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JOURNAL OF THE

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MISSOURI RIVER PROJECTS

BY BRIG. GENERAL D. G. SHINGLER*

(Presented at a joint meeting of the Boston Society of Civil Engineers and Hydraulic Section, B.S.C.E., held on February 20, 1952.)

MISSOURI River Projects is a story of man's attempt to control and regulate one of the nation's most wild and destructive rivers. Some time ago I heard the governor of one of our Missouri Basin states describe three great unpredictables in human experience; namely, the outcome of a horse race, the state of a woman's mind, and the course of the Missouri River. The inclusion of the "Big Muddy" in this category has historic justification. However, the Corps of Engineers' experience indicates that the relevance of this remark may be short lived in view of the effectiveness of measures now in progress designed to tame this river.

This traditionally unruly and destructive river drains an area of 529,000 square miles, whose development has been hampered by the scourge of flood and drought. Comprising one-sixth the land area of the United States, the Missouri Basin is today a testing ground for a new frontier of water resources control and conservation. It is the scene of the largest river development program ever undertaken in America—one which is transforming a wild and destructive river system into a controlled resource which will benefit the basin and the nation immeasurably.

The Missouri River has its source in the Rocky Mountains along the eastern slope of the great Continental Divide. From Three Forks, Montana, it courses 2,460 river miles to its junction with the Mississippi 16 miles above St. Louis. It flows through or forms the boundary for seven mid-western states, Montana, North and South

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Dakota, Iowa, Nebraska, Kansas and Missouri. It is 110 miles longer than the Mississippi River from its source in Itasca Park, Minnesota, to the Gulf of Mexico.

Closely linked with the early history and pioneering era of the West, the Missouri was for more than a century the principal artery of travel and transportation into the northern half of the Louisiana Purchase. Up its tortuous, debris-soaked channel moved the early fur traders in French bateaus, canoes, keel boats and later in steamboats.

The possibilities of navigation upon the Missouri were first disclosed by the Lewis and Clark expedition in 1804-5 and 6. Lewis and Clark were the first to fully explore the river from its mouth to its source. The era of steamboating on the Missouri extended from 1819 to 1885 when river transportation was largely supplanted by the faster moving railroods.

One of the dramatic incidents of Missouri River steamboating was the excursion of Capt. Grant Marsh's "Far West" up the Yellowstone River to the Little Big Horn scene of the Custer Battle of 1876. The "Far West" removed 52 wounded soldiers of the 7th Cavalry and brought back to Bismarck, North Dakota, the first news of the disastrous defeat of General Custer's command.

The Missouri and its thirty principal tributaries drain an area comprising all or parts of ten states. This drainage area has a population of 8 million. Producer of about 45 per cent of the nation's corn, 34 per cent of its wheat, one-fifth of its butter, one-sixth of its pork, one-fifth of its beef and one-fourth of its mutton, the Missouri Basin can well lay claim to being the breadbasket of America.

A region of climatic extremes, the basin's topography has a comparably wide variation. It ranges from an elevation of 14,274 feet above sea level on snow-covered Gray's Peak in Colorado, to 329 feet on Annie Kautemann's farm in St. Charles County, Missouri. It encompasses the Great Northern Plains area as well as the eastern slope of the Rockies.

The rainfall range is from 8 inches annually in certain arid sections of the northwest, to 44 inches in the extreme southeast. And the temperature range sometimes spreads from 40 degrees below zero in winter to 110 degrees above in the summer. These extremes of climate and rainfall are an index to some of the basic problems affecting the Missouri Basin and its predominantly agricultural

economy. They produced the prolonged and destructive drought of the 1930s and the costly floods of the 1940s and early 1950s.

Paradoxically, this vast basin frequently experiences too much water in the form of floods in one portion, while undergoing drought and lack of water in another. By the same token there is a wide divergence of interests and conditions affecting the local economy of various segments of the Valley of the Missouri. In the arid and semi-arid northwest the people are primarily interested in water for irrigation and domestic supply. In the Central and Southeastern part the chief concern is prevention of destructive floods.

But the Valley's interest in control and conservation of its water resources is broader than these two basic concepts. It needs more power for agricultural production and industrial expansion; it is interested in development of navigation on the Missouri River; improvement of municipal water supply and sanitation, fish and wildlife conservation and public recreation. Closely related is the valley's concern for acceleration of soil conservation and improved land management.

With this background of physical and economic aspects of the Missouri Basin, I shall now discuss the flood control and river development program authorized by Congress in the 1944 Flood Control Act. This law was based upon the reports and recommendations of two Federal Agencies acting in cooperation with the basin states. One was submitted by the Corps of Engineers, primarily for flood control and the improvement of navigation. The other was prepared by the Bureau of Reclamation of the Department of the Interior, based primarily upon irrigation needs. The Bureau of Reclamation has been in charge of Federal irrigation development in the West since 1902. The Corps has had responsibility for rivers and harbors work on the nation's waterways for more than a century and a half, and for flood control since enactment of the Federal Flood Control Act of 1936.

The authorized Missouri Basin program is in many respects a pioneering project in Federal river basin development in cooperation with basin states. Its basic concept is to harness the Missouri River and its major tributaries by means of a system of more than 100 reservoirs. These impoundments will have a combined storage in excess of 100 million acre-feet. They will be operated to store up the melting snows of the mountains and the flood waters of the Great Plains to serve four basic public needs.

In combination with municipal levees and floodwalls, and agricultural levees along the main river from Sioux City to St. Louis, the reservoirs will provide flood protection for more than five million acres of land and scores of towns and cities. The stored water will make possible the irrigation of over five million acres of land, the production of three million kilowatts of hydroelectric power; will contribute to improvement of navigation, municipal water supply, fish and wildlife conservation and public recreation. Included in the program is the authorized 9-foot navigation and stabilization project on the main river below Sioux City, which not only will provide a stable channel for navigation, but prevent erosion and destruction of hundreds of thousands of acres of rich bottom farm land.

The accomplishment of all the multi-purpose benefits of the basin program will entail a balanced and coordinated use of the river flows, which only adequate reservoir storage can effect. The adequacy of flow has been subject in the past to some discussion. But the most recent and detailed engineering survey conducted by the Missouri Basin Inter-Agency Committee in 1951 officially determined that the controlled flow would fulfill all requirements satisfactorily.

This survey showed that the average annual flow in the Missouri River at Sioux City, Iowa—a control point below the lowest main stem dam—was 24,552,000 acre-feet. Data on which it was based covered the period of known records from 1898 to 1949. In the same report there was included data showing the anticipated requirements of the river development program. It was estimated that 16,530,000 acre-feet would meet the requirements of the program as of 1960, and that the requirements would be 22,746,000 acre-feet by the year 2,000, as irrigation development progresses.

As an average, the flow of the Missouri at Sioux City approximates 33,900 cubic feet per second. This is more than doubled at Hermann, Missouri, near the mouth of the river, where the flow averages approximately 81,700 cubic feet per second, due to the discharge from lower basin tributaries.

It has been just six years since first appropriation of funds for construction of initial projects under this comprehensive plan were made available by Congress. In those six years approximately 57 per cent of the authorized projects of the Corps of Engineers have been completed or placed under construction. Similarly, the works of the Bureau of Reclamation have kept pace, so that today

about 45 per cent of the whole program is in some state of construction or is completed.

At this point it is appropriate to emphasize that the division of the Missouri Basin work falls into two logical categories, in line with the experience and assigned responsibilities of the two major constructing agencies, the Corps and the Bureau. The Corps is constructing those projects designed primarily for flood control and navigation; the Bureau those planned primarily for irrigation development.

Obviously, a basin program of such magnitude requires coordination and cooperation, not only between the constructing agencies and other Federal Departments having related interests, but between the Federal Agencies and the states and local communities. I can assure you that is being accomplished in a very satisfactory manner.

For example, certain multi-purpose reservoirs being built by the Corps of Engineers, both on the Missouri and some tributaries, will provide storage of water for irrigation projects of the Bureau of Reclamation. In such cases we ask for and receive the advice of the Bureau engineers as to how much storage they require for irrigation water. We then design our projects to provide such storage, in addition to primary storage for flood control, navigation, and other beneficial uses.

Conversely, it is desirable in the public interest that some of the reservoirs of the Bureau be designed to include flood control storage, as well as irrigation water. In such cases the Corps of Engineers determines the needed flood control storage and so advises the Bureau. Other Federal Agencies such as the Department of Agriculture, Federal Power Commission, Department of Commerce and Federal Security Agency cooperate in supplying data and information in their respective fields.

Something of the magnitude of the Missouri Basin undertaking may be gained from the size and purposes of the main stream structures which are being built by the Corps. Including already completed Fort Peck Dam in Montana, there are six of these main river dams in the program. They will have a combined storage of approximately 75 million acre-feet, hydro power plants planned for an ultimate total capacity of one and one-half million kilowatts, and will create lakes ranging from 30 to 250 miles in length.

The engineering techniques of massive earthen dam construction are exemplified in the Missouri River dams. The larger of these are about two miles long, and range from 160 to 250 feet in height above stream bed. Concrete outlet works and side chute spillway structures are typical in these projects. As members of the engineering profession you will recognize that the unstable character of foundation conditions on the rivers of the Great Plains area, together with economic factors, dictate the earthen type of dam.

Fort Peck Dam in northeast Montana, figure No. 1, built by the Corps in the 1930s was the first major Missouri River control project. Located 590 miles below Three Forks, Montana, this dam has been in operation for stream regulation in the interests of navigation and flood control since 1939. It is 1868 miles above the mouth of the Missouri. Fort Peck Dam was built by

FIG. 1.—SPILLWAY GATES AND MILE-LONG SPILLWAY CHUTE—FORT PECK DAM.

the hydraulic fill method, whereas the other five main stem dams will be rolled earth fill reservoirs. The dam is 250 feet high and 21,000 feet long, creating a reservoir 180 miles in length with a full pool capacity of 19,412,000 acre-feet. Into the fill went 126 million cubic yards of earth. Its power plant now has an installation of 85,000 kilowatts, with a future potential of an additional 80,000. The Fort Peck project operation is estimated to have prevented approximately 50 million dollars of flood damage in the last 13 years.

Moving downstream 414 river miles from Fort Peck Dam, we come to the Garrison Dam, figure No. 2. This project is now 42 per cent completed and when finished will be one of the largest rolled earth dams in the world. The present construction schedule contemplates completion in 1955, with the first power unit going on the line in the spring of that year. The dam will be 210 feet high, 12,000 feet long and 2,600 feet wide at the base. Huge fleets of earth moving equipment, which have hit a peak of 150,000 cubic yards in 24 hours, will place a total of 70,000,000 cubic yards in the main embankment. Concrete yardage for the outlet works

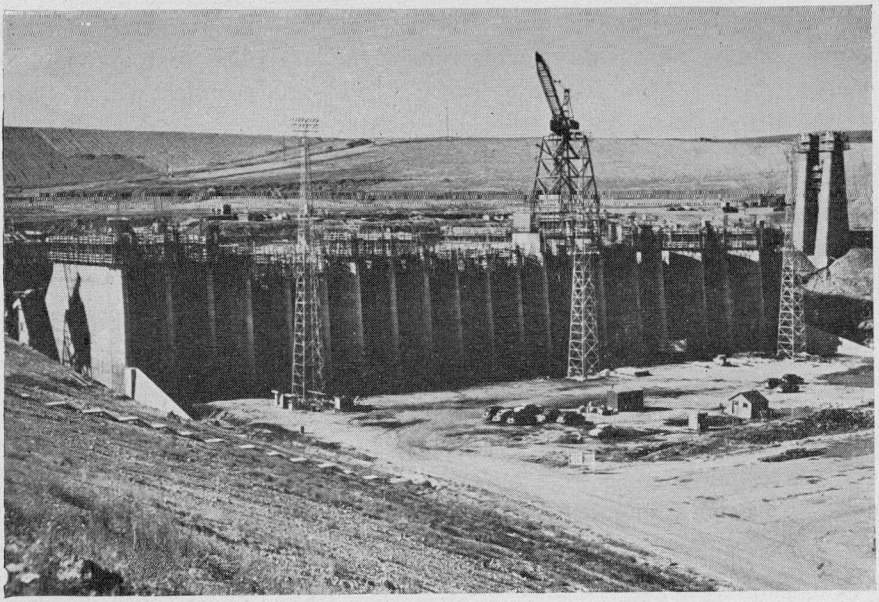

FIG. 2.—INTAKE STRUCTURE—GARRISON DAM.

and spillway will total one and one-half million. The outlet works include eight steel reinforced concrete tunnels, each 1,200 feet long through the right abutment of the dam. Ranging in internal diameter from 22 to 29 feet, five of these tunnels will be for power development and three for flood releases. The tunnels have been completed and the embankment is over 50 per cent placed.

Rolled earth dam construction requires the selection of materials which have been carefully laboratory tested. The earth obtained from selected borrow areas adjacent to the dam site, is placed generally in 8-inch layers at the right moisture content, then compacted with heavy sheepsfoot rollers to 6 inches. The resultant structure has a density as great or greater than natural earth compacted by centuries of time.

Oahe Dam, the third large Missouri River project in our series, is 330 river miles downstream from Garrison Dam. Started in 1948 with construction of access facilities, this project is in the initial work stage. Its general features are similar to those at Garrison with a dam height of 242 feet, a length of 9,300 feet, and reservoir area extending 250 miles upstream. Into the embankment will go 78 million cubic yards of earth, and it will have an ultimate power installation of five 85,000 kilowatt hydroelectric power units. This project will control the drainage area of 243,490 square miles. This compares with 8,257 square miles which comprises the area of your State of Massachusetts.

On some of our large projects earth "wagons" with capacities up to 30 yards are in use, and steam or electric-diesel shovels up to eight yards. This development of large mechanized equipment in the construction industry makes economically feasible the building of huge earthen dams.

The fourth in the series of Missouri River dams is known as the Big Bend project in Central South Dakota. Since it is primarily a power structure and not expected to be built for some years, I shall pass on to the fifth dam in South Central South Dakota. This project, the Fort Randall Dam, takes its name from an old Indian Fort of that name which was located near the damsite, and in which Chief Sitting Bull was once imprisoned. The only landmark remaining of the old fort is the ruins of a stone chapel now standing in a South Dakota cornfield.

Work on the Fort Randall Dam was started in 1946 and is now

FIG. 3.—AERIAL VIEW OF FORT RANDALL DAM UNDER CONSTRUCTION.

nearly half completed, figure No. 3. Initial power is expected to go on the line from this project in November, 1953. The full power installation of 320,000 kilowatts is expected to be installed within the next two years thereafter. Smaller in size than Garrison and Oahe, Fort Randall will be 160 feet high, with an impounding capacity of 6,300,000 acre-feet.

The concrete intake structure is the equivalent in height of a 15-story building. The completed 12 outlet tunnels are 900 feet in length and the power house foundation and stilling basin features are well along toward completion. The main embankment is virtually completed, except for closure of the river section, which is scheduled to be accomplished next July. This multiple-purpose project will contribute substantially to main stream flood control, regulation of channel flow in the interests of navigation, power production and incidental benefits.

Figure No. 4, showing the stilling basin area includes a section of concrete wall which represents an interesting feature of the Fort Randall project from an engineering standpoint. This is the use of what is known as "anchored walls." It consists of a reinforced con-

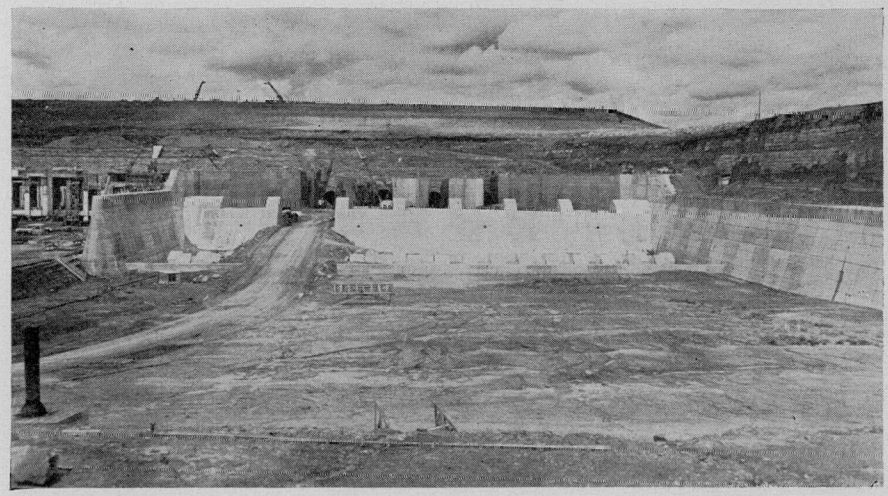

FIG. 4.—FORT RANDALL STILLING BASIN LOOKING UPSTREAM TOWARD A SECTION OF DAM. ANCHORED SLAB WALL IS AT RIGHT.

crete slab cast against a near vertical rock face and made integral with the rock mass by carefully designed anchors. The rock, Niobrara chalk, to which the slabs are anchored, is the bedrock of the area. It is commonly referred to as soft rock, and its high moisture content makes it very susceptible to damage by freezing and thawing. It has a characteristic tendency to resist bond with concrete.

The anchored wall is built as the sidewall for the spillway chute in the stilling basin and discharge channel areas. It has a clear height of 60 feet. The concrete is slightly under 8 feet thick normal to the sloping face and has reinforcing mats in both faces. The bottom of the wall extends 10 feet below the top of the floor slab, a minimum of which is in undisturbed chalk. The slab is anchored to the chalk mass by 2-inch square reinforcing bars of varying lengths, spaced at 8-foot centers each way, and grouted into holes drilled into the chalk. The anchors are secured in the concrete by adequate embedment. A thin concrete slab at the top acts as a roof to discharge the surface runoff from the area behind and over the wall. Approximately 1,180 feet of anchored wall of varying heights has been, or will be built, at the Fort Randall project with an appreciable savings to the Government over the cost of walls of cantilever or gravity design.

Also of interest in connection with the job at Fort Randall is a king-sized floating dredge, possibly the largest of its kind ever constructed. This dredge was designed and built for the Western Contracting Company, one of the project contractors, at a reported cost of one million dollars. It was built in sections which were shipped to the project and there assembled. It has a 36-inch suction and 30-inch discharge, with a large cutter head. Weighing 1,500 tons this floating earth-eater dredged 2,352,000 cubic yards of earth in the approach and outlet channels for the dam in the 1951 working season. The operators reported a 24-hour period in which it dredged over 50,000 cubic yards.

Having dwelt at some length on features of the Fort Randall Dam, I shall discuss only briefly the last in the chain of Missouri River Dams. It is the Gavins Point Dam located near Yankton, South Dakota, on the Nebraska-South Dakota border. First construction work on this project is being initiated this spring. A relatively small dam, it is designed as a reregulating structure to smooth out power surges from the Fort Randall Dam and to permit maximum peaking of power at the Fort Randall power plant. The planned power installation at Gavins Point is 100,000 kilowatts. Its construction is expected to extend into 1956.

Figure No. 5 shows a view of one of the completed flood control projects of the Missouri Basin program. The concrete floodwall shown flanking the Missouri River is part of a 12-mile levee and floodwall system at Omaha, Nebraska, completed by the Corps in 1949. A similar levee project on the other side of the Missouri provides flood protection for the neighboring city of Council Bluffs, Ia. These two municipal protection works prevented damage estimated at $3,000,000 in a high water period in the spring of 1950.

Shifting now from the main stem of the Missouri River into the tributary Kansas River Basin, the Kanopolis flood control dam on the Smoky Hill River in Kansas was completed by the Corps in 1948. It has prevented flood damages estimated at over $6,000,000 downstream since placed in operation. It is one of the projects planned to provide flood protection in the Kansas Basin, which last July was the scene of one of the most destructive floods in the history of the Mid-West. Kanopolis lake provides a recreation for Kansas which has no natural clear water lakes or streams. Last year over 200,000 persons visited the lake to enjoy fishing, boating and other water sports.

FIG. 5.—CONCRETE FLOOD WALL IN OMAHA.

FIG. 6.—AERIAL PHOTOGRAPH OF KANSAS CITY DURING FLOOD OF JULY, 1951.

The Kansas River, which drains approximately 60,000 square miles, flows into the Missouri at twin cities of Kansas City, Kansas, and Kansas City, Missouri. In the flood plain at the junction of the river lies the greatest concentration of business and industry to be found anywhere in the Missouri Basin. Thus the Kansas Cities area presents one of the most serious flood prevention problems in the basin.

Figure No. 6 shows the destructive flooding which occurred at the Kansas Cities in July, 1951. You will recall that last summer's Kansas-Missouri flood caused damage estimated at nearly one billion dollars. Of this amount $461,000,000 was at Kansas Cities, where four great industrial areas were completely flooded. A fifth large Kansas City industrial area was evacuated, as a precautionary measure, but saved from the flood by a new levee and floodwall system completed by the Corps several years ago. The flood forced 17,550 persons from their homes in the Kansas Cities, affected 1,536 businesses and destroyed 298 business buildings.

Our over-all flood protection program for this large metropolitan area includes extensive levees and floodwalls to protect approximately 30 square miles of intensively developed industrial and residential sections of the two cities. This project is about 70 per cent completed, insofar as the local protection works are concerned. However, completed protection of the Kansas Cities is dependent upon completion of a proposed system of reservoirs in the Kansas River Basin which are essential for reducing the stages of the Kansas River. Two million acres of flood lands were flooded last year in the Kansas-Missouri flood.

I have referred previously to the authorized navigation and stabilization program of the Missouri River from Sioux City to the mouth, a distance of 760 river miles. This calls for a 9-foot navigation channel. Many methods are employed to confine the wandering river to a fixed channel and stabilize the banks. These include the installation of permeable pile dikes, which we call training dikes, to bring the river to proper alignment. The silt-laden river then builds up the bank line through accretion behind the dikes, following which bank paving is utilized at critical points.

I should like to revert briefly to the subject of coordination of the Missouri Basin river control program which I mentioned earlier. In addition to the continuing interchange of engineering data between

the Federal agencies and the states and the coordination at that level, we have a specific top-level coordinating body functioning in the basin. I refer to the Missouri Basin Inter-Agency Committee. This committee was established early in 1945 and has continued to function throughout the construction period.

The committee has a membership consisting of one representative of each of six Federal agencies, and the governors of each of the ten basin states. The represented Federal agencies are the Corps of Engineers, Bureau of Reclamation, Department of Agriculture, Department of Commerce, Federal Power Commission and Federal Security Agency. It meets once each month to review the basin program, the progress of construction and to study problems of policy and coordination. As a member of that committee, I have had the privilege of personal experience which convinces me that it is doing an invaluable job of coordinating a tremendous development program. It has instigated and completed studies of the adequacy of the river flow for the purpose of the program; has developed a comprehensive Six-Year program of construction scheduling and is now engaged in making a survey of the power needs and potentials of the basin.

Again I commend to your attention the great progress that has been made in six years on a program of such tremendous scope in a region so large and with such complex interests. The fact that some 45 per cent of the work is under construction or completed is indicative of the wide public interest in the development and the support it is receiving. Many benefits already are being realized from completed projects.

It is our hope and expectation that through this great river development program, the huge undeveloped region of the Missouri Basin can contribute more and more to the nation's essential economy and strength. I believe you will agree that the engineering profession has played and must continue to exert, an important role in the development of the natural resources of our country.

ON AVERAGING RESULTS OF COLIFORM TESTS

By Harold Allen Thomas, Jr., Member*

The fermentation tube test for coliform bacteria since its inception has been a most valuable tool in the quality control of water supplies. By means of the "most probable number" formulation (1, 2, 3), results may be translated to bacterial densities with a sufficient degree of approximation to serve as a useful guide in the day-to-day determination of water quality.

In an important sense, however, the mathematical theory has not been complete. A deficiency limits the utility of the coliform test: No satisfactory method has been evolved for averaging and thereby summarizing the results of a series of tests collected at different times from a station or stations on the supply. Methods that have been used are either (1) mathematically invalid and misleading or (2) valid but difficult to interpret in a meaningful way so as to be of real worth.

All methods based upon the averaging of individual MPN values of single tests fall into one or the other of these two categories and so are not satisfactory, though on the surface they appear to be simple and direct. Another method in common use—that based on the over-all average per cent of positive tubes—gives results that markedly underestimate actual coliform densities.

