Everett, Mass.
Robert T. Koopman, 450 Charles Street, Malden 48, Mass.
Saul Klashman, 88 Hancock Street, Cambridge 39, Mass.
Frank E. Kostka, 47 Kilton Street, Taunton, Mass.
Aaron M. Kreem, 19 Fayston Street, Roxbury, Mass.
Kay K. Krekorian, 47 Pinkert Street, Medford, Mass.
Arthur S. Lamprey, Jr., 46 Walter Street, Salem, Mass.
George E. Lybrand, 9 Cushing Street, Medford, Mass.
James E. Linehan, 22 Mason Street, Salem, Mass.
Allan B. MacPherson, 892 Essex Street, Lawrence, Mass.
Robert A. MacEwen, 26 Delano Park, Roslindale, Mass.
William F. McKay, Jr., 39 Cleveland Street, Gloucester, Mass.
Robert E. McGauley, 706 Main Street, Haverhill, Mass.
Leo McClung, 31 Grove Avenue, Hingham, Mass.
Dana Wesley McKechnie, 3 Ober Street, Beverly, Mass.
Robert L. Meserve, 64 Magoun Avenue, Medford, Mass.
John E. Meyer, 63 Euston Road, Brighton, Mass.
George Millman, 32 Florence Avenue, Revere, Mass.
John E. Murphy, 82 Gainsboro Street, Boston, Mass.
James P. Murray, Jr., 18 Euston Street, Brookline, Mass.
David W. O'Neill, 18 Welles Park, Roslindale 31, Mass.
Charles A. Parthum, 67 Loring Avenue, Salem, Mass.
Donald R. Paulson, 230 Market Street, Brockton 25, Mass.
Hiram Pearlman, 52 Suffolk Street, Malden 48, Mass.
Gino Perrotta, 54 Phipps Street, Quincy, Mass.
Leo D. Picardi, 61 Trenton Street, East Boston, Mass.
Angelo J. Polvere, 700 Hyde Park Avenue, Roslindale, Mass.
Raymond Quealy, 8 Gilson Terrace, Somerville, Mass.
Robert W. Richardson, 371 Waverly Oaks Road, Waltham, Mass.
William Ruble, 14 Woodbine Road, Medford, Mass.
Arlo R. Savery, 187 Copeland Street, Brockton 28, Mass.
Robert I. Scoville, Jr., 47 Waverly Street, Brookline, Mass.
Francis R. Sholock, 57A Jefferson Avenue, Chelsea 50, Mass.

Charles D. Shaker, 99 Harvard Street, Malden 48, Mass.
George K. Tozer, 22 Devon Avenue, Beverly, Mass.
Robert E. VanDernoot, 28 Dennison Street, Roxbury 19, Mass.
John F. Vance, 250 Park Street, Attleboro, Mass.
Robert A. Watt, 579 School Street, Belmont 78, Mass.
John J. Walsh, 11 Rockland Place, Malden 48, Mass.
Walter A. Zell, 67 Bloomingdale Street, Chelsea 50, Mass.

DEATHS

EDWIN F. ALLBRIGHT, November 12, 1948

FRANK H. MASON, December 21, 1948

JOSEPH H. O'BRIEN, August, 1947

JOHN SHEA, August 3, 1948

WILLIAM J. SULLIVAN, December 31, 1948

SANFORD E. THOMPSON, February 26, 1949

JOSEPH A. TOSI, October 8, 1948

ANNUAL REPORTS

REPORT OF THE BOARD OF GOVERNMENT FOR YEAR 1948-1949

Boston, Mass., March 16, 1949

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 16, 1949.

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

Honorary	9
Members	711
Associates	4
Juniors	54
Students	55
	833

Reinstatements

Members	3
Juniors	3
Students	5
	11

Summary of Additions

New Members	69
New Associates	1
New Juniors	5
New Students	9
	84

Summary of Transfers

Juniors to Members	3
Students to Members	3
Students to Juniors	25
	31

Summary of Loss of Members

Deaths	15
Resignations	7
Dropped for non-payment of dues	14
Juniors who failed to transfer	5
Students who failed to transfer	6
Total	47
Members exempt from dues	76
Remission of dues	2
Applications pending on March 1, 1949	89

Honorary Membership is as follows:

Dr. Karl T. Compton, elected, February 17, 1932
Prof. Charles M. Allen, elected, January 14, 1942
Arthur W. Dean, elected, January 14, 1942
Charles R. Gow, elected, January 14, 1942
Arthur T. Safford, elected, January 26, 1943
Charles M. Spofford, elected, December 19, 1945

Charles W. Sherman, elected, February 19, 1947
Dugald C. Jackson, elected, March 24, 1948
Edwin S. Webster, elected, March 24, 1948

The following members have been lost through death:

Joseph H. O'Brien, August, 1947
Hartley L. White, September 20, 1947
Mark E. Kelley, February 2, 1948
William M. Smith, April 22, 1948
C. Frank Allen, June 6, 1948
Pere O. Haak, July 12, 1948
Charles D. Kirkpatrick, August 3, 1948
John Shea, August 3, 1948
Frederic W. Bateman, September 2, 1948
Edwin J. Beugler, September 12, 1948
Joseph A. Tosi, October 8, 1948
Edwin F. Allbright, November 12, 1948
Frank H. Mason, December 21, 1948
William J. Sullivan, December 21, 1948
Sanford E. Thompson, February 26, 1949

Meetings of the Society

March 31, 1948—Address of retiring President, Harvey B. Kinnison, "Water Investigations of the U. S. Geological Survey", followed by a dinner.

April 21, 1948—Centennial Anniversary Banquet and Dance at the Hotel Continental, Cambridge, Mass. Centennial Address, "Looking Ahead", by Carl S. Ell.

May 19, 1948—"Construction of the 6th Avenue Subway in New York City, by Charles B. Spencer.