The value of a good method of averaging is obvious. It could be used in the detection of long-term trends in bacteriological quality resulting from modifications in treatment and from improvement or deterioration of conditions in the watershed or the distribution system. It would facilitate the assessment of the magnitude and significance of seasonal fluctuations in coliform density. Moreover, with appropriate averages, the quality of one supply could be rated accurately against that of others. Thus the method might form the basis of a rational classification of supplies with respect to health hazards from enteric pathogens.

A long-standing source of dissatisfaction with the standard 10-milliliter test stems from its inability to give a precise indication of

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coliform densities of less than about 1 coliform per 100 milliliters. The 100-milliliter test overcomes this objection in part. However, it requires more media and incubator space. As yet it has not come into general use. The situation remains that negative tubes occur so much more frequently than positive tubes, that MPN values recorded usually are zero and at best yield but rough estimates of actual densities. A water plant operator striving to attain a degree of purification beyond that stipulated in the USPHS Drinking Water Standards (4) obtains very little quantitative information from single tests of five 10-milliliter tubes. However, with a valid method of averaging a series of these tests, he could obtain information of much value. This of course derives from the fact that the precision of a mean of a series of measurements increases inversely with the square root of the number of measurements. In good water works practice this number is sufficiently large to offset the inherent lack of precision of the single test.

The purpose herein is to present a valid procedure for averaging the results of coliform tests of the quality of water supplies. The method provides an unbiased estimate of the mean coliform density in a supply during the period covered by the tests. While a facile understanding of the derivation requires some knowledge of mathematical statistics, the method itself is simple. The work involved in the computation is insignificant in comparison wth the labor performed in collection and analysis of bacteriological samples.

It is pertinent to remark that the formulation does not supplant or in any way invalidate the most probable number concept. Rather it is an extension of the underlying principles of probability theory to the problem of averaging a time-series of tests.

Typical Coliform Data. Some insight may be had by considering a specific case.

A clear-well containing a chlorinated plant effluent was sampled twice daily during the period January-December 1946. The 12-hour samples were incubated in five 10-ml lactose tubes, and all tubes showing gas were confirmed before being rated positive. The results at the end of 365 days (730 sets) are shown in Table 1.

The per cent of tubes that were positive was

$$100 \frac{1(38) + 2(22) + 3(16) + 4(14) + 5(20)}{5(730)} = 7.84\%$$

TABLE 1.—DISTRIBUTION OF POSITIVE TUBES IN 730 SETS OF FIVE 10-ml TUBES, JANUARY-DECEMBER 1946

No. of Positive Tubes in Set of Five, x	*No. of Sets Showing x Postive Tubes*	*% of Sets Showing x Postive Tubes, P_x*	*MPN,* Coliform per 100 ml*
0	620	84.93	0
1	38	5.21	2.23
2	22	3.01	5.11
3	16	2.19	9.16
4	14	1.92	16.09
5	20	2.74	∞
–	730	100.00	

*Most probable number = 2.303(100/10)log 1/(1—x/5), bacteria/100 ml.

Those sets showing three or more positive tubes amounted to 6.85% (= 2.19 + 1.92 + 2.74). The data indicate compliance with the first requirement of the standards that no more than 10% of 10-ml tubes be positive. However, the second requirement, that no more than 5% of sets show three or more positive tubes, is not met.

The operator, being aware that high MPN values occurred most often during periods of high runoff from the watershed, decided to alter the schedule of chlorination, but was obliged to operate during the next twelve months with substantially the same amount of chlorine as used in 1946. The results are shown in Table 2.

TABLE 2.—DISTRIBUTION OF POSITIVE TUBES IN 730 SETS OF FIVE 10-ml TUBES, JANUARY-DECEMBER 1947

No. of Postive Tubes in Set of Five, x	*No. of Sets Showing x Positive Tubes*	*% of Sets Showing x Positive Tubes, P_x*	*MPN, Coliform per 100 ml*
0	430	58.90	0
1	218	29.86	2.23
2	54	7.40	5.11
3	25	3.43	9.16
4	2	0.27	16.09
5	1	0.14	∞
–	730	100.00	

With the modification the results show 11.34% of tubes positive and 3.84% of sets with three or more positive tubes. Again the supply failed to meet the standards, but this time it failed the other way: The number of positive tubes during 1947 was 45% larger than during the preceding year. Despite this increase, however, it will be shown that the coliform density in the water going out to consumers in 1947 was substantially smaller than in 1946. With a proper analysis of the data of Tables 1 and 2, the value of the new chlorination schedule is clearly manfest.

Before turning to the theoretical aspects of the problem, it is informative to scan coliform data from twelve communities in four states collected during the three-year period 1947-49. The results were selected arbitrarily from a larger body of data; they are intended to show variety rather than normality. The standard 10-ml test was used in all twelve communities.

TABLE 3.—RESULTS OF COLIFORM TESTS IN TWELVE COMMUNITIES, JANUARY 1947 - DECEMBER 1948

Community	*Population*	*No. of Sets of Five 10-ml Tubes Tested*	*% of Tubes Positive*	*% of Sets with 3 or More Positive Tubes*
1	933	39	22.5	20.5
2	1,102	37	43.2	24.3
3	1,242	29	15.9	13.8
4	1,828	40	8.5	5.0
5	2,606	40	9.0	7.5
6	3,083	36	7.2	2.8
7	7,104	53	7.9	1.9
8	17,020	152	3.1	3.8
9	19,456	213	2.5	1.4
10	44,626	794	0.30	0.13
11	81,113	1,264	2.8	1.0
12	159,896	6,069	0.92	0.01

Evaluation is complicated by the fact that two indications of bacteriological quality are given and these are related to coliform density in essentially different ways.

Incorrect Methods of Averaging. The assumption is sometimes made that an over-all average density can be obtained from the per cent of positive tubes by substituting that figure in the MPN formula. With the data of Table 1, for example, this method yields a value of

2.303 (10) log 1/(1—0.0784) = 0.82 coliform/100 ml. It will be shown that the method is incorrect and almost invariably underestimates the actual density; the error may amount to several hundred per cent. The foregoing formula is valid only for individual tests and can not be applied to a series of samples collected at different times.

Another approach has been to compute the arithmetic mean of the MPN values. However, if some of the sets show 5 positive tubes, this procedure fails, since the MPN for 5 positive tubes is infinity. Values of infinity or "greater than 16.1" cannot be averaged. To ignore such sets is to sacrifice important information. Moreover, even if by chance no completely positive sets occur, the method yields results that are apt to be markedly erroneous.

Theoretically the arithmetic mean of MPN values is always infinity if any coliform at all are present in the supply. Practically, the average MPN will also be infinity if the tests are continued long enough. This follows because the probability is finite for that combination of tubes all showing positive.

Since to many the averaging of MPN values seems intuitively to be the natural and correct procedure, it is worthwhile to elaborate further upon the fallibility of the method. Let it be supposed that the true coliform density in a certain supply remains constant at 1.05 bacteria per 100 ml, corresponding to 10% positive 10-ml tubes (Equation 3). Because of random sampling fluctuations, however, the results from individual sets may vary from 0% to 100% positive. The relative frequencies of the different possibilities when five tubes are tested may be calculated from probability theory (Equation 5). These are shown in Table 4.

TABLE 4.—RELATIVE FREQUENCIES OF VARIOUS RESULTS WHEN SETS OF FIVE 10-ml TUBES ARE TESTED—COLIFORM DENSITY 1.05 PER *ml*

x = *No. of Positive Tubes in Set of Five*	*% Positive Tubes in Set*	*% Frequency of Occurrence, 100Q_x**		*MPN, per 100 ml*
0	0	59.049	(59)	0
1	20	32.805	(33)	2.23
2	40	7.290	(7)	5.11
3	60	0.810	(1)	9.16
4	80	0.045	(0)	16.09
5	100	0.001	(0)	∞

$$*100Q_x = \frac{(100)\ 5!}{x!\ (5-x)!}\ (0.10)^x\ (0.90)^{5-x}$$

The theoretical mean MPN is infinity. $0.59049(0) + 0.32805(2.23 + \ldots + (0.001)(\infty) = \infty$. Even if the last term is ignored the average, 1.19 per 100 ml, is higher than the correct value by about 13%. It is true that with this density the probability that a set will show five positive tubes is almost negligible and actual series are unlikely to contain this result. However, with these series the weighted mean MPN usually differs significantly from the correct value of 1.05 per 100 ml. For example, the most likely outcome for a series of a hundred sets is that shown in Table 4 in parentheses. With this series the method gives a result having a definite bias:

$$(0.59)(0) + (0.33)(2.23) + (0.07)(5.11) + (0.01)(9.16) = 1.19 \text{ per } 100 \text{ ml.}$$

The result is again too high. The error of the method is apt to be much larger than the foregoing when the coliform density fluctuates over a considerable range during the annual cycle, as it usually does. With the 100-ml test the method yields grossly erroneous estimates.

The theoretical geometric mean of MPN values is likewise infinity, and the geometric means of actual series, if not infinity, are usually quite biased. Therefore this parameter, like the arithmetic mean MPN, is not a satisfactory estimator of the true mean coliform density in a water supply.

The median MPN value—the middle MPN value when the series is arrayed in order of magnitude—provides a useful average in multi-dilution fermentation tube tests such as are used in the examination of shellfish and bathing waters. Velz (5) has discussed the application of the median and the use of probability paper in connection with multi-dilution fermentation tube tests. However, in single-dilution tests, common in waterworks practice, the median MPN is of little value. Ordinarily it is zero, as it is for the data of Tables, 1, 2, and 4. In tests for potable water a predominant number of 10- ml tubes are negative, and the 50% quartile almost always falls in this group. In these cases, therefore, it is not a useful parameter. Even if not zero, the median of the standard test must be one of the numbers 1.1, 2.2, 3.7, 5.1, 7.1, 9.2, 12.6, 16.1, and ∞. These numbers are all larger than the upper limit of 1 coliform per 100 ml stipulated in the USPHS Drinking Water Standards. Moreover, they (and the corresponding set for the 100-ml test) are too few and widely spaced to form an adequate scale.

The variation in tests from day to day is of course due to two causes: (1) real changes in coliform density in the supply, and (2) random sampling fluctuations that would occur by chance even though the actual density remained constant. In the development of a rational method of averaging coliform results, it is pertinent to consider the first source of variation from the frequency distribution concept of mathematical statistics.

Duration Curves for Coliform Bacteria. Fluctuations in bac-

TABLE 5.—FREQUENCY DISTRIBUTION OF 1320 37° AGAR PLATE COUNTS FROM A DISTRIBUTION RESERVOIR, 1947-48

Bacterial Density, bacteria/ml (1)	*Observed Frequency (No. of Plates)* (2)	*Cumulative Observed Frequency* (3)	*Cumulative Observed Frequency as a Percentage* (4)	*Cumulative Theoretical Frequency as a Percentage*†* (5)
0-10	603	603	45.7	46.0
10-20	141	744	56.4	56.0
20-30	86	830	62.9	62.5
30-40	61	891	67.5	67.3
40-50	46	937	71.0	71.1
50-60	32	969	73.4	74.3
60-70	35	1004	76.1	76.9
70-80	28	1032	78.2	79.2
80-90	43	1075	81.4	81.2
90-100	21	1096	83.0	82.9
100-120	41	1137	86.1	85.7
120-140	29	1166	88.3	88.0
140-160	19	1185	89.8	89.9
160-180	31	1216	92.1	91.4
180-200	12	1228	93.0	92.6
200-250	29	1257	95.2	95.0
250-300	22	1279	96.9	96.5
300-350	5	1284	97.3	97.6
350-400	9	1293	98.0	98.3
400-500	12	1305	98.9	99.1
500-600	6	1311	99.3	99.5
>600	9	1320	100.0	—

*Per cent of time bacterial density was less than upper end of range of column (1).

†$C \int_0^{\lambda} e^{-a\lambda^p} d\lambda$, with C = 0.07009, a = 0.05477, and p = —0.700.

terial density may be represented by a cumulative frequency diagram analogous to the duration curves of flood frequency used in hydrological investigations. Typical bacteria density distributions are shown in Figure 1. It is convenient, but not essential, to plot such distributions on any type of probability paper that causes the lines to be free from sharp curvature. No particular type of probability paper has been found that will completely linearize bacterial data. However, linearization is not important for the purpose at hand.

Bacteria duration curves based upon plate counts give a more accurate representation of density fluctuations than do those based upon fermentation tube tests. The solid curve (I) of Figure 1 is based upon 1320 37°C agar plate counts of samples collected periodically from a covered distribution reservoir during a two-year period. The data are presented in Table 5. In forming the curve, the numbers in column (1) were plotted against those of column (4).

In the samples collected at regular and frequent intervals at representative stations, the ordinate of the duration curve may be taken to represent the proportion of the water consumed having a concentration less than that given by the abscissa of the curve. The median density and other quantiles may be read directly from the graph.

In this investigation duration curves were used in ascertaining which of a number of different types of theoretical frequency function was most suitable for representing fluctuations in bacterial density. Of the several mathematical functions tested, the "gamma" distribution (Pearson Type III) was found to be the most satisfactory. This distribution has been used extensively in the fitting of hydrological data and other measurements of engineering interest. It is relatively simple and may be fitted to a wide variety of shapes of bacteria duration curves. The gamma distribution has the following equation:

$$f(\lambda) = C\, e^{-a\lambda}\, \lambda^{p} \qquad (1)$$

where $f(\lambda)$ is the frequency corresponding to the bacterial density, λ, organisms per unit volume. The integral $\int_{\lambda_1}^{\lambda_2} f(\lambda)\, d\lambda$ is the proportion of the total water consumed having a bacterial density in the range between λ_1 and λ_2. The parameters a and p reflect the scale and shape of the distribution. The parameter C depends upon a and p: $C = a^{p+1}!$. These quantities are constants for a particular

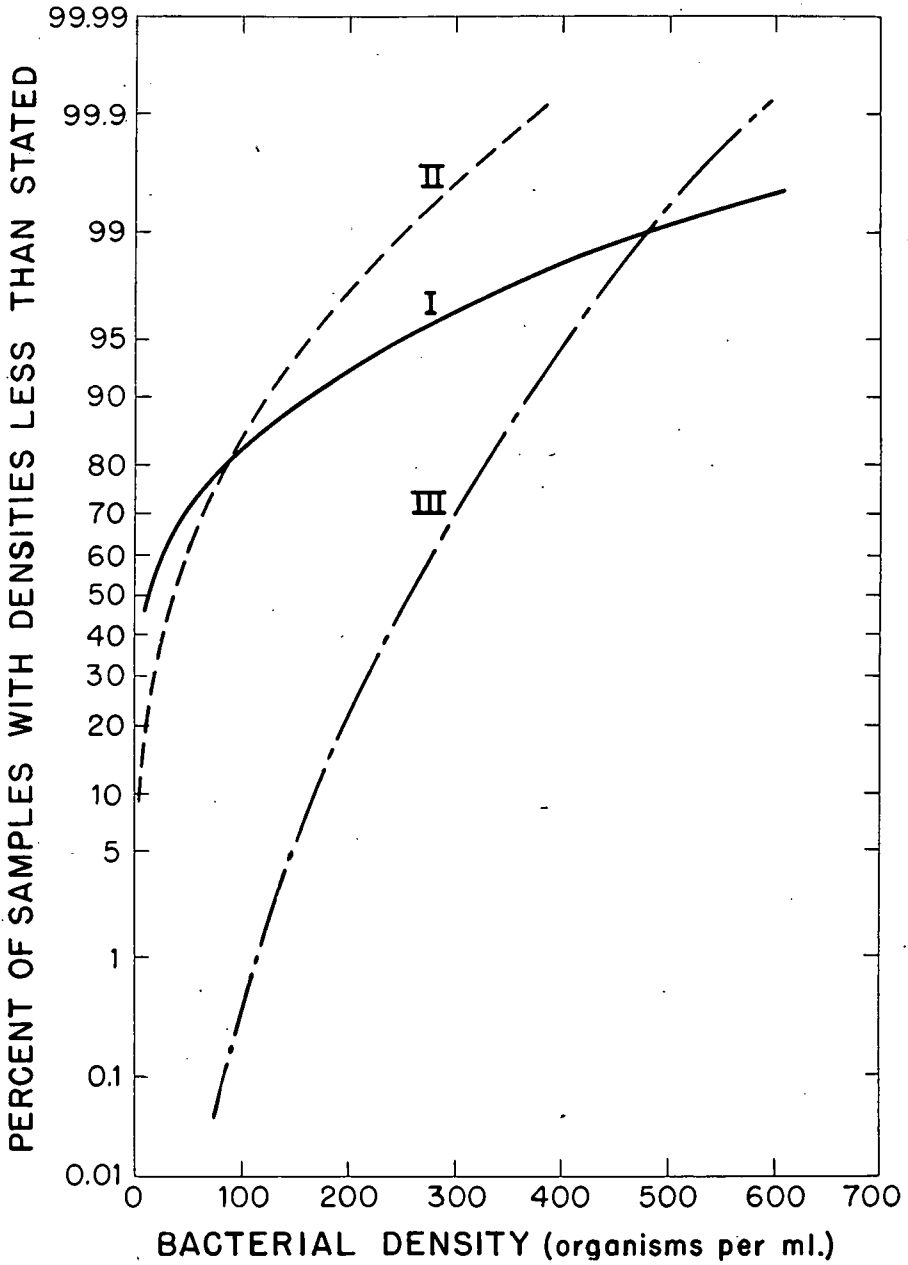

FIG. 1.—TYPICAL FREQUENCY DISTRIBUTIONS OF BACTERIA IN WATER SUPPLIES.

water supply during a particular sampling period; the numerical values will be different in different supplies. Changes in a supply either seasonal or permanent will be reflected by changes in the magnitude of a, p, and C.

An indication of the degree of fit attainable with the gamma distribution may be had by a comparison of columns (4) and (5) of Table 5. The latter column was calculated from the integral of Equation (1) between 0 and λ, taking a = 0.0548, p = —0.700, and C = 0.0711, as calculated by the method of moments. The other curves of Figure 1 are presented to exhibit the flexibility of the gamma distribution in fitting bacterial data of a wide variety of types. The parameters of the curves are listed in Table 6.

TABLE 6.—PARAMETERS OF BACTERIAL DENSITY FREQUENCY CURVES OF FIGURE 1*

Curve	*Parameter*		*Mean Density* $\bar{\lambda}$ *organisms/ml*
	a	*p*	
I	0.0055	—0.700	54.6
II	0.0183	0.000	54.6
III	0.0423	10.2	265

*See Equation (1).

The arithmetic mean of the densities represented by Equation (1) is

$$\bar{\lambda} = \int_0^\infty \lambda f(\lambda) d\lambda = C \int_0^\infty e^{-a\lambda} \lambda^{p+1} d\lambda = (p+1)/a \qquad (2)$$

It is therefore evident that if a and p can be determined from plate count or fermentation tube data, the mean coliform density may be evaluated. The mean densities for the distributions of Figure 1 are given in Table 6.

Formulation for Mean Coliform Density. The analysis of fermentation tube data is facilitated by the introduction of a variable denoting the probability that a tube will be positive. The relation between this variable and the bacterial density is given by the equation

$$\theta = 1 - e^{-\lambda} \qquad (3)$$

Here θ is the probability that a tube will be positive when inocu-

lated with k milliliters of water containing on the average λ organisms. If a very large number of tubes from a body of water having a density of λ were incubated simultaneously, θ would be the proportion of these showing positive. The validity of Equation (3) has been substantiated by many experiments. The most probable number formulation is based upon it.

The frequency distribution of density expressed in terms of θ in place of λ may be derived from Equation (1) using Equation (3):

$$f(\theta) = C(1-\theta)^{a-1} [\log_e 1/(1-\theta)]^{c} \tag{4}$$

Now if five tubes are inoculated simultaneously from a source having a density such that the probability of any individual tube's being positive is θ, then the probability that exactly x tubes of the five will be positive is

$$Q_x = \frac{5!}{x!(5-x)!} \theta^x (1-\theta)^{5x} \tag{5}$$

Equation (5) is based upon the addition and multiplication theorems for compounding probabilities. It was used in computing the frequencies of Table 4.

If sets of five tubes are tested at successive intervals while the density is fluctuating, the proportion of these sets that may be expected to show x positive tubes may be obtained from Equation (5) by weighting θ in accordance with Equation (4):

$$P_x = \int_0^1 Q_x f(\theta) d\theta = \frac{5!}{x!(5-x)!} \int_0^1 \theta^x (1-\theta)^{5-x} f(\theta) d\theta \tag{6}$$

With Equation (6) and the following identities,

$$\theta \equiv \frac{1}{5} \sum_{x=0}^{5} x \frac{5!}{x!(5-x)!} \theta^x (1-\theta)^{5-x} \tag{7}$$

$$\theta^2 \equiv \frac{1}{5(-1)} \sum_{x=0}^{5} x(x-1) \frac{5!}{x!(5-x)!} \theta^x (1-\theta)^{5-x} \tag{8}$$

it is possible to express the first and second moments of the frequency distribution (4) in the following form:

$$M_1 = \int_0^1 \theta f(\theta) d\theta = \frac{1}{5} \sum_{x=0}^{5} xP_x \tag{9}$$

$$M_2 = \int_0^1 \theta^2 f(\theta) d\theta = \frac{1}{(5)(4)} \sum_{x=0}^{5} x(x-1)P_x \tag{10}$$

Equations (9) and (10) may be integrated and written in terms of a and $\bar{\lambda}$ using Equations (2)and (4):

$$M_1 = \frac{1}{5} \sum_{x=o}^{5} x P_x = 1 - \left(\frac{a}{1+a} \right)^{\lambda_a} \tag{11}$$

$$M_2 = \frac{1}{20} \sum_{x=o}^{5} x(x-1)P_x = 1 - 2 \left(\frac{a}{1+a} \right)^{\bar{\lambda}_a} + \left(\frac{a}{2+a} \right)^{\bar{\lambda}_a} \tag{12}$$

Finally, if the observed proportions P_x are introduced in Equations (11) and (12), the value of the mean coliform density, $\bar{\lambda}$ may be computed by solving these equations simultaneously. This is done most readily by the following procedure:

(i) Calculate M_1 and M_2 from the observed proportions, P_x, of tubes showing x = 0, 1, 2, 3, 4, and 5 positives.

(ii) Compute the ratio

$$K = \frac{\log(1 + M_2 - 2M_1)}{\log(1 - M_1)}$$

(iii) Interpolate in Table 7 to find the quantity

$$r = 1/a \log(a + 1)/a$$

(iv) Calculate $\bar{\lambda} = -r \log(1 - M_1)$. Convert density to coliform per 100 milliliters.

Common logarithms are used throughout.

Application. The following examples show computational procedures and typical results.

Example 1. Calculate the mean coliform density for the data of Table 1. The values of P_x from Table 1 are 0.8493, 0.0521, 0.0301, 0.0219, 0.0192, 0.0274.

(i) $$M_1 = \frac{1}{5} \left[0(0.8493) + 1(0.0521) + 2(0.0301) + \ldots + 5(0.0274) \right] = 0.07836$$

(7.836 is the percentage of positive tubes)

$$M_2 = \frac{1}{20} \left[(2)(1)(0.0301) + (3)(2)(0.0219) + (4)(3)(0.0192 + (5)(4)(0.0274) \right] = 0.04849$$

(ii) $$K = \frac{\log[1 + 0.04849 - (2)(0.07836)]}{\log[1 - 0.07836]} = \frac{-0.04974}{-0.03544} = 1.4035$$

(iii) From Table 7, by interpolation, r = 4.9926

(iv) The mean coliform density =

$$\bar{\lambda} = 4.9926(0.03544) = 0.177 \text{ coliform per } 10 \text{ ml}$$
$$= \underline{\underline{1.77}} \text{ coliform per } 100 \text{ ml}$$

Example 2. For the data of Table 2 by a similar computation the following quantities are found: $M_1 = 0.11342$, $M_2 = 0.02068$, $K = 1.9178$, $r = 2.4088$, and finally the mean density, $\bar{\lambda}$, = *1.26* per 100 ml. This $\bar{\lambda}$ value is about 30% smaller than that of Example (1) and points up the value of the new chlorination schedule discussed in connection with the data of Tables 1 and 2.

Example 3. The formulation applied to the data of Table 4 that pertained to the special case of a constant, non-fluctuating, coliform density of 1.05 per 100 ml is of interest, inasmuch as it indicates that the method is unbiased. Here it will be found that $M_1 = 0.1000$, $M_2 = 0.0100$, $K = 2$, $r = 2.3026$, and $\bar{\lambda}$ = *1.05* coliform per 100 ml.