September 22, 1948—"The Proposed Redevelopment of the Water Power of the Connecticut River at Wilder, Vermont", by Edward G. Lee.

October, 1948—No meeting of the B.S.C.E.—American Society of Civil Engineers held their Fall Meeting in Boston and the B.S.C.E. members were invited guests in honor of the 100th Anniversary of the B.S.C.E.

November 17, 1948—Student Night. Joint with American Society of Civil Engineers, Northeastern Section and Structural Section, B.S.C.E. "Historical Development of Subaqueous Tunneling", by Ole Singstad.

December 15, 1948—"The Jordan Marsh Building": "The Architecture of the Building", by William C. Perry; "Structural Features of the Building", by Maurice A. Reidy; "Foundation Problems", by Charles B. Spencer.

January 26, 1949—"Ground Water Resources in Greater Boston", by Henry N. Halberg.

February 16, 1949—Joint Meeting with Structural Section, B.S.C.E. "Architectural Concrete", by Edward B. Oberly.

Attendance at Meetings

DATE	PLACE	DINNER	MEETING
March 31, 1948	Boston City Club	279	125
April 21, 1948	Hotel Continental	102	102
May 19, 1948	American Acad. of Arts and Sciences	*	112
September 22, 1948	American Acad. of Arts and Sciences	*	105
October, 1948	Guests of A.S.C.E. at their Fall Meeting, Hotel Statler		
November 15, 1948	Northeastern University	312	347
December 15, 1948	American Acad. of Arts and Sciences	*	196
January 26, 1949	American Acad. of Arts and Sciences	*	81
February 16, 1949	American Acad. of Arts and Sciences	*	68

*Collation served in Lounge after meeting.

Sections

Thirty-seven 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, 1949, issue of the Journal.

*Funds of the Society**

Permanent Fund. The Permanent Fund of the Society has a present value of about $64,800. The Board of Government authorized the use of as much as necessary of the current income of this fund in payment of current expenses. By vote of the Society (as prescribed by the By-Laws at the December 15, 1948 and January 26, 1949 meetings), the Board of Government was authorized to transfer an amount not to exceed $5,000 from the Principal of the Permanent Fund to cover unusual expenditures of the Centennial Year and to balance the Secretary's Budget at the end of the year. The amount used to balance the budget was $2,754.44.

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. Mr. Ralph S. Archibald, member, was awarded the John R. Freeman Scholarship for 1948-1949, and Col. Carroll T. Newton, member, has been awarded the John R. Freeman Scholarship for the year 1949-1950.

Edmund K. Turner Fund. In 1916 the Society received 1,105 books from

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

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 $50 of the income for the purchase of books, for the library.

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 this fund is "to be devoted to the library of the Society". The Board voted to use $100 of the available income for the purchase of books for the library.

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 $2,592.86 on June 30, 1948. John H. Schmertmann of Scarsdale, New York, a student in Civil Engineering has been awarded this Scholarship of $100 for the year 1948-1949.

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, provides that the income from this fund shall "be used for charitable and educational purposes". The Board voted on January 24, 1949, to appropriate from the income of this fund the sum of $75, to be known as the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald, and to be given to a student of Northeastern University.

Clemens Herschel Fund. This fund was established in 1931, by a bequest from the late Clemens Herschel, a former Past President and Honorary Member of the Society. The income from this fund is to be used for presentation of prizes for papers which have been particularly useful and commendable and worthy of grateful acknowledgment. The expenditure made during the year from this fund was for prizes.

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 for purchasing a cover for the movie screen.

William P. Morse Fund. This fund, a bequest of $2,000, was received April 8, 1948, 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 was 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 from this fund during the year.

Prizes

AWARD	RECIPIENT	PAPER
Desmond FitzGerald Medal	Thomas R. Camp	"The Merrimac River Valley Sewerage District Project"
Clemens Herschel Award	John B. Babcock, 3rd	"Centennial History of the Boston Society of Civil Engineers, 1848-1948"
Structural Section Award	Robert J. Hansen	"Long Duration Impulsive Loading of Simple Beams"

Hydraulics Section Award	George R. Rich	"Basic Hydraulic Transients"
Surveying & Mapping Section Award	Charles M. Anderson	"The Engineering Aspects of the Land Court"

Journal

The complete report of the Editor of the Journal for the calendar year 1948 will be published in the April, 1949, Journal.

Library

The report of the Committee on Library 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. The reports of these committees will be presented at the Annual Meeting.

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

FREDERIC N. WEAVER, *President*

REPORT OF THE TREASURER

Boston, Mass., March 16, 1949

To the Boston Society of Civil Engineers:

The financial standing of the Society on March 1, 1949, at the end of the accounting period, is shown in the following tables:

Table I—Distribution of Funds—Receipts and Expenditures

Table II—Record of Investments

The receipts from dues have increased annually during the past five years and this year amounted to $5,832.50 compared with $5,188.50 paid in as dues during the preceding year. In 1944-45 receipts from dues were $4,317.00.

The observance of the One Hundredth Anniversary of the Society and the activities incident thereto constituted the most important events of the year. Publication of a special issue of the Journal and the Anniversary Dinner and Dance had not been planned sufficiently at the beginning of the year to permit being included in the Secretary's Budget. It was necessary therefore to provide sufficient funds to defray the cost or deficit on these items by the transfer of funds from the Permanent Fund. The total amount of current expenses including the costs incident to the anniversary year was $15,112.29 or about $3,000 more than the previous year.

Operating costs have continued to increase during the year particularly such items as printing which have increased twice since the year began.

The receipts to the Current Fund of $10,223.77 was $4,888.52 less than the Current Fund expense and a transfer equal to the latter amount was made from the Permanent Fund to balance the account.