Other computations have been summarized in Table 8 and exhibit the general interrelation between the moments M_1 and M_2 of the density distribution and the mean density.

Table 8 shows that the mean coliform density changes markedly with small variations in M_2. The range of this parameter is M_1^2 to M_1. Only when M_2 happens to fall at the low end of this range will the formula $\bar{\lambda} = 2.303(10) \log(1/(1 - M_1))$ be correct; in all other cases it will underestimate the actual density. If M_2 is large, the error will be gross.

Permissible Limits of Moments of Density Distribution for a Mean Density of 1.0 Coliform per 100 ml. Limiting values of the moments M_1 and M_2 corresponding to a mean coliform density of 1.0 per 100 ml may be obtained from Equations (11) and (12) by setting the value of $\bar{\lambda}$ at 0.10 per 10 ml and solving for M_1 and M_2 simultaneously. Some results of this computation are shown in Table 9.

TABLE 7.—K VERSUS r FOR VARIOUS VALUES OF a

a	K	r	a	K	r
0	1	∞	2.2	1.7257	2.7933
0.001	1.1003	333.29	2.4	1.7402	2.7545
.002	1.1113	185.20	2.6	1.7532	2.7214
.003	1.1190	132.06	2.8	1.7650	2.6929
.004	1.1251	104.18	3.0	1.7757	2.6680
.005	1.1302	86.836	3.2	1.7854	2.6461
.006	1.1347	66.028	3.4	1.7943	2.6267
.008	1.1425	59.513	3.6	1.8025	2.6093
.010	1.1491	49.892	3.8	1.8101	2.5938
.015	1.1627	36.422			
.02	1.1738	29.281	4.0	1.8171	2.5797
.03	1.1919	21.705	4.5	1.8325	2.5499
.04	1.2068	17.668	5.0	1.8455	2.5259
.05	1.2198	15.126	5.5	1.8566	2.5061
.06	1.2314	13.364	6.0	1.8662	2.4895
.07	1.2420	12.063	6.5	1.8746	2.4754
.08	1.2518	11.059	7.0	1.8820	2.4634
.09	1.2610	10.258	7.5	1.8887	2.4529
.10	1.2697	9.6026	8.0	1.8945	2.4437
.12	1.2857	8.5907	8.5	1.8997	2.4355
.14	1.3003	7.8426	9.0	1.9046	2.4283
.16	1.3138	7.2646	9.5	1.9090	2.4218
.18	1.3264	6.8032	10	1.9129	2.4159
.20	1.3383	6.4255	11	1.9199	2.4057
.25	1.3652	5.7227	12	1.9260	2.3901
.30	1.3891	5.2343	13	1.9311	2.3973
.35	1.4106	4.8735	14	1.9354	2.3878
.40	1.4302	4.5950	16	1.9428	2.3738
.45	1.4483	4.3731	18	1.9485	2.3658
.50	1.4650	4.1918	20	1.9535	2.3597
.55	1.4805	4.0407	25	1.9623	2.3483
.60	1.4950	3.9126	30	1.9685	2.3410
.65	1.5086	3.8026	35	1.9726	2.3354
.70	1.5214	3.7071	40	1.9759	2.3312
.75	1.5334	3.6235	45	1.9785	2.3283
.80	1.5448	3.5493	50	1.9806	2.3255
.85	1.5557	3.4832	60	1.9837	2.3217
.90	1.5659	3.4240	70	1.9860	2.3190
.95	1.5757	3.3705	80	1.9877	2.3170
1.00	1.5850	3.3219	90	1.9891	2.3154
1.1	1.6023	3.2372	100	1.9901	2.3141
1.2	1.6182	3.1657	150	1.9933	2.3102
1.3	1.6328	3.1044	200	1.9950	2.3083
1.4	1.6462	3.0514	300	1.9967	2.3064
1.5	1.6587	3.0050	400	1.9975	2.3055
1.6	1.6703	2.9642	500	1.9980	2.3049
1.7	1.6811	2.9278	1,000	1.9990	2.3037
1.8	1.6912	2.8953	10,000	1.9999	2.3027
1.9	1.7006	2.8659	∞	2.0000	2.3026
2.0	1.7095	2.8394			

TABLE 8.—MEAN COLIFORM DENSITIES CORRESPONDING TO VARIOUS VALUES OF THE FIRST TWO MOMENTS OF THE COLIFORM DENSITY DISTRIBUTION: 10-MILLILITER TEST

First Moment, M_1 (1)	*Second Moment,* M_2 (2)	*Mean Coliform Density,* $\bar{\lambda}$, *per 100 ml* (3)
0.10	0.01	1.05
	.02	1.13
	.05	1.63
	.08	7.50
	.10	∞
0.50	0.25	6.93
	.30	8.31
	.35	11.3
	.45	142
	.50	∞
0.90	0.81	23.0
	.82	28.7
	.85	78.1
	.87	441
	.90	∞

(1) From Equation 11 (100 M_1 = % of positive tubes).
(2) From Equation 12.

TABLE 9.—LIMITING VALUES OF MOMENTS OF DENSITY DISTRIBUTION FOR A MEAN DENSITY OF 1.0 COLIFORM PER 100 ml: 10-MILLILITER TEST

M_1	M_2	M_1	M_2
0.00460	0.00392	0.08713	0.01941
.01511	.01182	.09091	.01515
.02369	.01740	.09427	.01046
.05345	.02958	.09471	.00977
.06697	.02989	.09516	.00906

If the value of M_2 exceeds that in Table 9 for an observed per cent of positive tubes, 100 M_1, then the mean coliform density will exceed 1.0 per 100 ml. It will be noted from Table 9 that it is possible for the mean coliform density to exceed 1.0 per 100 ml with percentages of positive tubes much smaller than ten if the value of M_2 is nearly as large as M_1.

95% Quantile of Coliform Density. Useful information relating to the magnitude of the fluctuations of coliform density in a

water supply may be obtained from calculation of various quantiles of the gamma distribution. For instance, it may be shown by a study of the integral of Equation (1) that the 95% quantile of coliform density is given approximately by the following equations:

$$\lambda_{95} = 1.55(\bar{\lambda} + 1/a), \text{ when } \bar{\lambda} a \leqq 5 \tag{13}$$

$$\lambda_{95} = 1.25(\bar{\lambda} + 2.5/a), \text{ when } 5 < \bar{\lambda} a \leqq 15 \tag{14}$$

$$\lambda_{95} = \bar{\lambda} + 1.65 \sqrt{\lambda/a} + 0.426/a, \text{ when } \bar{\lambda} a > 15 \tag{15}$$

Ninety-five per cent of the time the coliform density will be less than the density given by these equations; five per cent of the time this density will be exceeded. The value of a is determined from the computed value of K by interpolation in Table 7.

For the data of Table 1 (Example 1) the value of K is 1.4035 and a is found from Table 7 to be 0.3335; $\bar{\lambda}a = 0.177(0.3335) = 0.059$. Hence Equation (13) gives $\lambda_{95} = 1.55(0.177 + 1/0.3335) = 4.92$ per 10 ml, or *49.2* coliform per 100 ml.

For the data of Table 2 (Example 2), K = 1.9178, a = 10.7, and $\bar{\lambda}a = 1.35$. Hence by Equation (13) $\lambda_{95} = 1.55(0.126 + 1/10.7) = 0.340$ per 10 ml, or 3.40 coliform per 100 ml.

This is a clear indication of the efficacy of the 1947 chlorination schedule in eliminating wide fluctuations in coliform density in the plant effluent.

100-Milliliter Test. The application of the formulation to the 100-ml coliform test will be shown in connection with an analysis of data from a metropolitan area in which 100-ml fermentation tube tests were made during a three-year period on samples collected from the water plant effluent and from the distribution system.

The raw water had a high bacterial density. This circumstance, together with the existence of possible cross-connections with a non-potable supply, made it desirable to attempt to maintain a high chlorine residual in the mains by means of chlorine booster installations on the distribution system. The data are shown in Table 10.

Both groups of samples easily met the requirements of the standards for the 100-ml test that no more than 60 per cent of all tubes show positive and that no more than 20 per cent of the sets contain five positives. Moreover, it appears that the additional chlorination

TABLE 10.—DISTRIBUTION OF POSITIVE TUBES IN SETS OF FIVE 100-ml TUBES: JANUARY 1948 - DECEMBER 1950

No. of Positive Tubes in Set of Five, x	*Water Plant Effluent Samples*		*Distribution System Samples*	
	No. of Sets Showing x Positive Tubes	*% of Sets Showing x Positive Tubes*	*No. of Sets Showing x Positive Tubes*	*% of Sets Showing x Positive Tubes*
0	541	62.04	1598	81.36
1	150	17.20	137	6.98
2	83	9.52	76	3.87
3	51	5.85	55	2.80
4	31	3.56	47	2.39
5	16	1.83	51	2.60
Total:	872	100.00	1964	100.00
% of Tubes Showing Positive:		15.44		9.13

and contact time in the distribution system were effective in improving the quality of the plant effluent.

The analysis by the formulation embodied in Equations (11) to (13) is summarized in Table 11.

TABLE 11.—RESULTS OF ANALYSIS OF DATA OF TABLE 10

	Water Plant Effluent Samples	*Distribution System Samples*
M_1	0.1544	0.09134
M_2	0.06674	0.05260
K	1.6524	1.4550
r	3.0284	4.3004
a	1.4496	0.4701
$\bar{\lambda}$/100 ml	0.220	0.179
λ_{95}/100 ml	1.41	3.57

The computation indicates that the mean coliform density in the distribution system (0.18 per 100 ml) is indeed lower than that in the plant effluent (0.22 per 100 ml). However, the difference is not so large as might be inferred from the percentages of positive tubes. It is pertinent to note from the 95% quantiles that during a significant part of the time the density exceeded the 1.0 coliform per 100 ml stipulated in the standard. Moreover, the quantiles clearly indicate a much wider fluctuation of densities in the distribution sys-

tem than in the plant effluent and confirm the existence of dangerous cross-connections.

Limiting values of the moments corresponding to a mean coliform density of 1.0 per 100 ml for the 100-ml test similar to those for the 10-ml test given in Table 9 are presented in Table 12.

TABLE 12.—LIMITING VALUES OF MOMENTS OF DENSITY DISTRIBUTION FOR A MEAN DENSITY OF 1.0 COLIFORM PER 100 ml: 100-MILLILITER TEST

M_1	M_2	M_1	M_2
0.00689	0.00620	0.5000	0.3333
.02617	.02281	.5981	.3822
.04510	.03855	.6145	.3904
.1412	.1130	.6285	.3977
.2132	.1639	.6303	.3986
.4226	.2925	.6321	.3996

As in the case of the 10-ml test, it is possible for the mean density to exceed 1.0 per 100 ml despite a low percentage of positive tubes if the value of M_2 is high, approaching the magnitude of M_1.

SUMMARY

A method has been presented for computing the mean coliform density from a series of fermentation tube tests on samples collected over a period of time from a water supply system. In addition, the method provides an estimate of the 95% quantile of coliform density during the sampling period. It has been shown that other methods of averaging coliform tests results are subject to large errors.

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THE DELAWARE MEMORIAL BRIDGE SUBSTRUCTURE

By Ralph E. DeSimone*

(Presented at a meeting of the Boston Society of Engineers, held on January 30, 1952).

There is an interesting story to be told of every construction project. One could be told, for example, about each of the jobs we currently have under construction in the Boston area. Among these are the new Mystic Pier, No. 1, the substructure for the new bridge across the Merrimack River between Amesbury and Newburyport, the substructure of the new Meridian Street Bridge across the Chelsea, and the 64-inch water main beneath the Magazine Beach Parkway Section of the Charles.

Tonight, however, we would like to discuss one of our recent projects with which you are perhaps less familiar:—the river piers of the new $43,000,000 Delaware Memorial Bridge just south of Wilmington, Delaware.

Opened for traffic last August 15th, the Delaware Memorial Bridge, Figure No. 1, is the sixth largest suspension span in the world, with a clear central span of 2,150 feet. As you all know, size alone does not measure the challenge of a construction job, however. What made the Delaware Memorial Bridge interesting for us was the fact that construction of its river piers wrapped into one package virtually every problem likely to arise on any type of major bridge project.

Building a bridge toehold on the bottom of any river or body of water is an adventure story unmatched in any other field of construction,—a story that for sheer drama and challenging problems rivals the excitement of any combined sea and land military campaign. Just as in amphibious warfare, the construction assault on a river means that highly trained men, material and equipment have to be carefully marshalled and put into motion under a carefully coordinated plan.

The history of bridge building is in good part a story of bridge foundations. No one knows exactly where and when the first bridge was built for it predates civilization. Even before he learned to

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FIG. 1.—DELAWARE MEMORIAL BRIDGE.

make his first cutting tool, man learned to string vines across a stream or to burn the base of a tree so that, in falling, it straddled both banks. From those early beginnings, construction techniques developed in different patterns throughout the world.

Today's bridges come in all shapes and sizes. All have one thing in common with every bridge ever built, however. Whatever its design or size, each is only as good as its foundation.

Best proof of that are the number of Roman bridges still standing after almost 2,000 years. The thick piers of the Augustus Bridge, built at Rimini, Italy, in 20 B.C., are as strong as ever. The Pont du Gard, a towering triple-tiered span built at Nimes, France, in 14 A.D., to carry the Roman aqueduct 155 feet above the river, is still serviceable. Each stone in its foundations was cut and fitted so exactly that no mortar binding was needed.

The achievements of the Roman era were remarkable for their time, but today's bridge builders are spanning distances and obstacles

never dreamed of before. To keep pace, the contractor must maintain an organization which has the reserve power in men and equipment and financial resources to undertake vast new projects, and to meet any emergency situation which may arise along the way. Large bridge projects should never be tackled by anyone without that background.

In no other field does the contractor face more of a challenge and test of his ingenuity and resources. All construction work is subject to weather hazards, but they multiply when your job is exposed not only to rain, snow and ice, but to tidal changes, shifting currents, fog and all the combinations that go hand in hand with marine work.

Construction projects ashore in almost every case can be fenced in and can be carried out without interruption, or are so isolated as to be exempt from trespass. When building big bridge foundations, however, your work is usually strung across a busy waterway, heavily travelled by shipping vital to the economic life of the entire area. It is your double responsibility not only to schedule operations so as to cause the least interference to that shipping, but to protect your own fleet of marine equipment against collision and other navigational hazards.

The equipment problem is in itself one of the major factors to be kept in mind by the contractor if he hopes to tackle a big bridge foundation project. During the peak of our operations on the Delaware Memorial Bridge, our company had at work on the job a fleet of more than 50 pieces of floating equipment valued at approximately $2,500,000.

No contractor should attempt an equipment investment of that scope unless he has a long history of bridge building behind and ahead of him. There is a little story relative to the Delaware Memorial Bridge to emphasize that point. After we were announced as low bidder for construction of the main piers, one of our competitors in other fields of construction called to wish us luck and to say that they had prepared a bid which would have been just below ours, but had decided at the last moment not to submit it because they would have had to invest well over $1,000,000 in new equipment.

Equipment is only one part of the challenge, however. As bridges grow longer and larger, the contractor finds that each new project increasingly becomes a tougher problem in logistics. The problems

of unification of command ashore and afloat that have made military news in recent years are old ones to the builders of big bridge foundations. Just as in the case of a beachhead invasion, he must plan integration of his work ashore and afloat long before actual construction starts, so that each part of the jigsaw operation falls into place; so that material, manpower and equipment are on hand where and when needed.

Problems seldom run to pattern. No matter how similar the design, every bridge is different when it comes to translating the blueprints of its foundations into actual concrete and steel. Methods that prove perfect on one project would fail on another. With all respect for their thoroughness, blueprints don't tell you the hundred and one things about the river or other body of water on which you will be working:—problems that you have to learn the hard way. Every current, every foot of bottom must be studied and triple checked before a decision is reached on how each job is to be handled.

On that count alone, the contractor must have a good engineering and design force to work in cooperation with the consulting engineers in charge of the project. Because he is the first on the job, he must also be prepared to construct from scratch a complete shoreside base of operations. In many cases that means building special rail terminal facilities even before the most preliminary work can be started. And, once having started the project, he must be equipped with a headquarters management which, regardless of location, can integrate every phase of the work into one smoothly flowing operation.

The Delaware Memorial Bridge project offers perhaps the most dramatic example of some of the above problems.

Including approaches, the four-lane Delaware Memorial Bridge is approximately 3 1/2 miles long, and is 10,765 feet long from abutment to abutment. A heavily travelled coastal cut-off route between the southern terminus of New Jersey's new Turnpike and Delaware's duPont Highway, it features a 2,150-foot main suspended span, providing 175 feet of vertical clearance at mean high water, and two suspended side spans of 750 feet each which provide 165 feet of vertical clearance.

Consulting engineers on the project for the Delaware State Highway Department were Howard, Needles, Tammen & Bergendoff, with Homer Seeley as project manager and O. H. Ammann and Moran, Proctor, Freeman & Mueser as consultants. A. Gordon Lorimer was consulting architect.

Our part in the project entailed the construction of all ten river piers, including the two anchorages; two tower piers and six river approach piers, four on the west side of the river and two on the east. Started in September of 1948, this work was completed in June 1950.

The bridge's superstructure, with a total estimated weight of more than 73,000,000 pounds, was fabricated and erected by the American Bridge Company. The two towers, including 417 feet of steelwork above the foundations, rise to a total height of 440 feet above mean low water. Top elevation of the two anchorages are 162 feet above mean low water.

Work on each pier naturally posed its own special problems, but the most challenging ones centered on the two anchorages, for each involved problems of record proportions. Construction of the West Anchorage Pier required the floating and sinking into position of a 60-well caisson which—approximately 95 feet wide by 221 feet long—was the largest in surface area ever built. As you will see later, a special mooring system had to be devised to keep it in position while it was being sunk broadside to the tidal currents.

The original foundation designs also called for the East Anchorage Pier to be constructed by caisson method. Since steel sheet piling was immediately available and since river conditions permitted the use of either method, it was decided after consultation with the consulting engineers to build it by cofferdam. The big test came when it was time to pour the concrete seal for the foundation slab. All told, it measured 99 feet wide by 225 feet long and 32 feet deep. To build it, we had to pour almost 27,000 cubic yards of concrete non-stop for 7.4 days, which marked a new record for underwater concrete by tremie method.

One of our first requirements was to establish a base of operations as close to the site as possible. For the purpose, we picked Pigeon Point, a mile north of the bridge line, on the Delaware side of the river. A completely equipped Merritt-Chapman & Scott floating field office was the first piece of equipment sent to the job. Self-contained aboard a 170-foot barge, it puts our field force into business the minute it ties up at the site. Office space and equipment are provided for supervisory, accounting, payroll and purchasing departments. There are two complete drafting rooms and a completely equipped field hospital aboard, as well as living quarters for the project manager.

Telephone switchboards and power systems hook into the nearest outlets. Most important it carries the control center for our own two-way short wave radio network which permits the project manager and his assistants to keep in constant communication with every key piece of floating equipment on the job. This radio net was vital on the Delaware when you stop to consider that our work was spread along a mile and a quarter of wide, navigable river, and was not connected to either shore. To reach about one half of our work, it was necessary to cross a navigation channel through which passed all of the water borne traffic entering and leaving the Port of Philadelphia and other ports and locations north of the bridge.

Next came the job of setting up an operating base at Pigeon Point that could serve both as service and supply station for the fleet of more than 50 pieces of M-C&S equipment which subsequently converged on the job, and as a transfer point for the tremendous amount of material which would be shipped to the project site by rail and highway.

Among other things, a new 600-foot pier was built and equipped with two railroad tracks that tied in with the Pigeon Point terminal of the Reading Railroad. Floating machine, metal working and carpentry shops were brought to the site. Pierside storage facilities were put up to handle bulk shipments of materials, which necessarily had to be carefully scheduled. Conveyor systems were erected on the pier to transship directly to covered barges the bulk cement deliveries which, during peak operational periods, were needed at the rate of 20 carloads per day. Fresh water also was a problem, with a total of 1,260,000 gallons required for the East Anchorage tremie seal placement alone, and special well and piping facilities had to be installed in advance of the work.

All six river approach piers were built in sheet pile cofferdams, and the two tower piers by open-well caisson method. Their problems were comparatively routine, however, when matched against those involved in construction of the two anchorages.

The size of the East Anchorage Pier, 225 feet by 99 feet, involved unique problems in construction of its cofferdam. After some preliminary dredging at the location, the bracing was set first and the steel sheet piling was driven around it. The bracing was built in four approximately equal sections. Of all welded construction, each was 49 feet high and featured four sets of cross braced 14-inch

horizontal walers designed to provide 18 foot square clear dredging spaces throughout the cofferdam.

While the first section was being fabricated ashore, the cofferdam site was dredged to —42. On completion, the section was brought into position by barge and lowered into position by three floating whirley derricks. Like the other three that followed, the section weighed approximately 140 tons. To save shoreside space for material storage, the three other sections were partially fabricated ashore and then completed aboard barges. Each was positioned on an alignment marked by six spud piles, which also served as suspension supports. Once in place they were bolted to their neighbors by divers working under water.

Hundred-foot lengths of MZ-38 piling were progressively driven around the perimeter of the cofferdam as the bracing sections went into place. All setting was done by one floating crane fitted with a 165-foot boom. A total of 445 sheets of piling were used, driven home by four floating pile drivers fitted with heavy double acting hammers.

With the cofferdam in place, three revolving cranes rigged with two cubic yard hard digging buckets were brought into play to dredge 28 feet of river bed remaining inside the cofferdam down to elevation —68.5. The bottom of the Delaware on the Jersey side was mostly sand. Material close to the bracing which could not be removed by clamshell was pumped to the surface by air lift after being broken down by high pressure water jets. With dredging completed, the stage was set for what was probably the most critical operation of the entire project:—uninterrupted placement of the almost 27,000 yards of concrete required for the slab of concrete 225 feet long, 99 feet wide and 32 feet deep that forms the tremie seal of the East Anchorage.

A few of the figures involved in "feeding" the two floating concrete plants which successfully carried through the operation in 177 1/2 hours of non-stop work will give you a rough idea of the supply problems that had to be met. From the time the tremie operation started at 8:30 on the morning of August 15, 1949, Figure No. 2, until its completion at 6 P.M., August 22, almost 7 1/2 days later, approximately 46,000 barrels of cement were eaten up by the mix plants, at the rate of some 6,200 barrels per day.

Storage facilities made it impossible to have at hand at the start

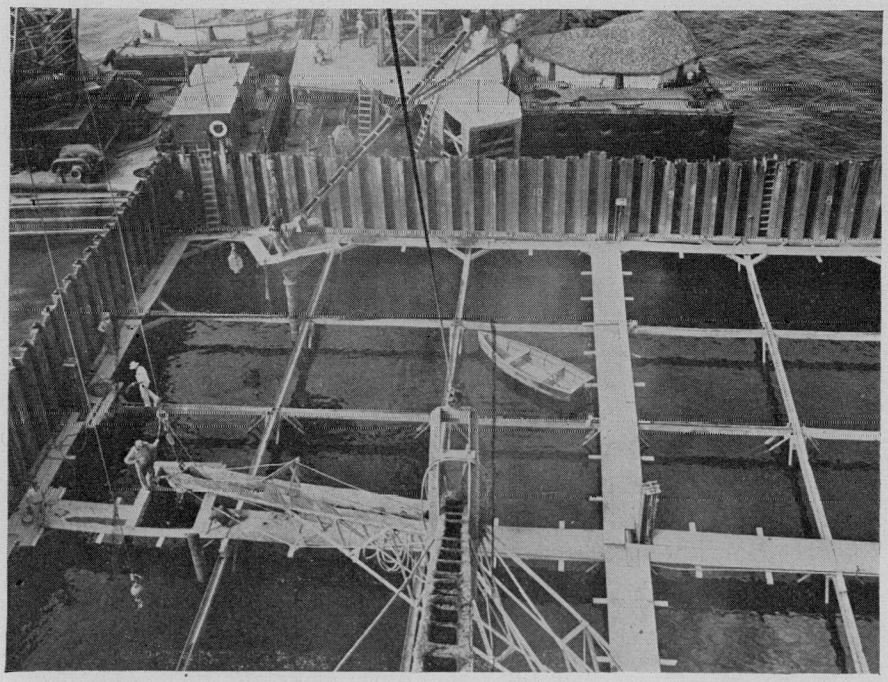

FIG. 2.—EAST ANCHORAGE PIER COFFER DAM DURING PLACEMENT OF TREMIE SEAL.

more than 16 of the 100 railroad cars of cement required. Minutely coordinated schedules had to be taped down to insure that the balance arrived at the rate of approximately 19 carloads per day for delivery to the covered 600-ton barges which carried the bulk cement to the pier site a mile away. Excellent cooperation by the railroads and by the cement companies helped a great deal to keep the work moving along on schedule.