The Board of Government authorized the transfer of the full amount of the

Permanent Fund income or $2,754.44 to pay current expenses. At the regular meetings of the Society on December 15, 1948 and January 26, 1949, it was voted "to authorize the Board of Government to transfer an amount not to exceed $5,000 from the principal of the Permanent Fund to cover unusual expenditures of the Centennial Year and to balance the Secretary's Budget at the end of the year". The actual amount of transfer required was $2,134.08.

The amounts which have been transferred from the Permanent Fund to the Current Fund during the last five years are shown in the following tabulations:

	1944-45	1945-46	1946-47	1947-48	1948-49
Receipts to Current Fund					
Dues	$4317	$4444	$4851	$5188	$5832
Other than dues	3518	3216	3560	3964	4392
Total Receipts to Current Fund	$7835	$7660	$8411	$9152	$10,224
Current Fund Expenditures	$9239	$8496	$9551	$12,113	$15,112
Deficit: Transferred from Permanent Fund					
Interest	$1404	$836	$1140	$2778	$2754
Principal				183	2134

About two years ago a loan was made from the invested funds of the Society for the purpose of establishing the "Publication Fund". During the past year the receipts from the sale of *Soil Mechanics* volumes was sufficient to permit paying off the balance of the loan. The Board of Government therefore voted to pay the loan and close the account.

There were about the usual number of changes in the securities held by the Society. The sales, purchases, stock dividends received, etc., were as follows:

Sales

1,000 Western Maryland R.R. 4% Bond
25 Shares of Commonwealth & Southern Corp.
15 Shares of Erie Railroad Co. Series A Preferred
15 Shares of Timken Roller Bearing Co.
225 Shares of United States Trust Co. of Boston Corp.

Purchases

1,000 U. S. Savings Bond Series G 2½%
20 Shares of Central Hanover Bank & Trust Co.
7 Shares of Pacific Gas & Electric Co.
10 Shares of Standard Oil of N. J.
15 Shares of Texas Company

Split-ups and Stock Dividends

22 Shares of Union Pacific R.R. were exchanged for 44 shares due to a split-up of the stock.
30 Shares of Union Carbide and Carbon Corp. were exchanged for 90 shares due to split-up of the stock.
1 Share of Texas Co. was received as a stock dividend and also 3 shares of Standard Oil of N. J.

During the past year the Boston Safe Deposit and Trust Company continued to act as Investment Counsel for the Society and Custodian of all securities. All sales and purchases have been made on the recommendation of the Investment Counsel and with the approval of the Board of Government.

The total book value of all securities plus cash on hand now stands at $104,455.82 a decrease of $5,548.70 from the previous year. This reduction is due to a substantial loss on one security that was sold, the withdrawal from the Permanent Fund to balance the Current Fund and withdrawal from the Freeman Fund to make payments to the Freeman Scholar.

The following table shows the comparative book values of the two principal funds at the close of the last five years and the ratio of market value to book value for all investments.

	Mar. 8, 1945	Mar. 8, 1946	Mar. 7, 1947	Mar. 8, 1948	Mar. 1, 1949
Permanent Fund	$63,690	$65,618	$67,228	$67,533	$64,830.01
John R. Freeman Fund	28,497	29,654	30,803	31,960	29,828.52
Market value in percent of book value	101.7%	114%	103.5%	95.5%	99.2%

The growth of the Permanent Fund is affected by the percentage of return on investments and also by the amount it has been necessary to transfer from the income and principal of this fund, in order to meet current expenses. The value of the Freeman Fund depends largely upon the expenditures made from the fund during the year and as stated above the withdrawal has been substantial this year because of a scholarship award. The allocation of book loss due to sale of depreciated securities has further reduced these funds as shown in Table 1.

The following table shows the book value of securities and bank deposits and the total value of all holdings, not including the value of the library and physical property, for the past five years.

	Mar. 8, 1945	Mar. 8, 1946	Mar. 7, 1947	Mar. 8, 1948	Mar. 1, 1949
Bonds	$35,816.10	$34,318.15	$38,340.01	$36,307.51	$36,444.73
Cooperative	11,535.19	12,068.89	12,613.26	13,150.97	11,380.60
Stocks	50,708.44	51,809.15	51,809.15	56,767,69	53,426.70
Cash	2,121.57	5,139.69	1,929.46	3,078.33	3,203.79
	$100,181.30	$103,335.88	$104,691.88	$109,304.50	$104,455.82
Publication Fund			1,700.00	700.00	
			$106,391.88	$110,004.50	$104,455.82

The Treasurer's cash balance of $3,203.79 includes $40 withholding taxes of the Office Secretary for the months of January and February, 1949, which is payable to the Collector of Internal Revenue in April, 1949. The Secretary's "change fund" of $30 is not included in the cash balance shown above.

Respectfully submitted,

HERMAN G. DRESSER, *Treasurer*

TABLE I.—DISTRIBUTION OF FUNDS—RECEIPTS AND EXPENDITURES

	Book Value March 8, 1948	Interest and Dividends		Net Profit or Loss at Sale or Maturity		Transfer of Funds		Book Value March 1, 1949
		Cash	Credit	+	—	Purchased +	Sold —	
	1	2	3	4	5	6	7	8
Bonds	$ 36,307.51	$1,033.61	$120.00	$ 21.44		$1,000.00	$ 982.78	$ 36,444.73
Cooperative Banks	13,150.97	72.50	229.63				2,000.00	11,380.60
Stocks	56,767.69	2,945.39			$2,161.32	3,699.82	7,040.81	53,426.70
Cash Available for Investment	979.99				31.66	723.80		1,703.79
Publication Fund	700.00			125.16			700.00	
Total (except Current Fund)	$107,906.16	$4,051.50	$349.63	$146.60	$2,192.98	$5,423.62	$10,723.59	$102,955.82

Column 1+3+6—7 = 8

Funds	Book Value March 8, 1948	Allocation of income, profit and loss Columns 2 & 3 Income 4.08%	Columns 4 & 5 Net loss	Received	Expended	Book Value March 1, 1949
Permanent Fund	$ 67,532.80	$2,754.44	$1,280.71	$ 712.00	$ 4,888.52*	$ 64,830.01
John R. Freeman Fund	31,959.97	1,303.54	606.11		2,828.88	29,828.52
Edmund K. Turner Fund	1,037.55	42.32	19.68		50.00	1,010.19
Desmond FitzGerald Fund	2,074.87	84.63	39.35		2.00	2,118.15
Alexis H. French Fund	1,096.23	44.71	20.79		67.83	1,052.32
Clemens Herschel Fund	1,203.05	49.07	22.82		173.86	1,055.44
Edward W. Howe Fund	1,001.69	40.85	18.99		6.00	1,017.55
William P. Morse Fund	2,000.00	81.57	37.93			2,043.64
	$107,906.16	$4,401.13	$2,046.38	$ 712.00	$ 8,017.09	$102,955.82
Current Fund Cash	1,500.00	4,888.52*		10,223.77	15,112.29	1,500.00
Publication Fund Cash	598.34			246.82	845.16	
Totals	$110,004.50	$9,289.65	$2,046.38	$11,182.59	$23,974.54	$104,455.82

Secretary's change fund of $30.00 should be added to show total cash.
*Transfer of $2,754.44 from income of Permanent Fund and $2,134.08 from principal of Permanent Fund.

Cash balance March 1, 1949:

Investment Fund	$1,703.79
Current Account	1,500.00
Total	$3,203.79

TABLE II.—RECORD OF INVESTMENTS

	Date of Maturity or Classification	Fixed or Current Interest Rate	During the Year March 8, 1948 to March 1, 1949: Interest Received	Additional Amount Invested	Sold or Matured: Cost	Profit + (or loss —)	March 1, 1949: Par Value	Book Value	Market Value
BONDS									
American Telephone & Telegraph Co.	Dec. 15, 1961	2¾%	27.50	....	....	....	$1,000.00	$1,014.75	$1,020.00
Canadian Pacific R.R.	July 2, 1949	4%	200.00	....	....	....	5,000.00	5,342.50	4,750.00
The Pennsylvania Railroad Company	June 1, 1965	4½%	45.00	....	....	....	1,000.00	1,017.74	1,000.00
Puget Sound Power & Light Co.	Dec. 1, 1972	4¼%	42.50	....	....	....	1,000.00	1,058.44	1,060.00
Southern Pacific Oregon	Mar. 1, 1977	4½%	180.00	....	....	....	4,000.00	4,191.30	3,880.00
Western Maryland R.R. Co.	Oct. 1, 1952	4%	26.11	....	982.78	+21.44	....	....	....
United States Savings Bonds, Series D	Jan. 1, 1950	....	120.00	....	....	....	3,000.00	2,820.00	2,820.00
United States Savings 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 Savings Bonds, Series G	Nov. 1, 1956	2½%	25.00	....	....	....	1,000.00	1,000.00	1,000.00
United States Savings Bonds, Series G	May 1, 1958	2½%	100.00	....	....	....	4,000.00	4,000.00	4,000.00
United States Savings Bonds, Series G	May 1, 1960	2½%	12.50	1,000.00	....	....	1,000.00	1,000.00	1,000.00
Totals			$1,153.61	$1,000.00	$982.78	+$21.44	$36,000.00	$36,444.73	$35,530.00

TABLE 2.—RECORD OF INVESTMENTS (*Continued*)

	Date of Maturity or Classification	Fixed or Current Interest Rate	During the Year March 8, 1948 to March 1, 1949				March 1, 1949		
			Dividends Received	Additional Amount Invested	Sold or Matured Amount Received	Sold or Matured Profit + (or loss —)	Number of Shares	Book Value	Market Value
CO-OPERATIVE BANKS									
Codman Co-Operative Bank	Matured Cert.	2½%	$ 50.00	....	....	....	10	$2,000.00	$2,000.00
Suffolk Co-Operative Federal Savings & Loan Assoc.	Matured Cert.	2¼%	22.50	....	....	....	5	1,000.00	1,000.00
Suffolk Co-Operative Federal Savings & Loan Assoc.	Savings Account	2¼%	229.63	....	$2,000.00	....	..	8,380.60	8,380.60
Totals			$302.13		$2,000.00			$11,380.60	$11,380.60

TABLE 2.—RECORD OF INVESTMENTS (*Continued*)

	Date of Maturity or Classification	Fixed or Current Dividend Rate	During the Year March 8, 1948 to March 1, 1949				March 1, 1949		
			Dividends Received	Additional Amount Invested	Sold or Matured Cost	Sold or Matured Profit + (or loss —)	Number of Shares	Book Value	Market Value
STOCKS									
American Tel. & Tel. Co.	Common	$9.00	$468.00	....	....	....	52	$6,189.04	$7,644.00
Bankers Trust Co., N. Y.	Common	1.80	64.80	....	....	....	36	1,590.00	1,440.00
Central Hanover Bank & Trust Co. of N. Y.	Common	4.00	120.00	....	....	....	30	3,210.00	2,680.00
Central Hanover Bank & Trust Co. of N. Y.	Common	4.00	80.00	1,760.00	....	....	20	1,760.00	1,780.00
Commonwealth & Southern Corp.	Cum. Pfd.	6.00	60.00	....	....	....	8	968.60	784.00
Commonwealth & Southern Corp.	Common	....	....	....	51.29	....	25	....	....
Consolidated Edison (Gas) Co. of N. Y.	Common	1.60	68.00	....	....	+14.63	50	2,556.12	1,100.00
Continental Insurance Co.	Capital	2.00	88.00	....	....	....	40	2,026.21	2,400.00
Erie Railway 5%	Pref.	....	....	....	1,133.05	—371.71	15	....	....
General Electric Co. of N. Y.	Common	2.00	90.00	....	....	....	50	2,341.47	1,800.00
Great Northern Railway	Pref.	4.00	52.50	....	....	....	15	778.67	570.00
Hartford Fire Insurance Co.	Common	2.50	25.00	....	....	....	10	761.25	1,260.00
Minnesota Power & Light Co., Minn.	Pref.	5.00	50.00	....	....	....	10	882.00	960.00
National Dairy Products Corp.	Common	1.80	90.00	....	....	....	50	1,154.74	1,400.00
New England Electric System	Common	.80	102.60	....	....	....	108	1,815.00	864.00
North American Trust Shares	July 15, 1955	21.6¢	294.00	....	....	....	1500	5,342.00	4,500.00
Owens Illinois Glass Co.	Common	3.00	60.00	....	....	....	20	1,187.75	1,040.00