About 45,000 tons of aggregate, roughly 45 scowloads in all, similarly had to be timed for delivery at the site from a commercial supply point more than 30 miles upriver. To insure maximum recovery of sand and gravel and to speed operations, a small bulldozer was swung aboard each aggregate barge while it was feeding the mix plants.

Our two mix plants, which produced a combined average of 152 cubic yards of concrete per hour throughout the operation, worked on opposite sides of the cofferdam with floating cranes periodically shifting hoppers and tremie pipes to insure level placement. The 32-foot thick seal was placed in four passes of eight feet each.

Mounted on a 50 by 135 foot steel hull, our mix plant No. 1 carries a 50-ton revolving crane aft for loading a three-compartment batching bin which holds 100 tons each of sand and gravel and 250 barrels of cement. Sand and gravel barges are worked from either side of the plant. Cement barges moor aft and from there, 5-inch Fuller Kenyon pneumatic pumps send the concrete by pipe into the batching bin. The aggregate bin discharges by direct drop into two 2-cubic yard mixers. An almost continuous flow of concrete can be maintained by charging one mixer while the other is discharging, with operations synchronized to the mixing cycle. Both mixers discharge directly into a 2-cubic yard bucket which, raised in a 90-foot steel hoist tower, is automatically tripped at the level of adjustable chutes leading to the tremie hopper.

Water and fuel barges similarly had to be kept shuttling back and forth between pipe terminals ashore and the floating mix plants. The 117 1/2 hour operation called into play virtually the entire fleet of more than 50 pieces of M-C&S floating equipment, with coordination maintained throughout via directions over the two-way radio central on our floating headquarters at Pigeon Point. Use of our floating power and light plant, mounting a bank of twenty 1,500-watt projector type floodlights atop a 50 foot tower permitted night operations matching daytime speed. Almost 400 men were directly employed on the tremie placement, many of them putting in long overtime because of the shortage of certain specialist workers required.

Seven days later, after the seal had set, the cofferdam was dewatered to the top of the tremie seal at —36.6. As planned, the lowest wales and shores of the bracing were firmly embedded in the concrete. To minimize internal stresses in the anchorage, the exposed upper part of the bracing was cut free before concreting operations were continued "in the dry." After laitance had been removed, sixty 15-foot diameter steel plate well forms were set atop the seal, and all space between them was interlaced vertically and horizontally with one-inch reinforcing bars. Concreting was then carried up in five-foot lifts, using prefabricated steel panels as forms for the exterior face. A heavily reinforced 10-foot thick distribution block was poured at plus five, where the well forms were capped. A granite facing was installed from minus five to plus 15, the blocks serving in lieu of concrete forms. This granite was furnished by the H. E. Fletcher Company and was cut at their West Chelmsford quarries.

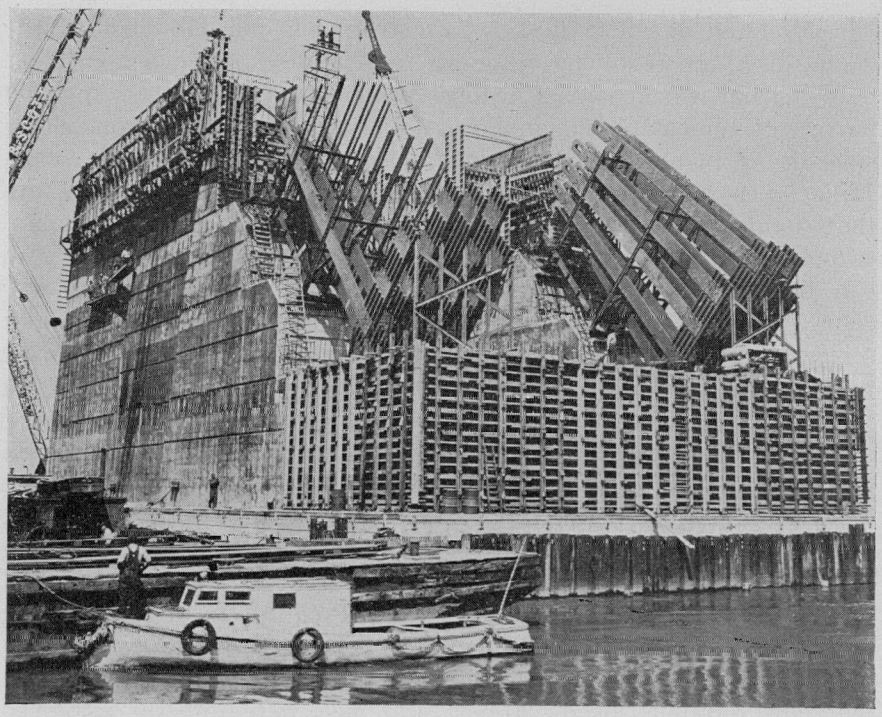

FIG. 3.—EAST ANCHORAGE PIER DURING CONSTRUCTION, SHOWING ANCHOR BEAMS TO HOLD 20-INCH DIAMETER SUSPENSION CABLES.

Successive concreting lifts, Figure No. 3, which raised the anchorage to approximately 162 feet above mean low water were carried out without undue difficulty, thanks again to our floating mix plants and the tremendous reach of the long derrick booms which placed the concrete in the top forms by mechanically activated bottom dump buckets. Divers meanwhile burned off the cofferdam sheet piling just above the top of the tremie seal. The balance was left embedded in the river as a permanent sheathing to protect the pier against scour.

The caisson method used on the Delaware side of the river for the West Anchorage Pier entailed a completely different set of problems.

Built on shipways at the Camden, N. J., yard of the New York Shipbuilding Corporation, the cutting edge section of the caisson was 13 feet high. Outside box-girder walls were four feet thick, and insider stiffening box-girder diaphragms were two feet, nine inches,

thick. Launched shipwise, it was towed to a pier at Philadelphia, where its sides were built up 34 1/2 feet before it was towed by tug to the bridge site.

Since this record sized caisson had to be positioned broadside against strong tidal currents, four 30-foot diameter sand islands were built in advance to serve as a corral while it was being sunk. Two islands were placed upstream, two downstream, and two steel-pile dolphins were driven between them on the shore side. After the caisson was towed into place at slack tide, another pair of dolphins were driven on the river side, penning the caisson into approximate position. A fender system was then built to allow four feet leeway for final positioning as the caisson was sunk.

Plans called for the West Anchorage caisson to be founded at —96. The river bottom at the site was approximately —20 and the final bottom elevation of the caisson was —92.7. This meant that it was sunk through approximately 73 feet of river bed, composed mostly of hard clay on that side of the river. The caisson was positioned with extreme accuracy, the final deviation in line from the theoretical location being less than one inch.

In view of its large surface area, special precautions had to be maintained throughout the sinking operation to insure exact balance as the caisson, with four derricks working around its perimeter, Figure No. 4, was driven down by clamshelling through its sixty 15-foot circular wells and by concreting between them. Prefabricated well risers and steel walling for the watertight cofferdam section were added to the caisson as it sank. When finally founded, its over-all height was approximately 106 feet.

Sharp-toothed heavy duty digging buckets were required while dredging through the hard clay of the river bottom, with high pressure rotary jets serving to loosen the clay between the wells. The jets were fabricated from 12-inch vertical pipe leading to twin four-inch horizontal distributor pipes fitted with one and a half inch nozzles. This arrangement permitted a water pressure of 350 pounds per square inch. The jet was kept centered in the well by a series of horizontal struts attached to the vertical 12-inch pipe. Mushroom-shaped weights were periodically lowered into each well to provide soundings used to keep the bottom uniformly levelled.

Following a final inspection at founding depth, a tremie seal was poured inside the cutting edge chamber and inside the wells to a

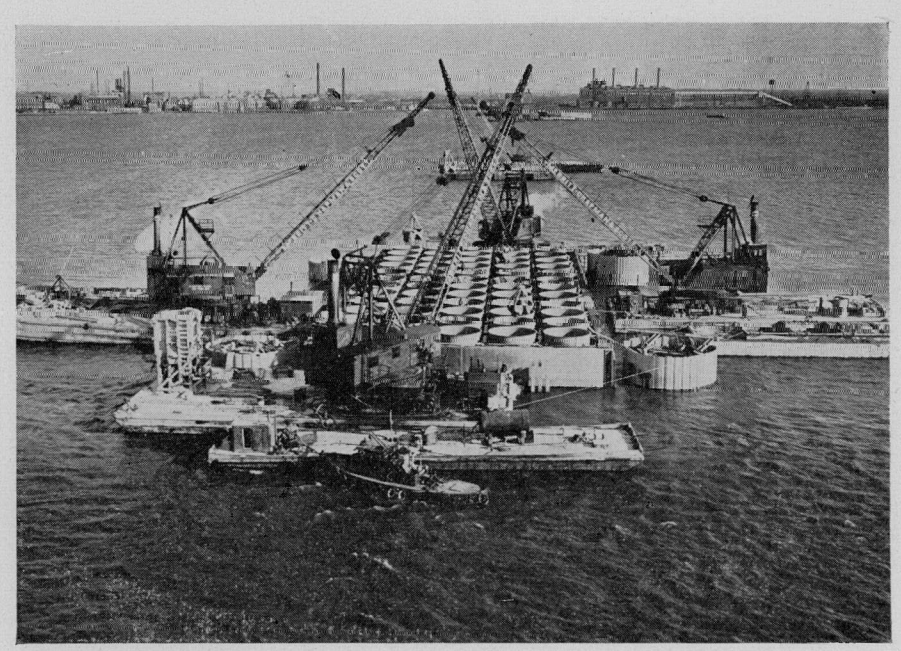

FIG. 4.—FOUR FLOATING DERRICKS DREDGE SIMULTANEOUSLY THROUGH 60 WELLS OF WEST ANCHORAGE CAISSON.

point 15 feet above the cutting edge. A 10-foot thick reinforced concrete distribution block was placed over the West Anchorage Pier at plus 5 and from this elevation it was carried up in successive lifts to approximately 162 feet.

On a smaller scale, caissons for the two tower piers were the same design and construction as that of the West Anchorage. Each was 69 by 116 feet in surface area, with twenty-eight 13 foot, two inch diameter wells set in four rows of seven each. In their case, two 70-foot diameter steel sheet pile sand islands, one upstream and one downstream, were used for mooring positions. The East Tower caisson was founded at approximately —118 and the West Tower caisson at approximately —89.

After placement of the tremie seals, dredging wells of the tower piers were dewatered to a point where their two outer rows of wells could be capped at —30 to create step-backs. All well caps were fabricated at a casting yard ashore. The two inner rows were then carried up to plus five and topped by a 15-foot reinforced distribution

block 45 feet wide by 116 feet long. Eight buttresses on each side were based at —30, the elevation at which the outer rows had been capped.

A comprehension by the engineers of the contractor's problems and their intelligent assistance in helping solve these problems go a long way in producing the net result which they both are seeking, that is, a better job for the client they both serve. The better the cooperation, the better the job.

Responsible contractors have a fierce pride in their work and in building a reputation for jobs well done. We measure a project not only in terms of earnings but in the satisfaction of those for whom it is built.

"INFORMATION PLEASE" ON SLUDGE DIGESTION

A PANEL discussion sponsored by the Sanitary Section, Boston Society of Civil Engineers, held at the rooms of the Society, 715 Tremont Building, Boston, Massachusetts, on Wednesday, March 7, 1951, beginning at 7:30 P.M., the panel comprising:

Mr. Joseph McCarthy of the Lawrence Experiment Station, Moderator.
Dr. Rolf Eliassen, Professor of Sanitary Engineering, M. I. T.
Mr. John D. McDonald, Supt. of Sewage Treatment, Springfield, Mass.
Prof. Edward W. Moore, Division of Applied Science, Harvard.
Mr. R. S. Rankin, The Dorr Company, Stamford, Conn.
Mr. L. E. Langdon, Pacific Flush Tank Company, Chicago, Illinois.
Mr. Frank L. Heaney, Chairman of Sanitary Section, presiding.

CHAIRMAN: This is rather an unusual type of meeting. I believe it is the first of this type to be held by the Sanitary Section. The panel members have not been advised as to the questions to be asked and there have not been any form or prepared statements or papers. Introduction of panel members and moderator.

MODERATOR: The subject to be discussed is Sludge Digestion and I hope you will allow a reasonable amount of latitude in this discussion without the impedence of tangent discussions. We will now open the discussion.

FIXED VS. FLOATING COVERS

MR. THOMAS A. BERRIGAN: What are the relative merits of floating and fixed covers in a multi-stage digester arrangement?

MR. RANKIN: Either type of cover may be used on either the primary or secondary tanks with equal results. The arrangement varies with the particular plant and is largely a construction and economic decision. The question of locating the gas storage should also be considered as an engineering decision for any design performance.

MR. LANGDON: A floating cover allows greater ease in operation due to the variability in digester volume, as governed by the operator. However, the selection and location of either type of cover is an economical decision. A fixed cover, for safety reasons, requires an

adequate volume of gas storage so that the too rapid withdrawal of sludge will not draw in air (causing an explosive mixture), but rather the gas from this reservoir will be siphoned back.

PROF. ELIASSEN: The factor of hydraulic mixing within the digester must also be considered. In a small plant, adequate mixing is insured by the action of sludge withdrawal, sludge addition and recirculation pumps regardless of the type of cover employed.

However, in a large tank, i.e. 100′ diameter and 25′ height, adequate hydraulic mixing must be insured by propeller mixing. Since there is a certain ratio of propeller submergence to propeller diameter, speed, etc., the rigidity afforded by a fixed cover is desired. However, flexibility in gas production must be provided by a separate gas storage sphere if the primary tank is to have a fixed cover.

MR. LANGDON: In answer to hydraulic mixing, the units employed at Cleveland, Ohio, have been in operation since 1932 with the absence of external mixing. A homogeneous mixture has been insured by normal tank gasification.

P. F. T. SCUM BLANKET TREATMENT

MR. FRANK FLOOD: How has the P. F. T. type scum control device worked out in practice in contrast to the propeller mixing which also serves to pull the scum down from above?

MR. LANGDON: Good success has been encountered in the case of scum consisting of digestibles, but scum consisting of undigestibles has presented a problem. One of the difficulties of scum blankets is that it forms an insulation blanket at the top of the tank causing the temperature at this point to drop considerably below the remaining portion of the tank. This condition leads to a cessation of digestion and a larger scum accumulation. The P. F. T. method of discharging hot liquor over the surface of this blanket gradually breaks it up due to heat and the inoculation with seeded liquor.

METHANE BACTERIA'S CONDITIONING OF DIGESTED SLUDGE

MR. FLOOD: Does the particular type of methane bacteria, a rated type, have any effect on the dewatering characteristics of the digested sludge?

PROF. MOORE: From the available data on methane bacteria, it is evident that there must be different strains of the accepted four types of methane bacteria, with some strains being more active than

others. It is possible that some of these strains may condition digested sludge accounting for dewatering difficulties. However, this is just an opinion and I suggest reference to Dr. Buswell.

Ratio of Gas Produced to Pounds of Solids Digested

Mr. Thomas R. Camp: What is the maximum ratio of gas production to pounds of solids digested?

Prof. Moore: Theoretically 1-1/4 pounds of gas or about 18 cubic feet per pound of organic matter digested, for complete digestion of a pure substance.

Mr. McDonald: In actual operation of the 4 primary tanks at Springfield, Mass., the average of 10 years' operation shows that we are getting 8.6 cubic feet of gas per pound of solids added; 11.8 cubic feet of gas per pound of volatile solids added; and 13.2 cubic feet of gas per pound of matter destroyed.

Recirculation Requirements

Mr. McDonald: To go back for a moment to the statements on hydraulic mixing and recirculation pumping, I can definitely say that you cannot get a velocity within the digester adequate to move the solids and break the scum blanket by recirculation pumping.

Internal recirculation by means of paddles or propellers is to be preferred. Recirculation by an external pump is much less effective. The capacity of a pump required to produce a velocity high enough to cause effective recirculation within the tank would be such as to be economically unfeasible. At lower capacities produced by pumps normally used for this recirculation, the velocity is so low that, following the line of least resistance, only the clearer sludge liquor is put in motion and there is little or no movement of sludge.

Grease Disposal

Mr. A. C. Bolde: Should grease be pumped to the digester or kept out?

Mr. Langdon: It is a matter of whether the grease matter is digestible or not. Grease composed mainly of digestible matter should be pumped to the digester, but grease containing mainly mineral, cutting or other non-saponifiable oils should be kept out. The latter will hinder and if present in great amounts will actually stop digestion.

MR. McDONALD: The answer to this is again economics—oils and greases consume man-hours and extra expense. However, they must be treated. If the digested sludge is to be used as a fertilizer, do not add the grease to the digesters. If the digested sludge is to be discarded, then it would be best to add the grease to the digesters, although it is detrimental to a certain extent regardless of whether it is saponifiable or non-saponifiable. However, the digester must have good mixing and scum control.

GREASE AND INCINERATION

MR. FLOOD: Can grease and scum be handled effectively with certain types of incineration?

MR. McDONALD: Yes, but the incinerator plant must be of sufficient capacity since the rates of feed of high grease content sludges must be reduced to control incinerator temperatures.

MR. FLOOD: There is one incinerator small enough to take the grease and scum directly from the conveyor belt from the top of the filter cake and burn it directly—as done at Detroit.

GEOGRAPHICAL DIFFERENCES IN DIGESTION RATES

MR. CAMP: Is it true that digestion rates in the south are a great deal higher than in the north due to a difference in flora?

MR. RANKIN: The apparent differences in digestion rates are due partially to the particular diets or environmental habits of the capita contributing to the various plants.

PROF. ELIASSEN: There should be no marked difference in flora which would characterize the California and Arizona digesters from those in Montana, Washington or Oregon.

PROF. MOORE: There is not very much difference if the engineer designs for the proper temperature and the operator maintains this temperature.

DETENTION OF SOLIDS IN LONG TRUNK SEWERS

MR. E. SHERMAN CHASE: Does retention of sewage solids in long trunk lines acclimate these solids for digestion so that when they reach the digester they are readily digested?

PROF. ELIASSEN: This is true to a certain extent as shown by the formation of hydrogen sulphide in such a situation. This formation shows anaerobic conditions which must be present in digesters.

PROF. MOORE: The physical state of the material is broken down in this long travel so that the solids are of smaller size. The molecular arrangement of these solids is also split into a smaller molecular formation by hydrolytic organisms. Both these actions are predecessors to digestion.

DIGESTER OPERATION

PROF. MOORE: Fluctuating temperatures are to be avoided.

PROF. ELIASSEN: Preheating of sludge before entering the digester seems preferable.

MR. LANGDON: Seeding and heating the raw sludge before entering the digester gives much more uniform operation.

MR. JOHN FRAME: How does the acclimatization of solids in a long trunk line sewer favor the digester?

PROF. ELIASSEN: The solids have arrived at a negative oxidation-reduction potential and have prepared themselves for digestion.

PROF. MOORE: The methane bacteria are strict anaerobes. The hydrolytic organisms preceding the methane bacteria and acclimatizing the solids are faculative anaerobes. These can survive and operate in the sewage under aerobic conditions, whereas the methane bacteria remain dormant under these conditions. Hence the hydrolytic organisms are capable of beginning the digestion by their hydrolytic cleavage action.

PROF. ELIASSEN: This hydrolytic action must take place before final good gas production can develop.

MR. BOLDE: How do seeded and nonseeded digesters compare?

MR. MCDONALD: Digestion will be established regardless of seeding or not, assuming the sludge to be digestible. However, the matter of time is to be considered. It is much more effective to heat than to seed.

MR. FRAME: There ought to be some point in the relationship between the amount of seed to be added and the type of seed.

MR. RANKIN: Seeding is not imperative. A digester will begin satisfactorily provided the pH is watched and industrial wastes are barred.

MR. LANGDON: The quality of the seed and the character of the waste to be seeded are factors to consider in seeding. There are no major advantages in over-all starting of a plant by seeding the

digester. If the tank is brought up to temperature first, then apply a 25% load for the first week or until there is an indication of some digestion before increasing the load, then there will be no difficulties encountered.

Thermophilic Digestion

Dr. Claire N. Sawyer: What are some of the facts on thermophilic digestion?

Mr. Rankin: A thermophilic digester can take a considerably greater loading. Normal thermophilic digestion is accomplished in 16 days as compared to the accepted 30 day digestion. However, problems of operation (heat losses, proper tank loadings, etc.) seem to outweigh some of the advantages in the average plant. It is also noted that thermophilic digestion odors if exposed are extremely bad.

Prof. Moore: Thermophilic digestion may have an advantage in killing the pathogens in sludge when it is to be used as a fertilizer.

Dissolved Solids

Mr. Flood: The quantity of gas produced is compared to the suspended solids, but how should we reconcile gas from dissolved solids?

Panel: The suspended solids test is not the best method for determining sewage solids but it is the one that is used. Suspended solids analyses do not correlate with turbidity tests hence showing a discrepancy in the test.

Fertilizer Value

Mr. Fisher: What is the future of the use of sludge as lawn fertilizer in this area?

Prof. Moore: Most of the fertilizer value of sludge is destroyed during digestion.

Mr. Langdon: Liquid sludge is effective as a soil conditioner but as a fertilizer it has little value.

Prof. Moore: Liquid sludge is similar to raw sewage fertilization but dried sludge has lost its nitrogen and potash in the liquid.

Series and Parallel Operation of Digesters

Mr. Robert Butler: What are the advantages of series and parallel digester operation?

Mr. Langdon: If the solids are to be digested and then disposed of by incineration, then parallel digestion is recommended for best operation and complete utilization of the tanks. If the solids are to be completely digested, disposed of and the supernatant liquor considered, then multi-stage digestion (series) is recommended. In this latter case, the secondary tank would act as a sludge concentrating tank and supernatant separator. However, parallel operation is to be preferred upon the relative evaluation of pounds of solids destroyed per number of tanks.

Mr. Flood: Operations at Washington substantiate the fact that maximum digester operation is obtained with parallel operation.

Mr. Langdon: The same observations were observed at Durham, N. C.

Merits of Three-Stage Digestion

Prof. William E. Stanley: What is the advantage of having more than two stages in series operations?

Mr. Langdon: None.

Prof. Stanley: Have you 3 or 4 or more stages in the Los Angeles County digester arrangement?

Mr. Rankin: Los Angeles County is reported to run on a 10-day schedule and in normal operation, the third stage acts as another sludge storage tank and possibly yields a better supernatant to be returned to the plant.

Mr. Langdon: It seems that ultimate digestion would be a very rapid primary stage of digestion, followed by a comparatively quiet secondary stage in which there will be means of providing for the by-products, the gas and the supernatant. In the activated sludge process, the supernatant liquor has already proven to be a valuable supplement to treatment in control of bulking. We have actually added supernatant to the dosing tank of trickling filters and in a short length of time the filter is cleaned and is much more efficient.

Prof. Eliassen: It seems that rapid digestion (especially with industrial wastes) is still rather theoretical—some other rapid means of treating the supernatant and the separation of liquids and solids would be much more practical.

MR. RANKIN: The problem lies in finding a method by which wastes may arrive uniformly to the digester on a continuous basis. Sludge digestion is one of the few remaining intermittent processes of sewage operation and it is this intermittent factor which offers limitations.

PROF. STANLEY: Would it be possible to cut the digestion time if only a pure culture of the methane organism were present in the digesters in the absence of the other organisms?

MR. LANGDON: The obtainment and control of pure cultures for fermentation (as in particular industries) offer problems and difficulties not encountered in sewage treatment but equal if not greater in intensity to the other problems of normal sewage plant digestion without pure cultures.

PROF. MOORE: Work with pure cultures of methane bacteria indicates that nothing is gained by sterilizing sludge and then inoculating with the methane organism. Apparently, enough of this strain is present in the sewage as it comes to the treatment plant.

PROF. ELIASSEN: An industrial plant maintaining pure cultures is interested in specific end products obtained only by pure cultures, i.e., alcohol fermentation, etc.

SUPERNATANT

MR. BUTLER: What is the practice in returning supernatant? Are you trying to return a clear supernatant or a seeded supernatant to start digestion in the tanks?

MR. MCDONALD: The goal is a clear supernatant—you definitely do not wish digestion in a settling tank or to seed the settling tank. A supernatant, low in suspended solids and low in B. O. D., is desired.

MR. LANGDON: The reason for withdrawing supernatant is to make room in the digester for future additions of raw sludge.

MR. HEANEY: This discussion will now close with a rising vote of thanks for the members of the guest panel on their excellent presentations.

THE PORT OF BOSTON MOVES FORWARD

By George L. Wey*

Brief History of the Port

The development of the Port of Boston commenced immediately upon the colonization of the Massachusetts Bay area in the seventeenth century. It was only logical that Boston became a port as it has one of the best natural harbors in the world. At first, Boston was a terminal port, and later, as the inland area was developed and settled, it became a transfer port. A terminal port is one in which the waterborne cargoes are used in the immediate area of the port for local consumption. A transfer port is such as we now have in Boston, a point of interchange of transportation between water and land.

You all know of the spectacular exploits of our early merchant marine and shipbuilding industries which made the Port of Boston renowned over the entire world and a major port in the United States for over three centuries. The port reached the highest point in its ascendancy at the beginning of the twentieth century, at which point it ceased to progress probably due to three major factors—the complacency of the Port interests and general public on maintaining the competitive position of the Port; the removal of the Port as a point of entry into the United States; and the decision of the United States Shipping Board after World War I to equalize all ocean transportation rates for all the North Atlantic Ports to Europe. The removal of the Port of Boston as a point of entry into the United States was to adversely affect our passenger trade; and when it was reestablished in later years, it was impossible to regain what had been lost.

The equalization of the ocean rates was very unfair to all of the Ports of New England, especially the major Port of Boston, in that certain North Atlantic Ports such as Philadelphia, Baltimore, and Norfolk have a shorter land transportation route from the hinterland than Boston; and the advantage which Boston had of a shorter ocean route to Europe could not be utilized as the ocean rates are the

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same for any of the North Atlantic Ports. However, this discriminatory grain freight rate differential established by the Interstate Commerce Commission has been recently overruled by the Supreme Court of the United States. The decision of the Court culminates action by port interests to remove this unfair condition to waterborne commerce of the Port.

One of the worst-felt aftermaths of the equalization of the ocean rates but not the freight rates was in the grain business of the Port. Up until World War II, very few full cargoes of grain were shipped from the Port of Boston. Mainly, the grain moved as bottom cargo. As a result of this lack of grain export business in the Port, the facilities were not maintained or improved to meet the competition of the other ports. Through the foresight of the Authority in the initial master-planning for revitalization of the Port, two of these grain handling facilities have already been modernized to handle the anticipated increased grain business on a competitive level with other efficient grain terminals along the Atlantic Seaboard.

It would be safe to say that many of the pier facilities of Boston are years behind the times. In the past twenty years, the ships in the maritime trade have increased in both size and draft, consequently requiring deeper and longer docks and larger sheds. The movement of cargo to and from a pier was at first primarily via railroads, but now truck transportation has made tremendous inroads on this business. The great use of heavy mechanical equipment at the present time to handle cargo has superseded the hand trucks of yesterday. Whereas in the old days it was not practical or feasible to stack cargo 18 feet high, the introduction of the fork-lift truck has changed this pattern entirely. The evolution of a terminal can no longer be made on a basis of the number of ship berths but must be on the supporting features for temporary storage and rapid movement on and off of the pier. The Port of Boston Authority is attempting to correct this situation by the rehabilitation of certain facilities, construction of new facilities, and the incorporating of the latest and best thinking on both the construction and operation of the Port terminal facilities for the handling of general cargo. The last modern pier to be built in the Port of Boston was Commonwealth Pier No. 5 in South Boston. This pier was built in 1912, and at that time, was the result of the best thinking of the day, as is obvious by its present condition and layout. It is still considered one of the

finest in the country. Around this period, the development of the Port was under the Directors of the Port of Boston, which organization also constructed the South Boston Dry Dock, at that time, the largest dry dock in the world. Shortly after the construction of these two structures, this organization was deactivated, and the functions turned over to the Waterways and Public Lands. Very little progress was made towards the promotion of the Port or construction of new pier facilities in the Port until the Port Authority was established under the jurisdiction of both the City of Boston and the Commonwealth. This organization was only promotional and advisory in character and therefore lacked the necessary authority to put into effect the various measures and program to achieve our rightful place in world commerce. Finally this organization gave way to the present Port of Boston Authority in 1945.

Port Terminal Modernization Plans

In 1945 the Port of Boston was not in a position to take advantage of the anticipated post-war waterborne commerce because of the inadequacies of the existing commercial facilities. There were ten less ship berths than in the pre-war period; and many of the others were inadequate for the larger vessels constructed during the emergency.

It is generally acknowledged that private enterprise cannot from an economic standpoint justify the construction of new port terminal facilities for the handling of general cargo. For that reason in recent years the private piers in a port were not maintained at the high standard necessary for modern efficiency of interchange process making it necessary for the State or local government to underwrite port terminal facilities. The many direct and indirect benefits to the Commonwealth from world waterborne commerce could not be overlooked. The Legislature provided the Authority with nineteen million, seven hundred thousand dollars for the purpose of modernizing the obsolete marine terminal facilities. However, the authorization contained strict regulatory conditions relative to its disposition. Before the Authority could spend any of this money, it became necessary that a lease be consummated for a proposed pier with a responsible party to use and pay rent on the facilities for a period of twenty years, during which time sixty percent of the cost would be amortized. As you can see, this is a rather difficult requirement, as the lessee

was making a commitment on a "Pig-In-The-Bag" basis, you might say—the project being all on paper. It was not always easy to deal with prospective lessees, as the size of facility desired by them was beyond the scope of the funds which they were willing to amortize as rent. After work on a project commences, it is often necessary to amend the existing agreement each time construction costs increase or unforeseen contingencies arise in order to provide the additional funds needed.

It is safe to say that, conditions being equal, the Port of Boston Authority has constructed the most economical functional waterfront terminals in the country today. In the early stages of a project, a vast amount of time is spent studying the economics of construction and design. Generally, four or five schemes are studied and estimates of cost prepared before one is finally selected for detail development. The lowest cost consistent with low maintenance, efficient function, and degree of permanency is primarily derived from maximum utilization of existing site conditions and the use of heavily loaded steel piles. Hoosac Pier No. 1 and Mystic Pier No. 1 have a unit construction cost of about $10.00 per square foot, which price includes substructure, superstructure, trackage utilities, fire sprinkler and alarm systems, trackwell bridge, offices, outside parking areas, and battery-charging buildings. If the existing topography of the site is necessary to be completely changed, as in the case of the proposed East Boston Pier, the unit cost can increase by 100%.

The present Master Plan for the Port of Boston, developed in 1947, consists of five major projects located strategically in the Port to best serve the three major railroads entering the area from the hinterland; the main highway arteries serving the tributary areas of New England; labor; and navigation. The Master Plan consists of Hoosac Pier No. 1, which is located in Charlestown between the Charlestown Bridge and the Boston Naval Shipyard on the site of the former Hoosac Tunnel Docks. . . . The Mystic Pier No. 1 in Charlestown, on the site of the former Mystic Piers Nos. 46 and 47. . . . The East Boston Pier No. 1, to be constructed on the site of the existing Commonwealth Pier No. 1 and the Grand Junction Pier No. 2. . . . The development of the Castle Island Terminal in South Boston . . . and the proposed development of the waterfront area along Northern Avenue between the Reserved Channel and Commonwealth Pier No. 5 in South Boston.

All of the terminals are finger-type piers and follow a basic pattern such as, twenty-five foot aprons on the main side berths, twenty-foot on the outboard berth, single-story sheds, twenty-foot overhead clearance in the shed for maximum practical stacking of cargo, flush rail tracks on the working aprons for direct interchange of heavy lift cargo between rail and ship, tracks in a depressed well in the center of the shed for flush loading of rail cars, truck docks at the land end of the building for the flush loading of trucks, ramps for truck access and egress into the transit shed, mechanically-operated trackwell bridges for efficient and flexible movement of cargo from one side of trackwell to the other, toilet facilities for longshoremen, warm rooms for perishable cargo, gear lockers for stevedores, two-story office section on the inshore end of the building to accommodate the various pertinent terminal and shipping interests, excellent day and night lighting of aprons and shed interior, and substructures and superstructures constructed of incombustible materials with the exception of the fender systems.

Special consideration is given to the welfare of labor in working conditions and conveniences. A forming or shaping hall for the longshoremen in a separate building which also houses the repair shops for pier cargo-handling equipment is provided. This shaping convenience, which no other port has provided, is a new policy feature established in the Port by the Authority for all new facilities.

As mentioned before, the grain facilities of the Port of Boston were inadequate, obsolete, and very inefficient. With this in mind, the Authority arrived at a plan of modernizing our facilities consistent with the economy of rehabilitating an old but usable grain elevator. Only two of the three port grain handling facilities would be modernized—the Hoosac one-million-bushel elevator and the so-called Grand Junction or Boston and Albany elevator, which also has a capacity of one million bushels. The shipping-out capacity of the elevator in both cases was only about ten thousand bushels per hour. In order to achieve a quicker turn-around of the ship, which is the most important factor as concerns the water carrier, it was planned to increase the shipping-out capacity to thirty thousand bushels per hour and provide loading of five ship hatches at once instead of the present two; the mechanizing of the bagging of grain; better operating communications and conveyor system controls; and the installing of remote control power driven winches to operate the shiploading spouts.

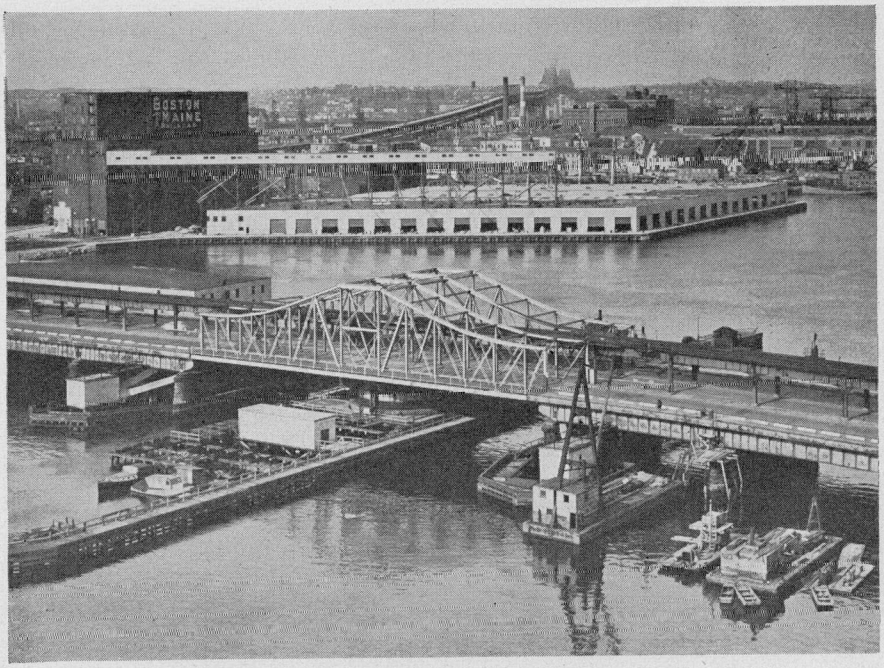

FIG. 1.—HOOSAC PIER NO. 1.

It was impractical to attempt to rehabilitate the Mystic Grain Facility, which had a five hundred thousand bushel capacity elevator, because of its obsolescence, deterioration, and poor functional layout. The grain galleries on Mystic Piers Nos. 48, 49, and 50 have been removed and the elevator demolished to eliminate a hazard and provide more parking and open storage area in the rear of the still operatable Piers Nos. 48 to 50, inclusive.

HOOSAC PIER NO. 1

Hoosac Pier No. 1, Figure No. 1, was completed about two years ago at a cost of about five million dollars, including the purchase of the site, engineering and supervision, interest during construction, and pertinent expenditures. This pier is considered to be one of the most modern and efficient general cargo terminals in the world. It is thoroughly functional in every respect—nothing is ornamental or monumental. The pier has accommodations for berthing and the working cargo of three deep-draft vessels simultaneously. It covers

an area of about 260,000 square feet with a single-story transit shed of about 200,000 square feet. This terminal replaces the former piers Nos. 40 to 44, inclusive, which had accommodations for a total of five ships at one time. The new facility is not only more compact and efficient, but also has a greater cargo handling capacity than the combined old piers.

The substructure found most economical in this location was a steel sheet pile bulkhead with a reinforced concrete relieving platform supported on timber piles. This is the only pier location in the Master Plan that has the proper foundation soil conditions to permit a bulkhead type of structure.

The interesting features of the superstructure are the vertical lift trackwell bridge for two-way traffic across the two tracks, comprehensive fire protection and alarm system, apron and shed interior electric illumination, the natural lighting in the shed obtained by the use of relatively flat skylights. This latter feature has a varied degree of effectiveness due to the roosting of seagulls. The use of a roosting inhibitor is being tested to eliminate this.

In connection with the modernization of the grain facilities of the Port, many of the unsatsfactory features of the existing system as to efficiency have been revised, such as the car door bulkhead openers, car-loading shovels, signal system between ship and grain gallery, remote-controlled power winches for moving and adjusting the shiploading spouts, interlocking of various chutes, gates and conveyors to prevent spillage and blocks.

A very interesting innovation is the three portable units for the bagging of grain for topping cargo. At three locations along the west berth, chutes are provided for dropping the required flow of grain into the portable bagging arrangements, each of which has a capacity of about 15 to 20 tons per hour. The portable units have a large hopper as a reservoir, automatic scales for weighing the bags, sewing machines to fasten the ends of the sacks, and portable conveyors to move the bags either to shipside or from the bagging area to the shed for stacking. In this manner, grain could be bagged in the pier prior to the arrival or during the loading of the bulk grain in the hold of the the ship. It is expected that several days will be saved on the turnaround time of a ship having a large quantity of sacked grain. The old system involved the bagging of the grain in the hold or on top of the hatch. The grain would go into a large wooden box

which would have openings for filling the bags with grain; the bag would be tied by hand and thrown into the hold. This was a very slow operation, and in many cases in the past, required more time than the entire bulk-loading operation.

The Consulting Engineers on the pier were Chas. T. Main, Inc., of Boston, and on the grain handling equipment, Jones-Hettelsater Construction Co. of Kansas City, Mo.

Mystic Pier No. 1

As soon as the Hoosac Pier was placed in operation in July of 1950, work on the preparation of the site of the second step of the Master Plan commenced with the demolition in August of the existing Mystic Piers Nos. 46 and 47. The commercial activities of four ship berths at this location were transferred to the new Hoosac facilities. The first test piles were driven in December, 1950. Completion of the project was originally scheduled for December, 1951; but due to material procurement difficulties in the summer, the date had to be extended to March 1, 1952.

The new terminal will have a total of four berths, one open and three supported by a transit cargo shed. It will not only handle the capacity of the former facilities, but also, due to the more efficient layout, will accommodate the cargo operations at Mystic Piers Nos. 48, 49, and 50. The compactness of the facilities should reflect in lower maintenance, supervision, and security costs. This pier will cover an area of 420,000 square feet, with a transit shed of about 246,000 square feet.

The substructure consists of a wide reinforced concrete wharf apron supported on long H piles driven to rock around the existing earth mole, figure No. 2. The piles have a length varying from 120

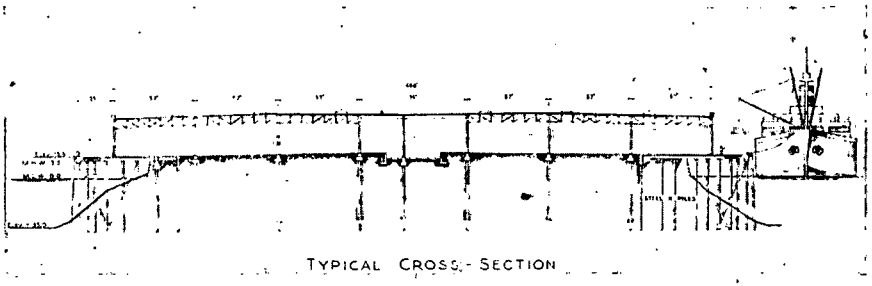

Fig. 2.—Mystic Pier No. 1.

to 160 feet long and are made up in the field by welding two sections prior to driving and one added after the lower piece has been driven. The long piles were required to penetrate through a very deep stratum of soft clay. Economy of the foundations is derived from the heavy loading of the piles and maintaining the existing earth mole which fitted excellently into the new development.

A considerable number of test piles were driven at strategic locations throughout the site at the beginning of this operation and certain ones were test loaded to remove any doubt as to the load bearing capacity of the pile. These tests also permitted closer scheduling of pile lengths, resulting in considerable waste and splicing savings.

The layout of this pier generally follows the basic pattern as mentioned heretofore, except for a few minor details. There are three tracks in the depressed well in the center of the shed, with a hydraulic lift type of trackwell bridge for movement of cargo across the tracks. Another different feature is the presence of two long truck and car loading platforms on the inboard end of the building, one on each side of the tracks entering the transit shed to facilitate the handling of cargo.

The Consulting Engineer on this project was Thomas Worcester, Inc., of Boston.

The construction by the Contractor, Merritt-Chapman & Scott Corporation, is especially noteworthy for expeditious accomplishment of the work. In a period of about a year approximately 70 miles of piles have been driven; 14,000 tons of iron and steel products used, and about 12,500 cubic yards of Portland cement concrete placed. In the driving of the piles, many difficulties were encountered from sub-surface obstructions. At one location, the piles had to be driven through the deck of the existing Chelsea Street Bridge in order to provide support for part of the wharf apron and railroad approach and at the same time maintain normal traffic movement.

This year, the Main Ship Channel to the Mystic Piers was dredged to 40 feet at mean low water by the Federal Government. This project was authorized by Congress primarily conditioned upon the Mystic Pier requiring a depth of 40 feet of water at the ship berths. This pier can handle the deepest draft vessels now afloat. Also this year, the South Channel or so-called Little Mystic was dredged to a minimum depth of 25 feet at mean low water upstream beyond the limits of the new pier. This dredging is part of an overall

plan to deepen the waterway above the existing Chelsea Street Bridge to 30 feet at mean low water in order to provide better navigation conditions for deep draft vessels using this waterway. The 25-foot channel at this time is limited by submarine cables crossing it. The owners of these structures have been ordered to lower them in order to permit the dredging of the channel at a future date to 30 feet.

One of the aspects of the Mystic Terminal Development which has been and is of much concern to the Authority is the elimination of the Chelsea Street South Bridge, the viaduct and the ramp to the terminal area, and the construction by the City of Boston of a new access highway from Medford Street at Terminal Street into the pier area. For the past four years, the Authority has been endeavoring to obtain the approval of the City of Boston of this project, but no definite action has been taken by them in this regard. The Boston and Maine Railroad has been willing to relocate its tracks to provide a right of way over their property. An attempt is being made to present a layout satisfactory to all parties affected by the new roadway. The existing highway approaches at the new terminal are both inadequate and improper to meet the needs of present vehicular traffic, in addition to being costly to maintain, a hazard and an obstruction to the future expansion of the port terminal facilities above the Chelsea Street Bridge.

Upon the completion of the Mystic Project, and the completion of the new access roadway to the pier by the City of Boston, the Mystic Terminal will be without question one of the finest terminals in the entire world for the handling of general cargo. There will be adequate facilities to handle the interchange of cargo between ship, rail, and truck, with open storage and parking. The spacious interior of the shed will allow easy and efficient movement of cargo equipment. The white-coated interior surfaces of the shed provide a maximum sense of cleanliness and reflectivity, along with the excellent artificial and natural lighting.

East Boston Pier No. 1

On July 1, 1949, the Authority consummated an agreement with the New York Central System for the purchase of Piers Nos. 2, 3, 4, and 5, including other facilities south of Marginal Street in East Boston and the lease of the proposed new facility to be constructed on the site of Commonwealth Pier No. 1 and the so-called Boston

FIG. 3.—PROPOSED EAST BOSTON PIER NO. 1.

and Albany Railroad Pier No. 2, together with the remainder of the property in the aforementioned purchase. This agreement provided for the purchase of the property at a price of $3,000,000, and the construction of a new pier, including certain improvements to the grain handling facilities, at a cost not exceeding $4,500,000.

The construction of the new terminal on the site of Piers Nos. 1 and 2 is the first step in a coordinated overall program for the development of all the piers involved in the purchase. The next step is a future consideration.

The proposed pier, figure No. 3, as previously reported, has not been changed in basic layout, which is, briefly, 600 feet long and 390 feet wide, with 25-foot working aprons on the side berths and 20-foot aprons on the outboard end; a transit shed with 20-foot overhead clearance, covering an area of approximately 196,000 square feet; four sets of tracks, one flush with the deck on each side apron, and two depressed in a well at the center of the shed; the side berths to have a piping arrangement for the transfer of bulk liquid cargo from ship to tank cars; ramps for vehicular access into the shed; offices, warm rooms, and truck docks will be provided at the inshore end of the building; more parking and open storage area will be provided between Clyde Street and Pier No. 1; a utility building will be built which will house pier cargo-handling equipment, repair shops, a gasoline station, and a large hall for the shaping of longshoremen for work on the pier. A chain link type of fence will be constructed

around the entire property to provide the necessary security for cargo and terminal facilities. The working area of the transit shed will be devoid of interior columns, making it the ultimate in modern and efficient operating layouts.

The plans provide for the complete demolition of Pier No. 2 and only a small portion of the outboard end of Pier No. 1 to permit the construction of a new access road to the proposed pier from Lewis Street. It is the intention of the Authority to fill around and under the remaining portion of Pier No. 1 and convert this structure into a commercial warehouse or truck terminal facility. This structure, incidental to the construction of the proposed pier, is not included in the lease with the railroad. At a small expenditure, this facility can be altered into a much-needed warehouse or truck terminal having a dry pipe sprinkler system, fire alarm system, railroad trackage, large parking area, ramps into the building, and the deck at truck platform height.

The contract plans and specifications for the construction of the proposed Pier No. 1 were completed by our consulting engineer on the project, Fay, Spofford and Thorndike of Boston. On September 21, 1950, two bids were received. The bids reflected the international crisis in the Far East by increased cost, which exceeded the original estimate, and also exceeded the amount of funds available. After considerable study by both the New York Central System and the Authority, it was decided in the best interests of the Port to reject all bids and delay the construction of this pier until the Mystic Pier No. 1 is completed and authorization is received from the General Court to exceed the previous authorization of $7,500,000 for the construction of the total project. If the international situation became more critical, the new pier could not be completed; and, thus, the Port would be without the use of the existing Piers Nos. 1 and 2 for a long time. The existing piers, although obsolete, inefficient, and badly in need of repairs, will be still usable until the new Mystic Pier No. 1 is completed in 1952.

The estimate of the cost of construction of $4,500,000, on which the agreement was based, was prepared in 1948. At that time the Authority did not have foundation soil exploratory borings to indicate the existing foundation conditions. The estimate was based on a type of pier design for normal foundation conditions, as no funds could be expended for foundation borings until the agreement had

been consummated for the lease of the new facilities. Exploratory borings which were taken after the necessary agreement had been consummated revealed a very poor foundation soil. It was necessary to adopt an entirely pile supported pier in lieu of the conventional earth-filled mole with pile-supported aprons, as originally contemplated. At the time that the change in the design occurred, material prices were on the down-grade, and labor costs had stabilized. However, when the plans and specifications were completed in the fall of 1950, materials and labor took a sharp climb upward.

The lessee, the New York Central System, has indicated a willingness to amend the present agreement by raising thc ceiling on the cost of construcing the new pier, whenever the General Court increases the bond authorization for the project. A reduction in the scope of the pier with the object of decreasing the cost to the funds available has been studied and found economically unsound and impractical.

If the Port is to continue to handle its present volume of commerce, it is imperative that the East Boston Pier be completed as soon as possible. The present deteriorated condition and unsightly appearance of Pier No 2 are a poor advertisement for the Port.