TABLE 2.—RECORD OF INVESTMENTS (*Continued*)

	Date of Maturity or Classification	Fixed or Current Dividend Rate	During the Year March 8, 1948 to March 1, 1949				March 1, 1949		
			Dividends Received	Additional Amount Invested	Sold or Matured Cost	Sold or Matured Profit + (or loss —)	Number of Shares	Book Value	Market Value
STOCKS									
Pacific Gas & Elec. Co.	Cum. 1st Pfd.	$1.50	90.00	....	....	....	60)	....)	2,040.00
Pacific Gas & Elec. Co.	Cum. 1st Pfd.	1.37	27.49	....	....	....	20)	1,922.02)	620.00
Pacific Gas & Elec. Co.	Common	2.00	150.50	175.00	....	....	77	2,133.79	2,387.00
Radio Corp. of America	1st Pfd.	3.50	70.00	....	....	....	20	1,720.75	1,420.00
Southern California Edison Co. Ltd.	Cum. Orig. Pfd.	1.50	60.00	....	....	....	40	1,161.22	1,600.00
Southern California Edison	Common	1.50	30.00	....	....	....	20	539.75	600.00
Southern Railway	Pref.	5.00	75.00	....	....	....	15	1,136.80	795.00
Standard Oil Co. of N. J.	Capital	4.00	101.00	813.95	....	....	52	3,300.62	3,484.00
Standard Oil Co. of N. J.	Capital	....	....	....	....	....	½	....	30.65
Tampa Electric Co.	Common	2.00	60.00	....	....	....	30	1,151.25	780.00
Texas Co.	Common	3.00	128.25	950.87	....	....	51¼	2,916.57	2,613.50
Timken Roller Bearing Co.	Common	....	11.25	....	1,018.97	—340.13	15	....	....
Union Carbide & Carbon	Capital	2.00	165.00	....	....	....	90	2,407.79	3,420.00
Union Pacific Railroad	Common	5.00	264.00	....	....	....	44	2,473.29	3,564.00
United States Trust Co.	Common	....	....	....	4,837.50	—1,464.11	225	....	....
Totals			$2,945.39	$3,699.82	$7,040.81	—$2,161.32		$53,426.70	$53,576.15

REPORT OF THE SECRETARY

Boston, Mass., March 1, 1949

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 16, 1949

	Account Number	Expenditures	Receipts
Office			
Secretary, salary and expense	(1)	$ 240.00	
Stationery, printing and postage	(2)	478.98	
Incidentals and petty cash	(3)	187.40	
Insurance and Treasurer's Bond	(4)	25.20	
Storage Room	(5)	8.00	
Quarters, Rent, Light and Tel.	(7)	1,788.86	$ 600.00
Office-Secy.	(8)	2,187.29	
Auditors and Investment Service	(9)	379.66	
Meetings			
Rent of Halls, etc.	(11)	150.00	
Stationery, printing and postage	(12)	38.75	
Hospitality Committee	(13)	820.85	546.86
Reporting and Stereopticon	(14, 15)	6.00	
Annual Meeting (March 31, 1948)	(16)	1,024.20	695.00
Centennial Banquet (April 21, 1948)		703.35*	388.00
Sections			
Sanitary Section	(21)	19.65	
Structural Section	(22)	20.50	
Transportation Section	(23)	25.10	
Northeastern Univ. Section	(24)	15.00	
Hydraulics Section	(25)	16.92	
Surveying and Mapping Section	(26)	22.92	
Journal			
Editor's Salary and Expense	(31)	324.00	
Printing and Postage	(32)	4,280.75	
Centennial Issue of Journal		913.62*	
Reprints	(33)	60.57	
Advertisements	(34)	22.50	1,164.75
Sale of Journals and Reprints	(33,35)		861.26
Library			
Librarian	(41)	——	
Periodicals	(43)	89.47	
Binding	(44)	108.40	
Fines	(45)		2.04
Badges for Members	(51)	18.90	18.90
Binding Journals for Members	(52)	35.17	30.07
Bank Charges	(53)	——	

*Special appropriations for Centennial Anniversary recognition.

	Account Number	Expenditures	Receipts
Miscellaneous	(54)	390.84	84.39
Engineering Societies Dues	(59)	667.92	
Subsoils Committee Expenses		41.82	
Dues from BSCE Members	(70)		5,832.50
Transfer Income Permanent Fund			2,754.44
Transfer Principal Perm. Fund			2,134.08
		$15,112.29	$15,112.29

Entrance Fees to Permanent Fund $712.00

70 new members; 5 juniors, 9 students; 9 juniors transferred to members; 3 students transferred to members; 25 students transferred to Juniors.

Publication Fund

This fund was authorized by the Board of Government on October 28, 1946, to provide for the cost of printing 1,500 reprints of "Contributions to Soil Mechanics".

During the past year a sufficient number of copies were sold to close out the Publication Fund.

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.

Respectfully submitted,

ROBERT W. MOIR, *Secretary*

REPORT OF THE AUDITING COMMITTEE

Boston, Mass., March 16, 1949

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 report 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.