Also included in this project is the modernization of the grain handling facilities at the East Boston Terminal. In order to place this port grain facility in a competitive position with other North Atlantic Ports, it was necessary to modernize and increase the capacity of the shiploading conveyor system of the east side of Pier No. 4 from 10,000 to 30,000 bushels per hour, including additional trippers to permit simultaneous loading of four ship hatches at one time; install remote control power-driven winches for the operation of the shiploading spouts; rehabilitate the grain gallery supporting structure; and install electric power outlets for connecting portable grain trimming machines, at a total cost of approximately $450,000. In order to ease the extremely congested and difficult traffic pattern in front of Pier No. 3, the grain gallery with supporting structure at the west side of Pier No. 3 was dismantled and removed. This grain gallery had been abandoned several years ago, as it was found that the remaining three shiploading systems would be adequate to handle the output of the grain elevator. The demolition of this structure also eliminated a potential fire hazard and a source of additional maintenance expense.

The contract plans and specifications for the modernization of the grain handling facilities were prepared by our consulting engineers, Wm. T. Donovan Company of Boston.

It was a well-known fact that the access roads were far from being ideal and that they should be improved. At the present time, large trailer trucks find it very difficult to make the various sharp turns and negotiate the narrow roads passing from the vicinity of the Sumner Tunnel to the piers off Clyde Street. This condition also has a direct bearing on the fire protection of the piers, as the difficult and congested traffic situation would impede the speedy arrival of fire apparatus at the piers. The only access into the terminal area, which is over Clyde Street, requires passing over the main tracks of the Boston & Albany Railroad. The Railroad movements into the terminal result in constant delays to the efficient movement of the trucks to and from the terminal. A study of the most practical solution to the highway problem indicates that a new roadway should be constructed from Lewis Street near the South Ferry across the westerly slip of Commonwealth Pier No. 1 in front of the proposed new pier. This location would fit in with the original plans of the Authority for the comprehensive development of all the piers in the East Boston Terminal area.

The proposed roadway would have a width of about 83 feet and a length of 350 feet, extending across the westerly dock of Pier No. 1, which at the present time has a depth of water of 35 feet at mean low water. The highway as proposed would be a combination of fill and pile-supported roadway. The estimated cost of making this improvement is approximately $200,000. Loss of time due to difficult traffic patterns and congestion is reflected in the increased cost of the commodities passing over the piers, therefore having an adverse effect on the promotion of commerce through the Port. In a market of constant rising labor and material costs, there would be no advantage in delaying construction in the hopes that construction costs will be lower. By the elimination of the proposed utility building, west approach, east roadway widening, the fill between existing Pier No. 1 and Clyde Street, and certain minor revisions in the design, a saving of $450,000 can be accomplished without detriment to the functional considerations. Further reductions would not be advisable. Based on construction costs as of November, 1951 and the above-mentioned reductions, the minimum cost of the proposed pier would be $9,000,-

FIG. 4.—LOCATION OF PROPOSED CASTLE ISLAND TERMINAL.

000, and the modernization of the grain facilities $450,000, making a combined total of $9,450,000 for the entire project.

CASTLE ISLAND TERMINAL

The Castle Island Terminal, figure No. 4, was constructed by the United States Army as a port of embarkation during World War II. It comprises an area of approximately 101 acres, of which 76.5 acres are owned by the Commonwealth, 18.3 acres by the Federal Government, and 6.3 acres by the City of Boston. The land owned by the Commonwealth and the City of Boston is under lease to the Federal Government until July 30, 1961, at which time the property is to be returned to the Commonwealth and the City of Boston. Since 1946 the Authority has been operating the Terminal as a commercial facility under an interim permit, first from the War Assets Administration, and later from the Navy Department, when

the property was transferred to that department. Negotiations have been carried on since 1946 with representatives of the Federal Government with a view of obtaining the return of the land of the Commonwealth and the acquisition of the improvements made thereon by the United States Government.

Full control must be obtained by the Authority before it can embark on the initial step of the development of the terminal as a commercial facility for the handling of waterborne commerce, as authorized by Chapter 625 of the Acts of 1948, at an expenditure not to exceed the amount of $1,200,000. The existing terminal was constructed primarily for military operations, and the structures are more or less of a temporary nature.

Last year negotiations with the Navy Department culminated in a definite proposition in which the Federal Government would transfer all right, title, and interest of the United States to the property known as Castle Island Terminal, including Government-owned land and improvements thereon, to the Commonwealth, provided certain Commonwealth-owned lands in South Boston needed by the Navy Department were conveyed to the Federal Government. At the present time, the necessary conveyances between the Commonwealth and the Federal Government are being prepared to effect the exchange of property. It is expected that the transfer will be completed before the spring of 1952.

The Castle Island Terminal has potentialities for development into an ideal commercial port terminal. It has a marginal wharf which is 4200 feet long, with a controlling depth of 35 feet at mean low water at the berths. There are two existing transit sheds, each 840 feet long, by 180 feet wide, one of which is a more or less permanent structure, but too far removed from the caplog for maximum efficiency and flexibility of operation. The other is a temporary wood structure having a close column which prohibits efficient cargo handling and stowage. There are also many small temporary buildings which constitute a fire hazard and will be removed in the first-stage development of the terminal. The roads of the terminal are practically non-existent, as they were originally of a temporary nature. The tremendous classification yard has a total capacity of approximately 650 rail cars. The entire terminal is lighted for night operations by banks of flood lights in structural steel towers located strategically throughout the terminal.

The first-stage development plan to convert this terminal into a modern, efficient commercial facility consists of the following improvements and alterations:

(1) Replace about 1000 untreated timber piles in the wharf apron which are undergoing a severe marine borer attack. It is proposed to eliminate this inherent weakness and insure the integrity of the wharf by replacing the untreated piles with properly creosoted piles. It is unfortunate that the wartime stress for speed of construction made it necessary to use many untreated piles in the wharf apron.

(2) The demolition of Transit Shed No. 2 and the construction of a new one-story transit shed of fire-resistant materials, approximately 500 feet long by 200 feet wide. The new shed will have offices for Customs, steamship companies, and stevedores; toilet facilities for the longshoremen; a warm room, a gear locker, with two shipside tracks flush with the deck and two in a depressed area in the rear of the shed for floor-level loading of freight cars. The rear of the shed will have a 15-foot wide canopy loading platform, and the track area will be paved to permit handling of truck cargo at the platform.

(3) Transit Shed No. 1 will be extended toward the caplog, a distance of about 60 feet, and increased in length about 180 feet toward Shed No. 2, in order to provide a better working apron and adequate covered transit storage area for the handling of two ships at one time. Offices will be constructed at each outboard corner of the building for steamship companies and Customs; there will be toilet facilities for the longshoremen, a warm room, and a gear locker. The existing wood apron doors will be removed and replaced with rolling steel doors. Skylights will be installed in the roof to provide adequate natural lighting for the interior of the shed and the electric lighting system will be revised to conform to present-day standards.

(4) The removal of all unnecessary trackage and the revision of the existing layout for efficient movement in classification of rail cars consistent with low maintenance. The existing holding yard capacity of approximately 650 cars will be cut down to about 80 cars. The shipside tracks along Transit Shed No. 1 will have to be moved closer to the caplog, in order that the ship's gear can handle cargo directly from cars to the hold of the ship.

(5) Replacement of the entire underground water supply system,

which has been found to be in very bad condition due to electrolytic and chemical action. This work is necessary to provide an adequate source of water supply for fire protection of the terminal, and for servicing the requirements of ships while at the docks.

(6) The existing temporary roads are in very poor condition and require reconstruction on a permanent basis. The entire existing layout of roads will be revised to permit a more desirable traffic pattern, security, and maximum use of the area comprising the terminal.

(7) A terminal administration building will be constructed in the rear of and between Sheds Nos. 1 and 2 for the housing of offices of the terminal operators; District Customs, including bonded, and gear lockers; and firms in the maritime trade and a restaurant and lunchroom for personnel working at the terminal. Adjacent to the terminal administration building will be a large parking area for cars having business at the waterfront facilities.

(8) A single-story storage building will be constructed in the rear of Transit Shed No. 1 for use as a supporting storage facility in connection with waterborne commerce. This shed will be of one story, approximately 120 feet wide by 300 feet long, with ramps for truck entrance into the sheds, and tracks in a depressed area in the rear of the building to permit floor-level loading of freight cars.

It is expected that a reduction in the scope of the first-stage development will have to be made in order to complete the work within the funds available, due to the increase in construction costs from the time the original estimate was made of the project in 1948. The revision of the scope will have to be made at the time construction is ready to commence. There have been many inquiries made by various commercial and industrial enterprises using waterborne commodities to establish plants on the terminal. All of these propositions are being studied. The original plans may have to be revised to fit certain requirements of the lessees. After the completion of the first-stage development, consideration will have to be given to further development after a complete study is made of the commerce requirements and the economic benefits to be derived from the proposed improvements.

Northern Avenue Development

The Northern Avenue project is the fifth step in our Master Plan for the modernization of terminal facilities in Boston Harbor. Legis-

lation to carry out the plans for the development of the Northern Avenue waterfront was submitted to the 1951 session of the General Court for their consideration.

Until World War II, the so-called New York, New Haven & Hartford Railroad Piers Nos. 1 to 4, inclusive, were used for intracoastal trade. During the war and since, these facilities have been abandoned and allowed to deteriorate to such an extent that rehabilitation is impractical. This location is ideal for development as a combined passenger and cargo terminal, since it is situated on the Main Ship Channel with a depth of 40 feet at mean low water; it is sheltered against rough water; the area encompassed would allow an extensive open storage and parking area for cars and trucks; close by is a large railroad classification yard, near the main arteries leading to and from the city and close to the business district of the city and to rail, air, and bus transportation. The development of the area would not disrupt any strongly-rooted businesses or require the taking of sound, usable structures. The existing structures are dilapidated and present a very unsightly appearance.

The project would consist of one combined passenger and general cargo terminal having a two-story building approximately 200 feet wide by 500 feet long; one general cargo terminal having a transit shed 200 feet wide by 500 feet long; another general cargo terminal having a transit shed 200 feet wide by 600 feet long; a three-story industrial center building approximately 200 feet wide by 600 feet long; a vehicular ramp to the second floor of the passenger terminal; roads, open storage, and a parking area. After further study it is felt that the project should be broken into two parts. The first part consists of the construction of a concrete wharf apron around three sides of the terminal; filling the tidewater area between Piers Nos. 1 to 4, inclusive; construction of the combined passenger and general cargo terminal, with a vehicular ramp to the second floor; the two transit cargo sheds; trackage; roads and parking areas. The second part, to be constructed at a later date, would be the industrial center, a three-story fireproof building having ramps, truck and rail loading platforms, elevators, and such other appurtenances and features as would be necessary for efficient and flexible use of the building for commercial, industrial, and warehousing operations relative to the maritime trade.

There have been many worth-while suggestions and recommen-

dations as to possible uses for this industrial building that would be advantageous to the development of the waterborne commerce of the port. Some firms have expressed an interest in a graphic arts center; the storage of raw bulk commodities; the use of the ground floors for a union truck terminal; and an international trade center, such as created in the Port of New Orleans. An international trade center would be extremely beneficial to the commercial and industrial welfare not only of the State, but also of New England as a whole.

The first project under consideration is the passenger and general cargo terminal. The passenger service would be exclusively on the upper deck, while the lower deck would be used primarily for the handling of general cargo. The second deck would also include the general offices of the Port of Boston Authority; offices for Customs and steamship lines; a restaurant; waiting rooms; and other conveniences and services pertinent to the passenger trade. There would be a freight elevator for transferring baggage and cargo between the first and second decks.

It is necessary that the offices of the Port of Boston Authority be transferred from Commonwealth Pier No. 5 in order to modernize that pier to permit more efficient operation and to meet more completely the transportation requirements of the present day. Pier No. 5 was constructed about forty years ago, and since that time transportation means and commerce conditions have changed. The pier, however, has not been altered from time to time to keep pace with these changes. The offices on Pier No. 5 are not consistent with modern standards for operation and the comfort of the employees, as it is primarily an adapted cargo shed.

At the time that Pier No. 5 was constructed, the interchange of cargo between ship and rail was in the majority. Today, truck transportation handles the greater amount of cargo on this pier. Therefore, the functional layout of the pier should be revised to take care of this changed condition. It is not uncommon nowadays to find the highway approaches to the pier cluttered with trucks waiting to load or unload on the pier. This situation is not prevalent at our new piers. Ineffectual movement of cargo off the pier has an adverse effect on the use of the port by shippers, and also is reflected in a decreased capacity of the pier to handle cargo across its aprons. The area occupied by the offices should be made into one branch of a circulatory roadway through the upper level of the pier to permit continuous

traffic movement and maximum truck loading or unloading operations. At the present time trucks cannot enter, load, and continue out in the same direction, but must turn around and leave by the same way they entered. The capacity of the pier to handle the interchange of cargo would be increased by at least 40 per cent. However, the modernization of Commonwealth Pier No. 5 would have to await completion of new quarters for the various functions of the Port of Boston Authority.

There is a definite need for adequate passenger service accommodations in Boston with facilities that are modern in every respect and designed solely for such service. Special attention would be given in the layout to the elimination of the confusion that exists in most passenger terminals because of the intermingling of passengers and visitors and the processing operations of disembarkation and embarkation. Marine passengers at the Port of Boston are required to use the same facilities as are used in general cargo handling operations—unheated, cluttered, dirty, and bare of conveniences in comparison with terminals of other modes of transportation.

The motor vehicular and pedestrian ramp to the passenger terminal, which is to be on the second deck of the proposed new structure, would enable passenger and other operations of the second floor to be carried on independently without interference from the first floor general cargo activities. The large parking area that would be available would permit visitors and passengers to arrive in their private cars. This is an aspect that is sadly lacking at most piers throughout the country.

The economic study indicates that the revenue derived from the rental of the actual passenger terminal, the offices, the concessions, the parking, and the general cargo function would be more than adequate to amortize the cost of the project as required heretofore for other proposed port terminal facilities. The only difficulty would be in obtaining long-term leases for a period of twenty years. A more operative requirement should be made in this aspect.

Although many do not concur as to the need of a passenger terminal as described above, it cannot be denied, after a thorough study, that the benefits, both direct and indirect, to be derived from the marine passenger business would make this a very worth-while venture, and would be in the best interests of the Commonwealth. Some of the recipients of the benefits from this trade would be hotels, taxi

services, airports, restaurants, stores, longshoremen, banks,—in fact, almost every walk of life. To develop and increase use of the port by passenger steamship lines, it is necessary that progressive facilities be made available to them.

The second terminal building mentioned heretofore, 200 feet wide by 500 feet long, would be an ideal location for the creation of a foreign trade zone. This area would be very well situated from the standpoint of security, and interchange between rail, ship, and truck transportation, as required by a foreign trade zone. Such a zone would permit foreign merchants to store samples and goods that they wish to market in the United States, for inspection by American buyers, or for transshipment to other foreign markets, without payment of import duty. In the zone, goods could be inspected, processed and blended, or packaged before entering Customs territory and becoming subject to the payment of the United States duty. Practically every major port in the United States has a foreign trade zone. There are definite benefits to be derived from foreign trade zones, such as an increase in employment for longshoremen and processing workers, the encouragement of new commercial businesses, the establishment of new industrial plants, and other desirable enterprises. This foreign trade zone would also stimulate the world-wide commerce of the port.

The general cargo facilities are within the economic considerations of construction and amortization. In the past, there have been several inquiries from terminal operators and passenger lines regarding rental of new terminal facilities at this location.

The transit cargo sheds would follow the same basic pattern established for the other steps in the Master Plan with effort made in attaining maximum column spacing in the building consistent with economical design and funds available. The buildings would have a dry pipe sprinkling system, shipside and loading platform tracks, maximum natural lighting, offices for customs and steamship lines, a canopied rear loading platform, working apron, flood lighting, and ramps for vehicular passage into the sheds.

As a result of the economic studies of the substructure design, the wharf will be a wide reinforced concrete apron, supported by long steel H and pipe piles to bedrock. In order to permit expansion of the passenger services facilities when future conditions warrant, the foundations of the transit shed adjacent to the passenger terminal

have been designed to carry an 80-foot wide passenger gallery with an observation mezzanine on the roof. The gallery would be connected to the main terminal by a 100-foot long second-floor bridge. The maximum length of the passenger terminal with gallery would be 1100 feet, permitting the accommodation of the largest passenger ships.

Another desirable aspect of the development is the terminal's large 800-car parking area which should assist considerably in relieving the parking problem, and indirectly the traffic conditions, in this section of the city.

Additional Port Requirements

The passenger facilities on the second deck of Pier No. 3 of the so-called Grand Junction Docks in East Boston now owned by the Commonwealth should be modernized and placed into service again.

This area, since Colonial times, has been one of the principal locations in the Port for the handling of waterborne cargo and passengers. Near the turn of the twentieth century, the existing Pier No. 3 was constructed as a combination general cargo and passenger terminal. Until the thirties, this terminal enjoyed considerable passenger trade. Since the war, the passenger trade is practically nil, as it is impossible to handle passengers because of the present deplorable condition of the facilities.

A necessary adjunct of any progressive port is provisions for handling waterborne passenger traffic. At the present time, the Port of Boston does not have a terminal which is adequate or proper under the present-day standards for the handling of passengers. There are a great number of ships in the maritime trade that carry both passengers and general cargo. Without adequate facilities for taking care of passengers, this trade is lost to other Atlantic Ports. Until the Northern Avenue Development passenger terminal is constructed, Pier No. 3 in East Boston should be modernized for this service as the initial step towards regaining the marine passenger trade.

Last year the Authority made a study of the desirability of making repairs and improvements to the passenger facilities of Pier No. 3. It was found that this terminal, as in the case of Commonwealth Pier No. 5, did not keep pace with periodical changes in terminal operation, transportation, and standard of living during the past 50 years. The passenger and freight elevators are unsafe; the stairways are inadequate; the toilet and Customs inspection facilities are improper;

there are no waiting rooms; the lighting and ventilation are poor; there in no flexibility in ship gangway locations; the terminal is difficult to clean; and the interior is very unattractive.

To modernize the passenger terminal facilities of Pier No. 3 besides rectifying the deficiencies mentioned above, the passenger galleries, one on each side of the pier, must be widened and lengthened to permit accommodation of larger vessels in the trade and decrease passenger and baggage congestion. The cost of making the necessary alterations and improvements to the existing structure is estimated to be about $400,000.

(a) *Consolidated or Union Truck Terminals*

Of much concern to the Port directly and indirectly is the constantly growing congestion in the city from motor vehicle traffic. A vitally needed central and circumferential artery with limited access is being constructed by the Massachusetts Department of Public Works through and around the City of Boston to facilitate the movement of the highway traffic to, from, and through the City. However, it will not relieve the terrific traffic congestion except in a small way within the business districts, to and through which port cargo must travel. Relief for this traffic problem must be forthcoming or otherwise congestion will slowly strangle the business of the Port and the City.

In this metropolis, which has borne such an important part in the history of these United States since Colonial days, a heritage of narrow and irregular streets is found in the present mercantile sections of the city. To revise the street layout of these congested business districts would be impractical. The only alternative is to provide relief through facilities and regulations that would minimize this problem. One type of relief is the off-street parking lots and structures. The City has recognized this necessity and has already embarked on a noteworthy program. Another relief measure that should be given consideration, whether constructed and operated by the Commonwealth or the City, is a consolidated or union truck terminal.

In the city today, it is not uncommon to find a single large truck unit that is picking up or delivering relatively small shipments of goods to many scattered points throughout the business district, resulting not only in the wasteful use of the so-called over-the-road equipment, but also an irritating impediment to traffic flow—in some

cases, complete stoppage of traffic at points of business calls. This condition can and positively should be regulated.

On the waterfront, large over-the-road trucks can also be found delivering and picking up small lots of cargo on the piers, resulting in congested pier approaches and inefficient interchange of cargo. This condition increases the trucking cost and eventually results in a higher commodity price to the consumer. This condition can and should be minimized to practical considerations.

At the present time, there are numerous truck terminals widely scattered throughout the city, most of which are makeshift terminals. Others are located in congested commercial areas, some on narrow streets and traffic-congested main thoroughfares. It is not an uncommon sight to find a large trailer truck completely blocking traffic when backing into one of the makeshift on-street terminals. This sort of condition reacts adversely against the welfare of both the City and the Port by unnecessary delays to movement of goods, higher costs of the transportation due to inefficiency and congestion, and the strangling effect it has on a general routine business activity.

An answer to a corrective measure would be a consolidation of the city truck terminal activity as much as possible into one or two of the so-called union terminals and the passage of such regulations as may be necessary in the best interests of the public for the distribution and pickup of goods by truck transportation within the congested business districts of the City of Boston. A union truck terminal should not only be desirable from the standpoint of the City and Port business, but also the trucking interests. The benefits to the trucking interests should be faster and more efficient pickup and delivery of goods, quicker interchange of freight between terminals, and the more efficient use of employees and trucks.

The Port of New York Authority has constructed two excellent so-called union motor truck terminals, one in New York City in 1949, and the other in Newark, New Jersey, in 1950. These are operated by the Authority. Several years ago the City of Boston Planning Board made a study and reported favorably on a union truck terminal similar to the ones initiated by the New York Port Authority.

In my opinion, this type of terminal and operation would not be equitable to existing conditions and requirements of the Boston Port area. The facility required here would be a concentration of motor carrier terminal activity at one location with the operation left

in the hands of the tenant carriers as much as possible. The ownership, maintenance, and supervision of the material facility, along with the administration of the operating rules and regulations, would be vested in a Governmental agency. This type of union motor truck terminal should meet with favor by all affected, both public and private interests.

From preliminary investigations and studies made on the proposition outlined above, it appears that any revenue derived from such an activity in rentals and concessions would be more than adequate to amortize the cost of the facility, besides providing for the annual maintenance and administration expenditures. There is an ideal site for such a terminal on the land owned by the Commonwealth in South Boston. The apparent many advantages and economics to be derived through the use of this type of facility cannot be overlooked when it affects so greatly the welfare of the City and of the Port.

(b) *Marinas*

One aspect of harbor development and utilization that has been neglected in the past is the construction of public marinas for travelling pleasure craft. A public marina practically serves the same function for these pleasure craft as a motel or motor court does for a motorist. It provides the transient yachtsman with a place to safely moor his yacht while he is sojourning for a few days, taking on fresh stores and supplies, replenishing fuel and water, obtaining mail from the post office, shopping, city sight-seeing, or visiting friends.

Boston Harbor, one of the most beautiful natural harbors in the world, would provide an ideal stopping place for the yachtsmen who travel along the coast. Besides being sheltered against storms and rough water, it has an historical metropolis which can supply all the wants and pleasures of the sojourning yachtsman. Motorboating and yachting are constantly growing in popularity throughout the country. Some of the so-called local yachtsmen make occasional trips of a few days' duration along the coast during the summer months; others spend longer periods of time at interesting distant places. Then, there are those who make stops along the way while travelling to the southern climes in the winter and in the summer return to the cooler New England States. A marina in an attractive location such as Boston Harbor would bring many direct and indirect benefits to the Port area such as work for boat repair yards; sale of fuel, sup-

plies, and goods; patronage of sight-seeing and recreational activities; etc. It would also be a worth-while promotional advertisement for both the Port and the City.

One location for a marina suggested about twenty-three years ago and still considered an excellent site is in Pleasure Bay, adjacent to Castle Island in South Boston. Another excellent location would be off of the site of the existing Northern Avenue Bridge, as recommended in the report made on the feasibility of filling Fort Point Channel in South Boston. There are also excellent sites in East Boston, in Boston proper adjacent to Charlestown Bridge, and on the Neponset River in the Dorchester District of the City of Boston. Each of these sites would have convenient access to municipal water, electric power, telephone service, good roads, frequent bus and street-car service to the center of Boston, and are near to local stores. Some of these sites are remote from city noises in pleasant surroundings, quiet waters, with more or less seclusion. Others are within walking distance of the heart of the city.

Almost every large municipality in the State of Florida has found it desirable and beneficial to have a public marina for the itinerant or cruising yachtsman. Some of the other states that have public marinas are New Jersey, Mississippi, Connecticut, California, New York, and Georgia.

A public marina does not compete in any way with commercial or yacht club activities but provides certain services and facilities which no other can supply to the yachts and motorboats making Boston a port of call. For these services and facilities a fair rate is charged, depending on the size of the craft and the amount necessary to amortize the first cost along with annual operation and maintenance expenditures. Such a public facility could be made available to private interest, if desirable, for operation on a lease basis.

The operating and construction costs of a marina depends greatly on the site and generally would require not less than a 48-boat capacity layout for economic justification. Because of the large variation in tides in Boston Harbor of nine and one-half feet, the construction of adequate mooring devices, gangways, and walkways to serve the boats would require a more elaborate setup than normally found.

Existing boat mooring facilities for both temporary and permanent residents, as found especially around Warren Bridge, should

not be tolerated. They constitute a fire hazard and result in a unsightly and cluttered waterfront appearance. With proper public facilities, these boats can be made to move into a marina where adequate supervision can be given to our waterfront appearance, fire-prevention measures, and control of tidewater pollution.

Although this subject of marinas is not of prime importance in the Port, it is desirable as a public service and is consistent with making the greatest use of nature's gift of a fine harbor.

Conclusion

The Port of Boston many years ago was left to drift into a dormant state until creation of the Authority in 1945. The Port cannot be revitalized overnight, so to speak, to make up time for a long period of disinterest; but the impact must be distributed carefully, as being done, over a long period of time on a predetermined plan which must be constantly re-studied to avoid under or over-development. In 1945 there were actually ten less deep-draft vessel berths in the Port than there were in the pre-World War II period. The number and condition of the remaining piers were sad in comparison to other major ports.

During the existence of the Authority, the Hoosac Pier No. 1 was constructed; the Mystic Pier No. 1 will soon be completed; and the proposed East Boston Pier No. 1 will be started next year, all steps within a coordinated comprehensive Master Plan for the modernization of existing obsolete terminal facilities. The water-borne commerce of the port has been gradually increasing; every effort should be made to keep this trend moving forward by promotion or more efficient marine terminal facilities, improved transportation and related activities, and a more cooperative spirit and understanding between labor and employer.

Actually, the Master Plan to date considers only the modernization of existing facilities; the creation of additional new ones will be a future consideration to be covered by subsequent reports as their need arises. The Port must be progressive and have a competitive spirit if it is to succeed—there is no room for any complacency.

DESIGN OF THE BUCKLIN POINT SEWAGE TREATMENT PLANT

By John S. Bethel, Jr., Member*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on May 17, 1952).

The Bucklin Point sewage treatment plant, located in East Providence, R. I., is the main plant of the Blackstone Valley Sewer Distribution Commission of the State of Rhode Island. The history of the District and the results of an engineering study relative to the problem of pollution of the Blackstone, Moshassuck, and Seekonk Rivers, together with recommended remedial measures, have been presented previously by Hammann (1) and Chase (2). These measures included interception of the bulk of the sewage and industrial wastes originating in the District and the construction of a 47-mgd. (million gallons per day) sewage treatment plant at Bucklin Point to remove sludge-forming solids and to disinfect the effluent prior to its discharge into the Seekonk River. It was further recommended that the remainder of the sewage and industrial wastes be intercepted and treated in several smaller plants. The Commission approved these recommendations and ordered construction of the first step of the necessary facilities.

It is estimated that in the year 1975 the total population of the District will be 221,800, and that the population served by sewers tributary to the Bucklin Point plant will be 158,600. The quantity of domestic sewage originating from the sewered areas at that time is estimated to be 17.8 mgd. It is also estimated that in 1975 the quantity of industrial wastes, mainly textile, tributary to Bucklin Point will be 29.4 mgd. The total flow requiring treatment at Bucklin Point, therefore, will be 47 mgd.

In addition to contributing the major part of the flow, industry will also contribute a large portion of the 5-day 20°C B.O.D. (Biochemical oxygen demand), and suspended solids, as shown in Table 1.

The effect of this industrial load is best illustrated by the fact that on the basis of B.O.D. and of suspended solids the computed equiva-

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TABLE 1.—ESTIMATED (1975) RAW SEWAGE FLOWS. ORIGIN AND COMPOSITION

Source	Flow		B.O.D.		Total suspended solids		Volatile suspended solids	
	Mgd.	%	*	%	*	%	*	%
Domestic sewage	17.8	38	19,710	31	18,680	44	14,235	43
Industrial wastes	29.4	62	43,470	69	23,310	56	19,035	57
Total	47.2	100	63,180	100	41,990	100	33,270	100

*Pounds per 24 hours.

lent populations are 315,000 and 250,000, respectively, compared with the estimated 158,600 tributary population.

Basic Data and Flow Diagram. A tabulation of the basic design data is presented as Appendix I, a list of the major equipment manufacturers is given in Appendix II, and a flow diagram and nomenclature of the structures are shown on Fig. 1.

Description of Plant. The Bucklin Point plant, located on the east bank of the Seekonk River at the northern end of East Providence, serves the cities of Pawtucket and Central Falls, the towns of Cumberland and Lincoln, and the northern portion of the town of East Providence. The treatment facilities include Parshall flumes; mechanically-raked bar racks with screenings grinders; mechanically-cleaned grit chambers with grit conveyors, washers, and grit-storage facilities; preaeration tanks equipped with swing-type diffusers; mechanically-cleaned settling tanks; chlorination facilities, and sludge digestion tanks, together with elutriation and dewatering equipment. The sewage flows through the plant by gravity.

Diversion Structures. Sewage will enter the plant from two interceptors, namely, the Blackstone Valley Interceptor, a 7 ft. 6 in. semi-elliptical reinforced concrete sewer serving the area to the north of the plant; and the East Providence Interceptor, a 48-in. diameter reinforced concrete pipe serving the area to the south. Immediately within the plant site a diversion structure is located on each interceptor, to limit the quantity of sewage going to the treatment plant. The two structures are similar and house an overflow weir, an electrically-operated sluice gate, and a wooden slide gate for emergency use. The

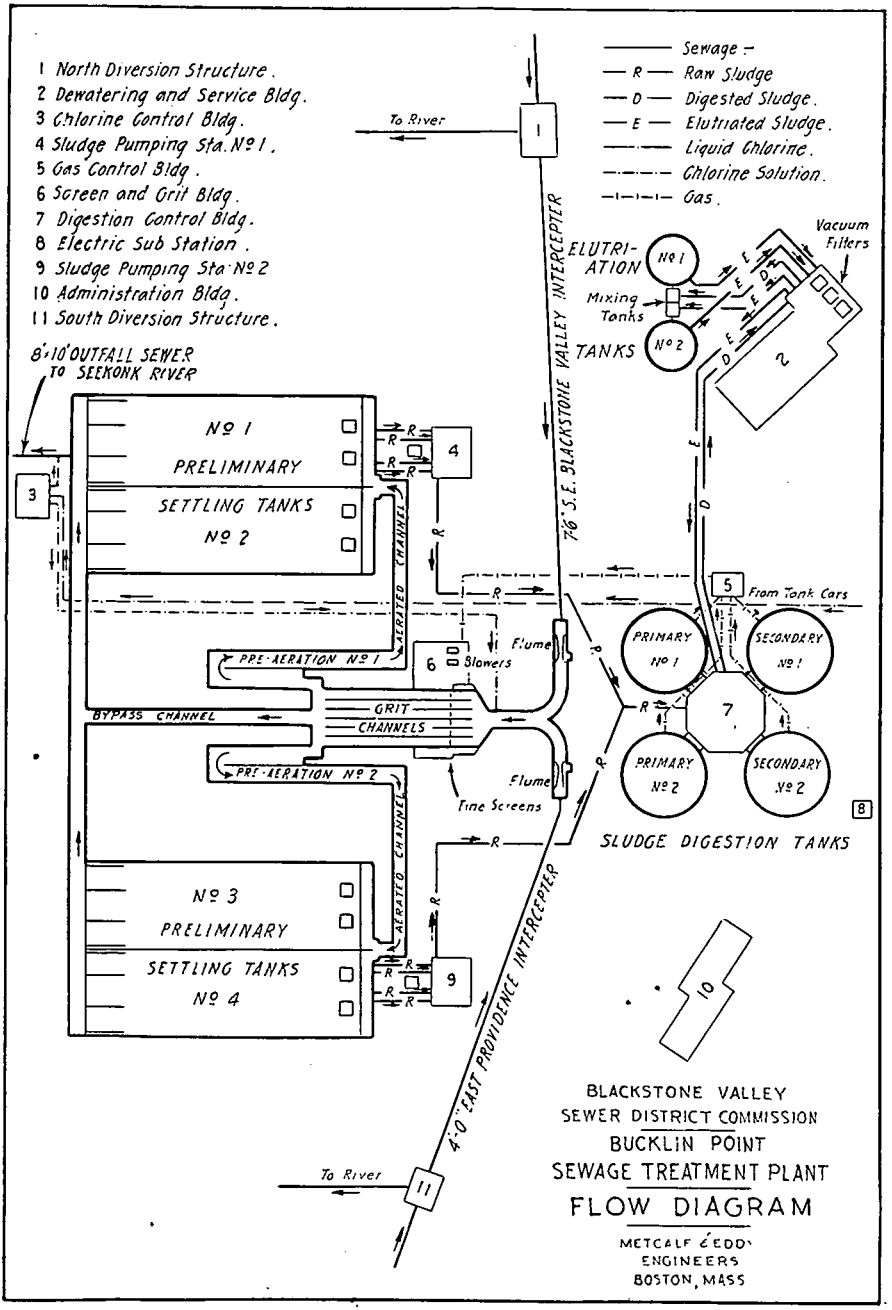

FIG. 1.—FLOW DIAGRAM.

weir in the north structure on the Blackstone Valley Interceptor will divert all raw sewage flows in excess of 128 mgd. to the Seekonk River, whereas the weir in the south structure on the East Providence Interceptor will divert all flows in excess of 41 mgd. The maximum rate of sewage flow through the plant is, therefore, 169 mgd.

During construction of the plant these structures were used as temporary chlorination stations, as has been reported previously by Hammann (3).

Parshall Flumes. From the diversion structures, the sewage will flow through two Parshall flumes. The throat width of the flume on the Blackstone Valley Interceptor is 6 ft., and on the East Providence Interceptor is 3 ft.

The schedule of flows to be passed through each flume and the total flows entering the plant, is given in Table 2. Provision has been made to record the quantity of sewage passing through each flume and to totalize these flows.

TABLE 2.—ESTIMATED (1975) RAW SEWAGE FLOWS FROM EACH INTERCEPTOR

	Quantity in mgd.		
	Blackstone Valley Interceptor	*East Providence Interceptor*	*Total to plant*
Peak flows	128	41	169
Daily max. flows	85	33	118
Av. daily flows	34	13	47
Daily minimum flows	26	9	35
Instantaneous min. flows			
Weekday	17	6	23
Weekend	7	4	11

Because the District covers an area of 75 square miles, it is possible for heavy rains to occur over one section of the District while no rain is falling in other areas. Thus, one of the interceptors may be carrying peak flows, while the other interceptor may be carrying considerably less than peak flow. Under these conditions it is possible that the flume carrying the low flow may be submerged. Recording instruments are provided at each flume to furnish a continuous record of the duration and degree of submergence.

Fine Racks. After passing through the flumes the two sewage

streams will converge in the inlet channel to the Screen and Grit Building, and will then pass through six mechanically-cleaned bar racks. Each rack is 5 ft. in gross width, with one-half inch clear opening between bars, and is designed to handle flows varying from 10 to 27 mgd. The depth of sewage at the racks will vary from 2.5 ft. to 5.0 ft. Racks will be inclined about 20 deg. from the vertical and will be cleaned from the back or downstream side by rakes mounted on endless chains. The rakes will have a speed of 10 to 20 fpm. (feet per minute) and will be manually operated or automatically controlled by time-clocks. The capacity of each rack at the higher speed will be 18 cu. ft. of screenings per hour.

Screenings will be deposited on a sorting table at each rack, will be ground in hammermill grinders, and will be returned to the sewage stream below the racks. Each grinder will be capable of grinding 40 cu. ft. of screenings per hour.

Grit Channels. After screening the sewage will pass through six grit channels. Each channel will handle flows ranging up to 27 mgd. at a maximum velocity of 1.29 fps. (feet per second). The velocity of the sewage through the grit channels will be controlled by a fixed rectangular control section at the end of each channel. Channels will be of trapezoidal cross-section and will be equipped with grit-collecting equipment consisting of V-buckets mounted at 5-ft. intervals on endless chains. The collection equipment in each channel will have two speeds, namely 5 ft. per min. and 10 ft. per min., and will be capable of removing 50 cu. ft. of grit per hour from each channel at the higher speed.

Grit will be removed from the channels by the collection equipment and will be discharged onto a single belt conveyor serving all grit channels. The belt conveyor will discharge the unwashed grit to an 80 cu. ft. surge hopper whose function is to provide an even flow of unwashed grit to the grit washers. Spiral conveyors will move the grit from the surge hopper to two jig-type grit washers, each capable of washing 280 cu. ft. of grit per hour. Bucket elevators and spiral conveyors will then carry the washed grit to an overhead storage hopper having a capacity of 1,600 cu. ft. Grit will be discharged therefrom by gravity to trucks, for disposal as fill.

Plant effluent will be used for grit-washing purposes. The liquid overflow from the grit washers containing light organic material will be returned to the raw sewage at the influent end of the grit channels.

Preaeration Tanks. The sewage leaving the grit channels will be divided between two preaeration tanks. Each tank consists of two passes, each 16 ft. wide with a water depth of 13.5 ft. The total length of both passes is 328 ft. and the total theoretical aeration period at 47 mgd. is 30 minutes. Each tank is provided with swing-type diffusers capable of furnishing 1,630 cfm. of air, an amount equivalent to 0.10 cf. of air per gallon of sewage at 47 mgd. Air will be compressed by two 2,200 cfm. helical displacement blowers driven by 100 hp. engines, which will use either digester gas or gasoline for fuel.

Settling Tanks. The aerated sewage will flow from the preaeration tanks through aerated channels to four settling tanks, and will enter the tanks through a series of submerged ports. Aeration of the sewage in the channels is required to prevent deposition of solids under the wide variation in flows. Aeration equipment is similar to that provided in the preaeration tanks.

Each settling tank is rectangular in shape, 230 ft. long, 68 ft. wide, with an average water depth of 10.80 ft., and will provide a detention period of 2.5 hr. at 47 mgd. A picture of one of the tanks, taken during the construction period, is shown in Fig. 2.

The tanks are equipped with mechanism for removal of sludge, consisting of sludge rakes supported from a carriage moving on rails and pulled by cables. As the mechanism moves towards the effluent end of the tank, sludge will be deposited in windows along the tank bottom. On the return trip the rakes move the sludge to sludge hoppers at the influent end of each tank. Scum will also be moved to the influent end of each tank and will be deposited in a scum trough by the same mechanism. Reversals in direction of the mechanism are accomplished automatically by the use of torque-reversing switches and stops located at the ends of the track. The speed of the mechanism will be 4 ft. per minute with a complete cleaning cycle being made once every two hours.

Because the mechanism must remove scum for the entire length of tank it is necessary to remove the tank effluent through slots in the top of submerged effluent troughs. Each tank has 140 ft. of these slotted openings, which will pass all flows up to the daily maximum at a maximum rate of 146 gpm. per foot of slot. After passing through the slots the sewage will flow through the effluent troughs, through the end wall of the tank, into the collection channel, and over weirs into the effluent channel. These weirs maintain the water level in the

FIG. 2.—PRELIMINARY SETTLING TANK LOOKING TOWARD EFFLUENT END.

tanks. Flows greater than the daily maximum will be discharged directly into the tank effluent channel over rectangular weirs set in the end walls of the tanks.

Sludge will be removed from the hoppers continuously by diaphragm pumps located in the Sludge Pumping Stations, each hopper being served by a pump whose discharge can be varied between zero and 50 gpm. These pumps will discharge the sludge into an outside storage hopper at each station, from which it will be pumped periodically to the digestion tanks by electrode-controlled, heavy-duty, triplex, 200 gpm. plunger pumps. Scum from the settling tanks will be flushed directly into the outside storage hoppers and will be removed with the sludge to the digestion tanks.

Chlorination. The effluent from the settling tanks will be chlorinated before it enters the outfall conduit. Provision is also made to chlorinate the raw sewage before it passes through the bar racks at the entrance to the plant.

Liquid chlorine will be received at the plant in 16- or 30-ton railroad tank cars, for which purpose a railroad siding and platform with the necessary piping for unloading the chlorine has been provided. Liquid chlorine will flow under normal tank car pressure and by gravity, through a wrought-iron pipe from the unloading platform to the Chlorine Control Building, a distance of about 1,100 ft. Two evaporators, each rated at 6,000 lb. per 24 hours, will convert the liquid chlorine to chlorine gas, following which pressure-reducing valves will reduce the gas pressure to approximately 30 psi. The pressure-reducing valves will also shut off the flow of chlorine gas to the chlorinators in case of failure of the evaporators or chlorinators.

Two 6,000 lb. per 24 hour chlorinators will measure and regulate the flow of chlorine gas. The chlorinators are solution-feed, air-operated, fully automatic machines, and will feed chlorine in proportion to sewage flow. The machines have a range of 60 to 1 through the use of dual-float rotameters. In addition, a recording rotameter in lieu of platform scales, will record the flow of gas to both chlorinators.

The chlorine solution will be dispersed into the effluent by a rubber-lined-and-coated diffuser. Plant effluent will be used for solution purposes and for injector operation, and will be pumped to the chlorinators.

Outfall. After chlorination, the plant effluent will be carried in an 8-ft. by 10-ft. rectangular conduit to the outfall structure where it will be discharged into the Seekonk River through a 7-ft. and a 5-ft. diameter reinforced concrete outfall. The chlorine contact period at the outset at peak flow will be 4.5 min. and the outfalls will discharge the effluent below low tide level near shore. However, future plans call for extension of the outfalls to the edge of the ship channel, to provide a longer contact period and better dispersion.

Sludge Digestion Tanks. Sludge and scum from the settling tanks will be pumped to two primary digestion tanks having flat slab concrete roofs. Three draft tubes and high-speed propeller mixers will be installed in each tank for scum control.

The raw sludge will be measured by a venturi meter as it enters the Sludge Digestion Control Building and will be discharged into one of the three mixer draft tubes, which point was chosen to effect maximum dispersion of the raw sludge in the digesting sludge. The water level in the tanks will be maintained about 1 ft. 6 in. below

the bottom of the roof slab, by means of adjustable overflow rings. Provisions are made to withdraw supernatant liquor at two levels, to withdraw bottom sludge, and to recirculate the tank contents through external heat exchangers.

Normally sludge will be transferred by gravity from either primary tank to one of two secondary digestion tanks equipped with floating covers. Piping is provided to automatically waste supernatant and to permit withdrawal of sludge from these tanks. The cover level and gas pressure in the secondary tanks will be automatically recorded. An alarm will sound if the water level in the tanks falls to a point one foot above the corbels or if the gas pressure in the tanks falls below 1 in. of water.

The primary tank contents only will be maintained at 90 deg. Fahrenheit by means of two external spiral-type heat exchangers having 1-in. wide sludge passages and an individual heat transfer capacity of 1,000,000 Btu. per hour. Supernatant will be recirculated through each heat exchanger at the rate of 200 gpm. and hot water will be recirculated at the rate of 195 gpm. The hot water will be furnished by a hot-water boiler equipped with a combination gas-and-oil burner. Two screw-feed centrifugal pumps will be provided, each with a capacity of 300 gpm., for transferring sludge between tanks and for pumping sludge to the elutriation tanks.

The Control Building will be furnished with a combustible gas-alarm system which will automatically sound a warning if the concentration of sewage gas in the building exceeds 20 to 30 per cent of the lower explosive limit of an air-sewage gas mixture.

Digestion Tests. It is estimated that nearly one-third of the suspended solids in the raw sewage entering the plant, have their origin in an industrial plant producing roofing felt and impregnated building board. Consequently, tests were undertaken to determine the effect of these solids upon anticipated digestion rates and to determine the quantity and quality of gas produced from the digestion of a mixture of these solids in sewage sludge. Five pilot digesters were set up, each digester containing 1,500 milliliters of a digesting sewage sludge obtained for this purpose. Provisions were made on each digester for adding raw sludge daily and for collecting and measuring the gas produced. Tests carried out over a period of 94 days at room temperature—70 deg. Fahrenheit—gave the results given in Table 3.

All mixtures of sludge digested readily after the proper flora

TABLE 3.—RESULTS OF DIGESTION TESTS

Digester No.	Per cent sludge: Industrial waste	Per cent sludge: Sewage	Period of test, days	Time required to establish active digestion, days	Cu. ft. gas per pound volatile solids, added
1	0	100	56	1	10.4
2	25	75	56	1	9.3
3	50	50	94	32	8.0
4	75	25	94	61	6.2
5	100	0	94	63	5.2

had been established, although 63 days were required to establish active digestion in Digester No. 5. After the first 56 days Digester No. 1 was abandoned, and the feed to Digester No. 2 was changed to 100 per cent industrial sludge. Although digestion continued in Digester No. 2, the gas produced per pound of volatile solids added fell to 7.1 cf. after this change had been made.

The gas from all digesters contained approximately 27 per cent CO_2. The pH in all digesters fell from an initial value of 7.3 to 6.7 or 6.8 during the first month, and remained there for the duration of the test. As a result of these tests it was concluded that the solids from these industrial wastes were capable of digestion with sewage solids, although the volume of gas produced per pound of volatile solids added would be less than if the solids were normal sewage solids. Based on the preceding tests it is assumed that the gas production per day at the end of the design period will be 182,000 cft.

Storage and Use of Gas. Gas will be stored in floating gas holders on top of the secondary digestion tanks, each holder being capable of storing 3,780 cf. of gas at a pressure of 6 in. of water. Gas will be used as fuel for the blower engines and as heat for the primary digestion tanks. Excess gas will be wasted through two waste gas burners. All gas will be metered.

Sludge Elutriation. Two counter-current elutriation tanks are provided to wash the digested sludge with plant effluent prior to dewatering on the vacuum filters. It is anticipated that 4.5 gal, of plant effluent will be used to wash each gallon of digested sludge.

The procedure will involve mixing the unwashed sludge in the first-stage mixing tank with the overflow from the second-stage elutriation tank. The mixture will flow to the first-stage elutriation settling

tank, in which the partially washed sludge will be allowed to settle. The overflow from this tank will be discharged into the raw sewage in the Backstone Valley Interceptor.

The sludge from first-stage elutriation will be pumped to the second-stage mixing tank, where it will be mixed with fresh plant effluent. This mixture will then flow to the second-stage settling tank. The washed sludge from this tank will be pumped to conditioning tanks located at the vacuum filters.

Sludge Dewatering. Elutriated sludge will be dewatered on three 6-ft. diameter by 6 ft. vacuum filters. Each filter will be provided with its own sludge-conditioning tank, in which a solution of ferric chloride or ferric sulfate will be added to the sludge. The conditioning tanks are equipped with agitators and will provide about 1 min. of retention time.

The vacuum filters are of the coil-spring type, using stainless steel springs with synthetic rubber cores. These filters were chosen over blanket-type filters after an economic study indicated that their higher first cost was offset by their lower maintenance and operating costs. Moreover, although the presence of the industrial wastes in the sewage left the matter of solids loading on the filter somewhat in doubt, it was concluded that the coil-spring filter would be superior to a blanket-type filter in solids output per unit of filter area.

A vacuum pump of 720 cfm. capacity will provide the vacuum for filter operation. Filtrate will be discharged to the plant sewer through a barometric leg, without use of filtrate receivers or filtrate pumps.

The dewatered sludge will be deposited on an 18-in. wide belt conveyor and will be discharged to trucks for disposal at a sludge dump. Conveyor scales will be provided to weigh the dewatered sludge.

Provision has been made for a future sludge dryer and incinerator to be installed in a future dryer building.

Process Water. Water from a pond on the plant property will be pumped by a 100-gpm. well pump into the process water system. Pressure on the system will be maintained between 40 and 60 psi, by a hydropneumatic tank and air compressor. This water will be used for all fresh water requirements except sanitary uses and drinking water.

City Water. City water will be obtained from the East Provi-

dence Water Company. A meter vault is provided for housing the water meters.

Plant Effluent. Plant effluent will be used for elutriating digested sludge, operation of the chlorinator injectors, washing grit, screenings grinder operation, engine jacket water cooling, and for washing down the treatment units prior to final clean-up with pond water.

Two 400-gpm. and two 100-gpm. pumps at 85 ft. head will provide flushing water for all general purposes except chlorinator injector operation. For that purpose two 150-gpm. pumps rated at 125-ft. head will be furnished. Provision has also been made to use the higher head pumps to boost the pressure in the general-purpose system if necessary.

Power. Power will be obtained from the Narragansett Electric Company's substation on the plant property. The plant power system will operate on 550-volt, 3-phase, 60-cycle alternating current. No standby generating equipment is provided. All power lines will be carried in an underground system of ducts.