GEORGE W. COFFIN
CHESTER J. GINDER
Auditing Committee of the Directors of the Boston Society of Civil Engineers

MR. GEORGE W. COFFIN, March 14, 1949
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 period March 8, 1948 to March 1, 1949 and report as follows.

All changes in securities owned were found properly recorded in the accounts. 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.

Cooperative Bank earnings were verified and found correct.

All bills paid have been approved by the President and Secretary. Payment of same was substantiated by comparison with paid checks returned by the bank.

A certified list of securities owned by the Society, and held by the Boston Safe Deposit and Trust Company as custodian, as at March 1st, 1949, was reconciled with the Treasurer's record.

Taxes withheld from employee's wages for the months of January and February amount to $40.00 and will be remitted to the Collector of Internal Revenue in the first quarterly payment, due in April.

In accordance with past practice the Secretary's change fund, $30.00, is not included in the assets reported by the Treasurer.

A verified copy of the Treasurer's Report is attached hereto and summaries of his ledger accounts are shown in detail. I found the records for the fiscal period ended March 1, 1949, in good condition and, in my opinion, they are correct.

Respectfully submitted,

WILLLIAM J. HYDE, *Certified Public Accountant*

REPORT OF THE EDITOR

February 14, 1949

To the Boston Society of Civil Engineers:

The Journal for the calendar year 1948 (volume XXXV) was issued quarterly, in the months of January, April, July and October, as authorized by the Board of Government on December 20, 1935.

During the year 1948 there have been published twenty-three papers presented at meetings of the Society and Sections. The table of Contents and Index for the year are included in the October, 1948, issue.

The four issues of the Journal contained 430 pages of papers, discussions, and the proceedings, 9 pages of Index and 32 pages of advertising, a total of 471 pages. An average of 1,300 copies per issue were printed. The net cost was $2,414.36 as compared with $1,350.07 for the preceding year.

The cost of printing the Journal was as follows:

Expenditures

Composition and printing	$2,747.90	
Cuts	835.78	
Wrapping, mailing and postage	55.24	
Editor	300.00	
Copyright	16.00	
		$3,954.92

Receipts

Receipts from sale of Journal and Reprints	$ 635.56	
Receipts from Advertising	905.00	
		$1,540.56
Net cost of Journal to be paid from Current Fund		$2,414.36

Respectfully submitted,

CHARLES E. KNOX, *Editor*

REPORT OF THE LIBRARY COMMITTEE

Boston, Mass., March 16, 1949

To the Boston Society of Civil Engineers:

The Library Committee submits the following report of its activities for the past year.

Shortly after the beginning of the fiscal year, our part-time librarian, Miss Ethel H. Bailey, accepted a position at M.I.T., and her services to the library of the Society were lost.

Binding of material which is to be a permanent part of the library has continued.

Subscriptions to leading engineering periodicals and publications were continued.

Miscellaneous U. S. Geologic Survey Reports and Bulletins were donated, with the approval of the Board, to the Ground Water Division of the U. S. Geological Survey, Boston, Mass.

Miscellaneous U. S. Coast & Geodetic Survey Reports, and special publications were donated, with the approval of the Board, to the U. S. Coast & Geodetic Survey.

Work was begun on the establishment of a historical collection of early engineering works. Many of the valuable volumes have been removed from the shelves and temporarily set aside pending the completion of the task. The committee recommends that the Board of Government instruct the Section Chairmen to call for volunteers or designate carefully selected members of their respective sections who are familiar with the early and historical works pertaining to their special fields to look over the volumes remaining on the shelves, and to advise the Library Committee as to which of these works should be added to the historical collection. No discards of early editions or out-moded volumes should be attempted until every effort has been made to ferret out other rare and historical works which may have been overlooked.

The Committee voted on February 28, 1949, to recommend that each Section, be instructed by the Board to submit to the Library Committee annually, on or before June 1st, a list of books recommended to be purchased for the library.

Work was begun on the establishment of a central reference collection of Standard Specifications. This collection, when completed, should periodically be supplemented to keep it up-to-date. The specifications acquired during the year are listed elsewhere in this report. Others will be added in the near future.

Attempts to dispose of the remainder of the publications listed for discard in Table V of the 1946-47 committee report and of periodicals listed for discard in Table VII of the same report to educational institutions or other interested parties have met with little success. The committee recommends that the remainder of this material be disposed of as waste paper.

The map file, authorized to be purchased by the 1947-48 Board of Government, and which has been on order since April, 1947, has recently become available and delivery in part has been made. Completion of delivery is promised for this month.

With regard to our membership in the Special Libraries Association the Committee voted, November 22, 1948, that inasmuch as less use is made of the publications of this association than was originally anticipated, and that the subject matter of said publications deal less with Civil Engineering or allied matters than was expected, that in view of the notice of increase in annual membership dues for 1949 to $20.00, that our membership not be renewed.

The following expenditures have been made during the past year:

For subscriptions to periodicals	$ 89.47
For binding	108.40
For new books	122.83
Map file	205.90
	$526.60

117 books were loaned during the year, and $2.04 was collected in fines.

The following books were purchased and added to the library:

A.S.C.E. Index to Transactors (1935-1947)
Hydraulics for Engineers, 3rd Ed.—Angus
Hydraulic Measurements, 2nd Ed.—Addison
Hydraulic Machinery—Beitler and Lindahl
Laboratory Control of Water Purification—Cox
Public Health Engineering, Vol. 1—Phelps
The Middlesex Canal, 1793-1860—Roberts
Applied Mathematics for Engineers and Physicists—Piper
Elements of Railroad Engineering—Raymond, Riggs & Sadler
Airport Planning—Froesch & Prokosch
Airports: Design, Construction and Management—Glidden, Law & Cowles
Transport Facilities, Services and Policies—Johnson
Construction Planning and Plant—Ackerman & Locher
Who's Who in Engineering, 1948 Edition
6 Volumes: International Conference on Soil Mechanics and Foundation Engineering
Specifications and Costs—Seelye
Architectural Specifications—Sleeper

The following books were donated by the authors:

Soil Mechanics in Engineering Practice—Terzaghi
Survival and Retirement of Water Works Facilities—A.W.W.A.