Service Building. The Sludge Dewatering and Service Building contains locker, toilet and lunch rooms for operating personnel. In addition, a completely-equipped machine shop is housed in this building, together with an adjacent office for a Maintenance Engineer, tool and parts storage space and automotive garage space.

Administration Building. The Administration Building provides office space for all of the Commission's adminstrative personnel as well as providing a meeting room for the Commission.

A modern laboratory for the control of the plant, including facilities for complete sanitary, chemical, and bacteriological tests, is housed in this building.

A meter and records room will contain a meter panel upon which all information required for evaluating the operation of the plant will be shown as a continuous record. Storage space for all records is provided in this room.

Organization and Acknowledgments. The Bucklin Point plant is being constructed by F. H. McGraw and Co., Hartford, Conn., under the detailed direction of the staff of the Blackstone Valley Sewer District Commission. Metcalf & Eddy were the designing engineers and are responsible for general supervision of construction.

The author desires to acknowledge the active cooperation of the

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Appendix I.
Bucklin Point Sewage Treatment Plant
Basic Design Data

1. *Year for design conditions.*	1950	1975
2. *Domestic population served.*	46,460	158,000
3. *Design flows in mgd.*		
a. average dry weather	20.2	47.2
(1) domestic sewage	7.04	17.82
(2) industrial waste	13.12	29.38
(3) per cent industrial waste	65	62
b. peak dry weather	50.4	118
c. storm peak	84	169
d. gallons domestic sewage per capita per day, average	152	112
4. *Sewage characteristics.*		
a. suspended solids		
(1) pounds per capita per day	0.382	0.264
(2) parts per million	106	106
b. 5-day B.O.D.		
(1) pounds per capita per day	0.710	0.398
(2) parts per million	196	160
5. *Parshall Flume.*		
a. Blackstone Valley Interceptor		
(1) width of throat, feet	6.0	
(2) maximum head at 128 mgd.	3.75	
b. East Providence Interceptor		
(1) width of throat, feet	3.0	
(2) maximum head at 41 mgd.	2.88	
6. *Fine Screens.*		
a. spacing between bars, in.	½	
b. method of cleaning	mechanical rake	
c. number of channels	3	6
d. width of channel, ft.	5.0	
e. maximum effective depth, ft.	5.53	
f. volume of rakings per mil. gal.,		
av., cu. ft.	3.0	
max., cu. ft.	10.0	
g. volume per day, av. cu. ft.	61	142
max., cu. ft.	840	1,690
h. weight per day, av., tons	1.52	3.55
max., tons	21.0	42.3
i. disposal of rakings	grinding and return to sewage	
j. number of grinders	3	6
k. capacity each grinder, lb. per hr.	1,800	

	1950	1975
7. *Grit channels.*		
a. velocity, fps.	0.56-1.29	
b. number of channels	3	6
c. width of channel, top, ft.	7.5	
d. width of channel, bottom, ft.	3.5	
e. depth of grit space, ft.	1.5	
f. max. water depth, above grit space, ft.	5.35	
g. length of channel, ft.	88.0	
h. quantity of grit per mil. gal.,		
av., cu. ft.	4.0	
max., cu. ft.	40.0	
i. quantity of grit per day,		
av., cu. ft.	81	188
max., cu. ft.	3,360	6,720
j. tons of grit per day, av.	4.0	9.4
max.	168	336
k. method of cleaning channels	bucket scrapers, elevators, and conveyors	
l. method of grit removal	washing and disposal as fill	
m. number of grit washers	1	2
n. capacity per washer, cu. ft./hr.	280	
o. grit-storage capacity, cu. ft.	1,600	
p. grit channel dewatering pumps	1	1
q. capacity per pump, gpm.	100	
r. maximum operating head, ft.	100	
s. type of pump	plunger, duplex	
8. *Pre-aeration tanks.*		
a. number of tanks	1	2
b. detention per tank, av. flow, min.	35	30
max. flow, min.	8	8
c. capacity per tank, cu. ft.	66,000	
d. channels per tank	2	
e. channel length per tank, ft.	328	
f. channel width, ft.	16.0	
g. water depth, av. flow, ft.	13.5	
h. air		
(1) cfm. per tank	1,630	
(2) cfm. per ft. of channel	5.0	
(3) cu. ft. per gal. sewage, av.	0.116	0.100
i. type of equipment	"swing" diffusers with seran tubes	

	1950	1975
9. *Aerated channels*		
a. number of channels	1	2
b. location	aeration tanks to preliminary settling tanks	
c. length, each, ft.	142	
d. width, ft.	7.0	
e. water depth, av. flow, ft.	6.8	
f. air		
(1) cfm. per channel (includes grit channel effluent)	570	
(2) cfm. per ft. of channel	3.0	
(3) cu. ft. per gal. sewage, av.	0.041	0.035
10. *Blowers, air.*		
a. number of blowers	2	3
b. capacity, each, cfm, at 7 psig.	2,200	
c. method of driving	gas-gasoline	
11. *Preliminary settling tanks.*		
a. number of tanks	2	4
b. detention period at av. flow, hr.	2.9	2.5
c. capacity per tank, gal.	1,230,000	
d. width of tank, ft.	68.0	
e. length of tank, ft.	230.0	
f. av. water depth, ft.	10.80	
g. overflow rate, gal./sq. ft./day		
(1) average flow	645	755
(2) maximum daily flow	1,620	1,890
(3) peak	2,690	2,690
h. method of sludge removal	rakes to sludge hoppers	
12. *Primary sludge.*		
a. total suspended solids removed, lb./day, dry basis, av.	10,970	25,700
b. volatile suspended solids removed, lb./day, dry basis, av.	8,540	19,700
c. equivalent population (0.20 lb. per capita per day)	88,600	210,000
d. sludge		
(1) moisture, per cent	96	
(2) gal. per day, av.	33,000	77,000
(3) gal per day, max.	79,000	185,000
13. *Sludge withdrawal pumps.*		
a. number of pumps per battery (2 tanks)	4	
b. unit capacity of pump, gpm.	0-50	
c. approximate discharge head, ft.	0	
d. type of pump	diaphragm, simplex	

	1950	1975
14. *Sludge booster pumps.*		
a. number of pumps per battery (2 tanks)	2	
b. unit capacity of pump, gpm.	200	
c. maximum operating head, ft.	115	
d. type of pump	plunger, triplex	
15. *Primary and pre-aeration tank dewatering pumps.*		
a. number of pumps per battery (2 tanks)	1	
b. unit capacity of pump, gpm.	300	
c. total dynamic head, ft.	40	
d. type of pump	screw impeller	
16. *Sludge digestion tanks.*		
a. number of tanks	2	4
b. total sludge capacity, cu. ft.	193,000	386,000
c. sludge capacity		
(1) cu. ft./lb. dry solids per day	17.5	15.0
(2) cu. ft./equivalent capita	2.17	1.84
d. primary tanks with fixed concrete roofs		
(1) number of tanks	1	2
(2) total sludge capacity, cu. ft.	129,000	258,000
(3) diameter, ft.	75	
(4) sidewall water depth, ft.	26.0	
(5) depth of inverted cone, ft.	9.3	
(6) number of mixers per tank	3	
(7) capacity of mixers, ea., gpm.	7,500	
(8) rate of turnover, min.	43	
e. secondary tanks with floating covers		
(1) number of tanks	1	2
(2) total sludge capacity, cu. ft.	64,000	128,000
(3) diameter, ft.	70	
(4) sidewall water depth, ft.	13.75	
(5) depth of inverted cone, ft.	8.75	
(6) gas storage per tank at 6-in. water pressure, cu. ft.	3,780	
f. gas production		
(1) cu. ft./lb. volatile solids/day	8.5	9.3
(2) cu. ft. per day	72,500	182,000
(3) cu. ft./equivalent capita	0.82	0.87
g. digested sludge solids		
(1) reduction of volatiles by digestion, per cent	60	
(2) lb./day, dry basis, av.	5,840	13,900
(3) moisture, per cent	94	
(4) gal. per day	11,700	27,800

	1950	1975
h. digestion tank dewatering pumps		
(1) number of pumps	1	2
(2) unit capacity of pump, gpm.	300	
(3) total dynamic head, ft.	44	
(4) type of pump	screw impeller	
i. digester heating		
(1) type of heating	external heat exchanger	
(2) number of heat exchangers	1	2
(3) capacity, ea., Btu./hr.	1,000,000	
(4) boiler capacity, output, gross Btu./hr.	1,580,000	675,000
(5) capacity sludge pump per heat exchanger, gpm.	200	
(6) capacity hot-water pump per heat exchanger, gpm.	195	
(7) type of pumps	centrifugal, non-clog	
17. *Digested sludge elutriation.*		
a. operation		
(1) hr. per day	4.8*	17.2**
(2) operating days per week	5	5
b. digested sludge		
(1) per cent moisture	94	94
(2) solids, lb./operating day, av.	8,200	19,500
(3) solids, lb./operating day, max.	10,250	24,400
(4) sludge, gal./operating day, av.	16,100	38,200
(5) sludge, gal./operating day, max.	20,200	47,800
(6) sludge, gpm., av.	56	37
(7) sludge, gpm., max.	70	46
c. diluting water		
(1) number of dilutions	3	4.5
(2) gal. per operating day	48,300	173,000
(3) gal. per minute	168	168
d. mixed liquor		
(1) gal. per operating day, av.	64,400	211,200
(2) gal. per operating day, max.	68,500	220,800
(3) gal. per minute, av.	224	205
(4) gal. per minute, max.	238	214
e. mixing tanks		
(1) number of units	2	
(2) capacity, ea., gal.	710	710
(3) detention period, min., av. flow	3.2	3.5
(4) detention period, min., max. flow	3.0	3.3
(5) type of equipment	picket fence	

*Dewatered sludge hauled to dump for disposal
**Dewatered sludge disposed of by incineration

	1950		1975
f. separating tanks			
(1) number of units		2	
(2) detention period per tank, hr. at max. rate	3.8		4.2
(3) capacity per tank, cu. ft.		7,300	
(4) diameter, ft.		30	
(5) side water depth, ft.		9.5	
(6) average water depth, ft.		10.3	
(7) overflow rate, gal./sq. ft. per day, av.	455		415
(8) overflow rate, gal./sq. ft. per day, max.	485		435
(9) type of equipment		picket fence	
18. *Digested sludge pumps for elutriation system.*			
a. number of pumps		4	
b. capacity, gpm. per pump		30-90	
c. maximum operating head, ft.		60	
d. type of pump		plunger, simplex, varidrive	
e. service			
(1) digested sludge to first-stage elutriation tank			
(2) first-stage elutriated sludge to second-stage elutriation tank			
19. *Elutriated sludge pump.*			
a. number of pumps		1	
b. capacity, gpm.		30-90	
c. maximum operating head, ft.		100	
d. type of pump		plunger, triplex, varidrive	
e. service			
(1) pump has common suction and separate discharge pipes from each cylinder to the individual filters.			
20. *Sludge filtration.*			
a. operation			
(1) hr. per day	4.8		17.2
(2) days per week	5		5
b. elutriated sludge			
(1) solids, lb./operating day, av.	8,200		19,500
(2) solids, lb./operating day, max.	10,250		24,400
c. ferric chloride conditioning			
(1) dosage, percent		3	
(2) lb./operating day	246		585
(3) gal. (40%) solution per operating day	52		124
(4) solution storage, gal.		1,500	
d. vacuum filters			
(1) number of units		3	
(2) diameter, ft.		6.0	
(3) length, ft.		6.0	

	1950		1975
(4) sq.-ft. area per unit		114	
(5) av. loading, lb./sq. ft./hr.		5.0	
(6) number of filters in operation per day	3		2
(7) vacuum pump capacity required, cfm.		685	
e. sludge cake conveyor			
(1) average load			
(a) lb./hr., dry basis	1,710		1,140
(b) lb./hr., wet cake (75% water)	6,820		4,560
(2) maximum load			
(a) lb./hr., dry basis		2,740	
(b) lb./hr., wet cake (75% water)		11,000	
21. *Sludge incineration.*			
a. average load			
(1) lb./hr., dry basis	—		1,140
(2) lb./hr., wet cake (75% water)	—		4,560
b. maximum load			
(1) lb./hr., dry basis	—		1,830
(2) lb./hr., wet cake (75% water)	—		7,330
c. ash			
(1) % volatile matter in sludge	—		58
(2) lb./day, av., dry basis	—		8,250
(3) lb./day, av., (4% water)	—		8,600
22. *Effluent pumps.*			
a. general usage			
(1) number of pumps		2	
(2) capacity, ea., gpm.		400	
(3) total dynamic head, ft.		80	
(4) type of pump		centrifugal, non-clog	
(5) service—elutriation, grit washing, screenings grinding, flushing water			
b. engine cooling water			
(1) number of pumps		2	
(2) capacity, ea., gpm.		100	
(3) total dynamic head, ft.		85	
(4) type of pump		centrifugal, non-clog	
c. chlorination system			
(1) number of pumps		2	
(2) capacity, ea., gpm.		150	
(3) total dynamic head, ft.		125	
(4) type of pump		centrifugal, non-clog	

	1950		1975
23. *Chlorination.*			
a. demand at av. dry-weather flow			
(1) max. chlorine demand, ppm.		18	
(2) max. chlorine demand, lb./day	4,900		7,100
(3) av. chlorine demand, ppm.		9	
(4) av. chlorine demand, lb./day	2,450		3,550
b. chlorinators			
(1) number of chlorinators		2	
(2) capacity of each unit, lb./day		6,000	
c. evaporators			
(1) number of evaporators		2	
(2) capacity of each unit, lb./day		6,000	
d. liquid chlorine container		16 or 30-T tank cars	

Appendix II

List of Manufacturers Supplying Equipment to Bucklin Point Sewage Treatment Plant

Building	*Equipment*	*Manufacturer*
Screen and Grit Building	Bar racks	Jeffrey Mfg. Co.
do.	Grinders	" " "
do.	Grit collection, conveying & washing equipment	" " "
do.	Gas engines	Climax Engine & Pump Mfg. Co.
do.	Blowers	Standard Stoker Co.
do.	Air filters	Dollinger Corp.
Preaeration Tanks	Air diffuser assemblies	Chicago Pump Co.
Settling Tanks	Sludge collectors	Dorr Company
Chlorination Building	Chlorinators	Builders-Providence, Inc.
" "	Evaporators and reducing valves	Wallace & Tiernan
" "	Chlorine recorders	Schutte & Koerting and Foxboro Co.
" "	Plant effluent pumps	Fairbanks-Morse & Co.
Pumping Stations	Diaphragm sludge pumps	Dorr Company
" "	Plunger sludge pumps	Carter Pump Co.
" "	Screw-feed pump	Chicago Pump Co.
Digestion Tanks	Mixers	Dorr Company
" "	Floating covers	" "
" "	Heat exchangers	American Heat Reclaiming Corp.
" "	Sludge transfer pumps	Chicago Pump Co.
" "	Sludge recirculation pumps	Fairbanks-Morse & Co.
" "	Gas control equipment	Vapor Recovery Systems Co.
" "	Gas alarm system	Davis Instrument Co. of New England
Elutriation Tanks	Mixers and thickeners	Dorr Company
Dewatering Building	Elutriation pumps	Carter Pump Co.
" "	Vacuum filters and accessories	Komline-Sanderson Engineering Corp.
	Miscellaneous Equipment	
	Meters and instruments	Builders-Providence, Inc.
	Sluice gates	Rodney-Hunt Machine Co.
	Electric switchgear	General Electric Co.
	Well pump	Peerless Pump Co.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

APRIL 15, 1952.—A Joint Meeting of the Boston Society of Civil Engineers with the Northeastern Section of the American Society of Civil Engineers, was held this evening at Northeastern University, 360 Huntington Avenue, Boston, Mass.

President, Oscar S. Bray of the Northeastern Section, American Society of Civil Engineers, presiding at the Joint Meeting, called upon President Emil A. Gramstorff, of the Boston Society of Civil Engineers to conduct any BSCE matters of business necessary.

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

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

Grade of Member.— Matthew H. Chamberlain, *Robert S. Kleinschmidt, Eugene B. Lunden, Frederick H. Paulson, Oliver E. Strong, Merit P. White.

Grade of Junior.—Stanley F. Gesek, William J. Kerins, *Charles F. Kennedy.

*Transfer from Grade of Student

President Gramstorff announced that the next meeting of the BSCE would be held on May 21, 1952, at the American Academy of Arts & Sciences, Mr. Harrison P. Eddy, Jr., Partner of Metcalf & Eddy would address the Society on "Sanitary Engineering Experience in Hawaiian Islands".

President Oscar S. Bray then introduced the speaker of the evening, Mr. H. S. Spitz, Assistant Chief Engineer, Construction Engineering Bureau, U. S. Steel Company, who gave a most interesting illustrated talk on "The New Fairless Works".

The dinner was attended by one hundred five members and guests and one hundred fifty six attended the meeting.

The meeting adjourned at 9:20 P.M.

ROBERT W. MOIR, *Secretary*

MAY 21, 1952.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts & Sciences, 28 Newberry Street, Boston, Mass., and was called to order by President Emil A. Gramstorff.

President Gramstorff announced that the reading of the minutes of the previous meeting held April 15, 1952 would be waived unless there was objection.

It was *VOTED* "to dispense with the reading of the minutes of the April 15, 1952 meeting".

President Gramstorff announced the death of the following members:—

Arthur W. Dean, who was elected a member December 18, 1895 and who died March 20, 1952.

Charles F. Dingman, who was elected a member January 27, 1915 and who died March 3, 1952.

John O. Miller, who was elected a member April 20, 1927 and who died March 7, 1952.

Bernard S. Rose, who was elected a member November 17, 1909 and who died April 12, 1951.

Frank H. Morris, who was elected a member September 16, 1896 and who died March 17, 1952.

Lewis J. Johnson, who was elected a member May 20, 1801 and who died April 15, 1952.

Harvey C. Harris, who was elected a member March 25, 1925 and who died April 9, 1952.

Howard E. Bailey, who was elected a member November 16, 1927 and who died May 9, 1952.

Ernest E. Lothrop, who was elected a member May 17, 1916 and who died March 6, 1952.

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

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

Grade of Member:—Harl P. Aldrich, Jr., Carmine N. Ciriello, William R. Colton, Elmer G. Dana, William A. Fisher, William J. Hallahan, Carroll I. Johnson, Grant H. Potter, Donald P. Rand, Charles F. Sullivan.

President Gramstorff stated that a Petition to BSCE for a Resolution concerning Professional Practice as Relating to Competitive Bidding, signed by five members of the Society was considered by the Board of Government at its meeting on April 7, 1952, and that this present meeting had been designated by the Board for a discussion and such action as the members wish to take, in accordance with provisions of the By-Laws. (Notice of such action circulated to members by letter dated May 5, 1952.)

Motion was made and it was so *Voted* "that action on this Resolution be deferred until such time as a larger representation of the members is present".

President Gramstorff announced that the Board of Government at its meeting held on May 19, 1952 voted to participate as a co-sponsor of the "Third Conference on Coastal Engineering", to be held at the Massachusetts Institute of Technology, October 22-24, 1952.

President Gramstorff introduced the speaker of the evening, Mr. Harrison P. Eddy, Partner, Metcalf & Eddy, who gave a most interesting talk on "Sanitary Engineering Experiences in Hawaiian Islands". After Mr. Eddy's talk, a very interesting travelogue movie of Hawaii, in color, was presented through the courtesy of United Air Lines.

The meeting adjourned at 9:15 P.M. and the members gathered in the Lounge where a collation was served.

Sixty three members and guests attended the meeting.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

MARCH 5, 1952.—A meeting of the Sanitary Section was held at 7:15 P.M., at the Society Rooms after an informal dinner at Patten's Restaurant. Ninety-five members and guests attended the meeting. Forty members and guests attended the informal dinner.

Chairman William E. Stanley presided. The annual report was read by Clerk A. C. Bolde. It was approved as read and filed. George F. Brosseau, Chairman of the nominating committee,

presented the following list of nominees for officers and other members of the Executive Committee to serve until the next annual meeting of the section in March, 1953. Chairman, Fozi M. Cahaly, Vice-Chairman, John S. Bethel. Jr., Clerk, Ariel A. Thomas, Executive Committee, C. Hugo Bergman, Edward W. Moore, Clair N. Sawyer.

The nominations were closed and the clerk cast one ballot for the nominees as presented.

Several papers and a prepared discussion on "Boston Harbor Pollution" were presented as follows: "The Pollution Problem of Boston Harbor and Tributaries", by Edgar F. Copell, for Arthur D. Weston; "Proposed Remedial Works", by Frank Morgenrath; "Chlorination of Storm Water Overflow", by Dr. Rolf Eliassen and James C. Lamb III (read by Dr. Eliassen). "Prepared Discussion" by Thomas A. Berrigan.

A lively discussion of the relative merits of tunnels and gravity sewers and the effectiveness of the proposed storm water treatment tanks near Boston University Bridge followed the prepared papers.

The meeting adjourned at 9:30 P.M.

ARIEL A. THOMAS, *Clerk*

MAY 17, 1952.—The Bucklin Point Sewage Treatment Plant of the Blackstone Valley Sewer District Commission was inspected at 10:00 A.M., at East Providence, Rhode Island. Approximately 65 persons attended.

A creamed turkey dinner was served to 50 at the Wannemoisett Country Club, followed by several short talks.

1. William J. Halloran, Chairman, BUSDC, welcomed the group and discussed the Commission's activities.

2. John S. Bethel, Jr., Project Engineer for Metcalf and Eddy, described the plant units.

3. Charles Hammann, Chief Engineer, BUSDC, discussed unusual features of the Commission and the plant.

Chairman Fozi M. Cahaly presided and introduced the guests and officers at the head table.

The outing adjourned at 4:30 P.M. after the inspection of the Omega Sewage Pumping Station.

ARIEL A. THOMAS, *Clerk*

STRUCTURAL SECTION

MAY 14, 1952.—A meeting of the Structural Section was held at the Society Rooms with Chairman Edward C. Keane presiding. Mr. J. R. Thompson and Mr. H. A. Vanderbeek, both of the American Bridge Division, U. S. Steel Company, spoke on various aspects of the United Nations Secretariat Building. Mr. Thompson discussed some financial problem connected with the job, and Mr. Vanderbeek presented details of design, fabrication and erection of the structural steel. The talks were followed by a Kodachrome movie showing demolition of the buildings which previously occupied the site preparation of the foundations, as well as fabrication and erection of the structural steel.

The attendance was 52.

RUTH D. TERZAGHI, *Clerk*

TRANSPORTATION SECTION

FEBRUARY 27, 1952.—Following a small dinner gathering at Patten's Restaurant, the meeting was called to order by Chairman Ernest L. Spencer, at 7:30 P.M.

The minutes of the previous meeting held on November 28, 1952 were read and approved.

The report of the Nominating Committee was given by the Chair. Their report nominated the following officers for 1952; Chairman, H. Lowell Crocker; Vice-Chairman, Hugh P. Duffill; Clerk Herman G. Protze; Members of the Executive Committee Ernest L. Spencer, Joseph F. Willard, Stanislaw J. V. Gawlinski.

On motion from the floor, duly seconded and voted, the chair was instructed to cast one ballot for the above officers.

It was further voted that due to the small attendance occasioned by a raging blizzard outside, the meeting be adjourned and the speaker be asked to address a future meeting.

ERNEST L. SPENCER, *Clerk pro-tem*

HYDRAULICS SECTION

MAY 3, 1952.—An inspection trip of the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute was held at Holden, Massachusetts. The group met at the laboratory at 10:00 a.m. and was called to order by the Vice-Chairman, L. W. Ryder in the absence of Chairman McCoy.

The meeting was immediately turned over to Professor Leslie J. Hooper, Director of the laboratory who discussed briefly the history of the laboratories and the work in progress. Experimental work under way for the Ordnance Department of the U. S. Army was described by Mr. Robert Henderson, an assistant at the laboratory, and work under way on civilian test programs was described by Mr. Lawrence Neale, also an assistant at the laboratory.

The group visited the projectile launching test and then was broken into three sub groups. The facilities of the laboratory and the experiments in progress were demonstrated to and discussed with the sub groups.

A catered luncheon was served at 12:45, and meeting adjourned at 1:45.

A total of thirty-one were present including Professor Hooper and his two assistants.

ARTHUR T. IPPEN, *Clerk*

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