The following volumes of the Standard Specifications of the A.S.T.M. were donated to the E.S.N.E. for use of the members:

Vol. 1a, 1b, 2, 3a, 3b, 1946 edition with 1947 supplement and index.

In addition a complete file of the Guide Specifications issued by the Office of Chief of Engineers, U. S. Army, has been obtained and supplementary sheets are periodically received.

Respectfully submitted,

ROBERT W. MOIR, *Chairman*

REPORT OF THE JOHN R. FREEMAN FUND COMMITTEE

Boston, Mass., March 16, 1949

To the Boston Society of Civil Engineers:

Last April the Committee offered scholarships for a year's research work in hydraulics. These scholarships provided $3,000 for a single man and $3,600 for a married man. Two awards were made. These are the first awarded since 1938. One, of $3,600, plus not over $1,000 for expenses, was given to Mr. Ralph S. Archibald of Melrose who started October 1. His project is Radioactive Tracers in Flow Tests. He is carrying on his work at the Graduate School of Engineering at Harvard. The second one was awarded to Carroll T. Newton, Lieutenant-Colonel, Corps of Engineers, U. S. Army, whose permanent address is also Melrose. He will start July 1 in a year's research on Sediment Scour Transportation and Decomposition. He is planning to do his work at the University of Minnesota. These awards were more fully described in the Society Journal of October 19, 1948.

At the request of the John R. Freeman Fund Committees of the two national societies, the American Society of Mechanical Engineers and the American Society of Civil Engineers, meetings were held in New York to explore the possibilities of a combination of the three funds in awarding scholarships, the purpose being to provide enough money to give a combined scholarship each year and to increase the prestige of the scholarship awards. At a meeting held in New York on January 19 this was discussed and the plan proposed for submission to the various committees and societies was as follows:

Each Society would retain control of its funds and freedom of action in devoting them to a joint scholarship program or to their own program through its committee on the Freeman Fund. The Freeman Committees of the three Societies would act as a Joint Award Committee for the Freeman Fund Scholarship with a chairman chosen annually and applications for the award should be addressed to the chairman of the Joint Award Committee. In general this Joint Committee would attempt to rotate the scholarships for award among the three societies.

The awards already made by your Committee will take up most of the funds available until the spring of 1950 so that it could not enter any such arrangement until then. While your Committee is in favor of closer cooperation with those of the other societies than has existed in the past, it has not yet voted to present this to the Board of Government.

Respectfully submitted,

HOWARD M. TURNER, *Chairman*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SANITARY SECTION

Boston, Mass., March 2, 1949

To the Sanitary Section, Boston Society of Civil Engineers:

During the past year, five meetings have been held as follows:

March 3, 1948. Annual Meeting and election of officers. Dean Gordon M. Fair presented a paper entitled "Europe Revisited". Attendance 110.

June 5, 1948. Annual Inspection Trip, visited Lawrence Water Treatment Plant and the Lawrence Experiment Station. Attendance 16.

October 6, 1948. Mr. Stuart Coburn presented a paper entitled "Headaches from Combined Treatment of Industrial Wastes and Sewage". Prepared discussions were presented by Mr. Joseph A. McCarthy and Prof. Clair N. Sawyer, attendance 54.

October 14, 1948. Joint Meeting of Sanitary Division of the American Society of Civil Engineers and the Sanitary Section. Four papers were presented. Mr. Thomas R. Camp presented a paper on "Pollution Abatement Policy", Mr. E. Sherman Chase presented a paper on "Pollution of the Androscoggin River by Industrial Wastes and Control Measures Thereof", Messrs. William L. Hyland and Murray H. Mellish presented a paper entitled "Utilidors for Water, Sewer, and Other Underground Utilities in Arctic Climates". Mr. Karl R. Kennison presented a paper on "The New Water Supply Tunnel of the Boston Metropolitan District Commission". Attendance 151.

December 1, 1948. A paper was presented by Messrs. E. Sherman Chase and Charles G. Hamman, entitled "The Proposed Blackstone Valley Sewage and Wastes Collection System and Treatment Works". Attendance 96.

Four meetings of the Executive Committee have been held during the year.

The total attendance for the year was 427.

The average attendance was 85.

Respectfully submitted,

FRANK L. HEANRY, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE STRUCTURAL SECTION

Boston, Mass., February 25, 1949

To the Structural Section, Boston Society of Civil Engineers:

During the past year the following meetings were held:

March 10, 1948. Annual Meeting and Election of Officers. Dr. Robert J. Hansen spoke on "Long Duration Impulsive Loading of Simple Beams". Attendance 37.

April 14, 1948. Mr. Oscar H. Horovitz spoke on "Steel Erection, Structural and Reinforcing". Attendance 100.

May 12, 1948. Mr. Frank A. Cundari spoke on "Estimating Building Costs, Development of Unit Prices". Attendance 64.

November 17, 1948. Joint Meeting with Main Society. Mr. Ole Singstad spoke on "Historical Development of Subaqueous Tunneling". Attendance 343. (Student Night)

December 8, 1948. Dr. Charles H. Norris spoke on "Localized Buckling of Structural Members". Attendance 53.

January 12, 1949. Mr. James H. Carr, Jr., spoke on "Wood Trusses". Attendance 52.

February 16, 1949. Joint Meeting with Main Society. Mr. Edward B. Oberly spoke on "Architectural Concrete". Attendance 68.

Total attendance was 717.

Average attendance was 102.

There also were two meetings of the Executive Committee, April 2, 1948 and December 18, 1948.

Respectfully submitted,

ARTHUR E. HARDING, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE TRANSPORTATION SECTION

Boston, Mass., February 25, 1949

To the Transportation Section, Boston Society of Civil Engineers:

The following meetings were held during the past year:

April 28. 1948. Joseph Knoerle presented a paper entitled "Master Highway Plan for the Metropolitan Boston Area". The attendance was 46.

November 23, 1948. Professor Dean A. Fales presented a paper entitled "Style Versus Safety". The attendance was 30.

February 23, 1949. Mr. Russell C. Chase presented a paper entitled "The Practical Aspects of Design and Construction of Asphalt Pavements". The attendance was 30.

The total attendance was 106, and the average attendance was 35.

Respectfully submitted,

WILLIAM L. HYLAND, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE HYDRAULICS SECTION

Boston, Mass., February 10, 1949

To the Hydraulics Section, Boston Society of Civil Engineers:

The following meetings were held during the year:

May 5, 1948. Professor James W. Daily spoke on "Laboratory Investigations of the Mechanism of Cavitation." Attendance 31.

November 3, 1948. Mr. Robert T. Colburn presented a paper entitled "Design Features, Clark Hill Dam and Power Plant." Attendance 42.

February 2, 1949. Annual Meeting and election of officers. Rev. Daniel Linehan, S.J. and Mr. Scott Keith spoke on "Seismic Reconnaissance for Underground Condition." Attendance 107.

The total attendance was 180.

The average attendance was 60.

Respectfully submitted,

GARDNER K. WOOD, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SURVEYING AND MAPPING SECTION

Boston, Mass., January 19, 1949

To the Surveying and Mapping Section,
Boston Society of Civil Engineers:

The following meetings were held during the past year:

February 25, 1948. Joint Meeting—Surveying and Mapping Section with Transportation Section. Speaker was Mr. Elmer C. Houdlette who talked on "The Use of Aerial Topographical Maps in the Preparation of Highway Layouts". Attendance 60.

April 7, 1948. Mr. Frank L. Cheney spoke on "Municipal Engineering Surveys". Attendance 60.

October 27, 1948. Motion Picture entitled "Topographic Mapping by Photogrammatic Methods". Attendance 70.

January 19, 1949. Annual Meeting with the election of officers. Mr. William W. Drummey spoke on "The Relationship Between Architect and Engineer-Surveyor", and Mr. R. Newton Mayall spoke on "The Relationship Between the Landscape Architect and the Engineer-Surveyor". Attendance 60.

The total attendance was 190 exclusive of joint meetings. The average attendance was 63 exclusive of joint meeting.

Respectfully submitted,

HUGH P. DUFFILL, *Clerk*

NORTHEASTERN UNIVERSITY SECTION

Boston, Mass., March 7, 1949

To the Northeastern University Section,
Boston Society of Civil Engineers:

The following meetings were held during the past year:

February 19, 1948. Noontime meeting of Section. The film "Navigation Charts" was shown. Attendance 30.

February 25, 1948. Professor Albert Haertlein of Harvard University, a member of the Massachusetts Board of Registration of Professional Engineers, addressed the group on the topic "The Registration Law." Attendance 125.

March 17, 1948. Professor Boris Bakhmeteff of Columbia University addressed the Northeastern Section of A.S.C.E. on "The National Science Foundation and Research in Post-War America." Attendance 25.

March 25, 1948. The Engineering Societies of New England presented a forum on "Mass Transportation" moderated by Mr. Edward Dana of the Metropolitan Transit Authority. Attendance 27.

April 15, 1948. At an evening meeting of the Section Rev. Daniel J. Linehan, S.J., of the Weston College Seismological Laboratory spoke on "The Engineering Aspects of Seismology." Attendance 140.

May 8, 1948. The New England Conference of Student Chapters, A.S.C.E., was held at Worcester Polytechnic Institute. Northeastern student attendance 12.*

May 15, 1948. The members of the Section assisted the University in the Fiftieth Anniversary celebration.

*Not included in attendance average.

June 1, 1948. At a noontime meeting of the Section, Mr. George Haskins of the Massachusetts Department of Public Works spoke on the topic "The State Engineer." A film was also shown. Attendance 30.

October 12-15, 1948. The Section members attended and aided in the Fall Meeting of A.S.C.E. in Boston, Massachusetts. Average attendance 75.

November 17, 1948. Mr. Ole Singstad, Consulting Engineer from New York, spoke at the Annual Student Night, sponsored by the Boston Society of Civil Engineers and the Northeastern Section, A.S.C.E. The topic: "Historical Development of Subaqueous Tunnneling."' Attendance 40.

December 7, 1948. Professor Hardy Cross, Head of the Department of Civil Engineering, Yale University, spoke before the Northeastern Section, A.S.C.E., on the topic "The Faith of an Engineer." Attendance 40.

December 16, 1948. A field trip was made to C. L. Berger and Sons, manufacturers of surveying instruments. Attendance 23 (Seniors).

December 16, 1948. The announcement of the election of officers for the year 1949 was as follows:

President—Charles F. Quigley
Vice-President—Harold E. Cady
Secretary—Richard D. Raskind
Treasurer—Robert B. Nunley

Mr. William W. Drummey of Drummey and Duffill, Inc., Architects and Engineers, then spoke on the subject "The Relationship of Architect to Engineer." Attendance 30.

January 13, 1949. A smoker was held at the University. Movies including the *World Series of 1948* were shown. Attendance 65.

February 8, 1949. At an evening meeting of the Section, Mr. A. L. Delaney of the Portland Cement Association spoke on "Quality Concrete Through Good Placement Practice." Attendance 28.

The total attendance during the year was 678.

The average attendance during the year was 52.

Respectfully submitted,

ARTHUR L. QUAGLIERI, *Clerk*

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BOSTON SOCIETY OF CIVIL ENGINEERS

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