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INTRODUCTION TO COMPREHENSIVE SANITARY SURVEY OF WOONSOCKET, RHODE ISLAND

BY CHARLES G. HAMMANN*

[Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on October 5, 1955.]

THE City of Woonsocket is highly typical of many other New England communities, particularly those in which the basic industry is textile in character or those in which the economy is dependent upon one type of industry. Similarities in geography, topography, history, economy and the like are all matters with which I am certain this audience is familiar. Accordingly, no purpose would be served by discussing them in detail at this time. Rather, this portion of the presentation will be restricted to such introductory material as is considered essential to an understanding of the technical data to be presented immediately following.

Woonsocket is the fourth largest municipality in Rhode Island. It is located in the extreme northeastern section of the State, its northern boundary being the Massachusetts-Rhode Island line. The City is approximately 8.8 square miles in area and has a population slightly in excess of 50,000. It is bisected by the Blackstone River which, together with several tributary streams, drains the entire community.

Historical documents indicate that a settlement existed at the present site in 1700 if not somewhat earlier. However, its incorporation as a town did not occur until 1867 at which time that portion of the City which presently lies easterly of the Blackstone River was "set off" from the Town of Cumberland. The portion westerly of the river was acquired from the Town of Smithfield in 1871. Status as a city was attained in 1888.

*Director of Public Works, Woonsocket, R. I.

Industrial and commercial enterprise appear to have commenced in 1712 with the establishment of a corn and fulling mill. Between then and 1720, a forge was constructed followed by a scythe factory. The first cotton mill was erected about 1822 and another in 1827. The manufacture of woolen goods was begun in 1831 by Edward Harris whose name subsequently became prominent throughout the United States.

It is certain that these interests as well as the community in general derived impetus and benefit from the Blackstone Canal between Providence and Worcester which was opened in 1828. However, chief growth and accelerated prosperity occurred with the advent of the Providence and Worcester Railroad which began operating in 1847. With the exception of fluctuations common to the area or the nation, development continued at a favorable rate until 1929 when the impact of depression affected the entire country. As in many other New England cities and towns, economic recovery lagged behind that experienced elsewhere due primarily to lack of industrial diversification. To offset this situation, the Industrial Foundation of Greater Woonsocket was created in 1954 and, since that time, has been vigorously engaged in attracting new and diversified industry.

With respect to the administration of its affairs, Woonsocket seems to have fared normally both as a town and, until sometime after the turn of the century, as a city. In this regard, it is emphasized that it is not intended that anyone shall be criticized nor discredited in this presentation. Therefore, no effort will be made to trace developments in this respect nor to be precise concerning timing. It is sufficient to note that the local government retrogressed seriously and the situation worsened progressively. By the late forties, the City had become notorious for the vice and corruption which prevailed within its limits.

Of lesser importance, perhaps, but still highly significant, was the dereliction which prevailed simultaneously with respect to the City as a physical entity. With the exception of a sewage treatment plant completed in 1930 as a result of pressure exerted by State health authorities, the disregard and neglect of public facilities during this era were regrettable to say the least.

Schools not only had become outmoded and inadequate but hazardous conditions existed in many instances. Fire stations had become antiquated and were improperly located in relation to community expansion. Paved highways had been allowed to deteriorate and many miles of otherwise developed streets remained undrained, unpaved, or both. The entire water system had become inadequate. No action had been taken to supplement the impounded supply known for several years to have been grossly insufficient. Urgently needed treatment works never had been developed. Storage and distribution facilities had been neglected to the point where they were incapable of safely meeting quantitative and fire protection requirements. The sewerage system had not been expanded in keeping with structural development and appreciable portions of the City remained unsewered. Moreover, the system was incapable of receiving wastes from many industrial establishments faced with compliance with State pollution control laws under prohibitive circumstances. The previously mentioned sewage treatment plant had become ineffective in that only a portion of the domestic sewage and practically none of the industrial wastes originating within the community could be purified. No accomplishments had been realized with respect to providing modern and sanitary means of refuse disposal in lieu of the open dump which had been a source of justifiable complaints for many years. Nothing had been done to relieve traffic congestion and lack of parking facilities, both of which had become problems of tremendous import.

Faced with these and other undesirable conditions, a finally aroused public went to the polls in 1952 and overwhelmingly approved a new home rule charter, the first such document in the history of Rhode Island, and elected a reform administration. Pursuant to the provisions of that charter, the first non-partisan, off-year election ever held was conducted in April of 1953. Essentially the same administration was returned to office.

As soon as practicable after its inauguration on May 1, 1953, the administration approved the following transactions which were executed by the municipal agencies responsible for the respective functions:

1. An independent tax revaluation survey which was to be the basis of a tax assessment equalization as well as the foundation for a revised financial structure.
2. A study of all facilities relating to fire protection by the National Board of Fire Underwriters.

3. A comprehensive study of requirements essential to restore the physical plant of the Education Department to modern standards.
4. An investigation and report of improvements needed in the municipal highway system.
5. A traffic and parking survey (Federal-aid project).
6. A comprehensive study and recommendations concerning sanitation requirements including water, sewer and refuse disposal facilities.

Each of the foregoing actions have been completed and the respective reports have been presented to the City. In keeping with charter provisions, the results are being considered by the Planning Board which must approve all capital improvements. Present indications are that the Board will approve and present for legislative action a capital program based upon the recommendations contained in these reports and consistent with the financial capacity of the community. Unfortunately, the devastating impact of the recent flood may have an adverse effect upon the rate of progress but it is generally felt that the necessary work will be carried out expeditiously and, equally important, that it will be predicated upon sound financial and engineering principles.

At this time, I yield to John S. Bethal, Jr., Partner, Metcalf & Eddy, who will discuss in greater detail the study made and conclusions reached by that firm with respect to the sanitation needs of Woonsocket.

COMPREHENSIVE SANITARY SURVEY OF WOONSOCKET, RHODE ISLAND

By John S. Bethel, Jr.*, Member

As Mr. Hammann has stated in the first portion of this combined presentation, most of the industries in Woonsocket are related to textiles in some form. At present Woonsocket is undergoing a period of industrial transformation similar to that being experienced in many other northern textile centers, and it is not possible to predict the type of industries that Woonsocket will support in the future. However, the trend throughout the New England area appears to be away from soft-goods industries which require large volumes of water and produce large quantities of wastes, to hard-goods industries requiring little water and producing small volumes of wastes. This transformation was important in our study of water and sewerage problems.

Population Studies

The prediction of future populations of a city is generally based upon statistical methods. These methods are usually successful if there are within the City's limits large areas for industrial expansion and housing development; if the City has shown an increase in population over the years; and if the growth of the city can be compared to the growth of larger cities having similar industries and occupations. However, the problem of predicting the future population of Woonsocket could not logically be solved by usual methods for the following reasons: the areas available for future industrial and housing expansion are limited; the population of Woonsocket has been static for nearly 30 years at 50,000 persons, with only minor fluctuations during the depression of the 1930s and later during the years of industrial activities stimulated by World War II; and finally, no historical data are available for estimating future growth based upon comparison with other New England cities which have lost a large number of their industries and are in the process of attracting new ones. Because of the inapplicability of these conventional methods,

*Partner of Metcalf & Eddy, Boston, Mass.

it was necessary to conduct a detailed study of the present density of population of the various areas of Woonsocket and to estimate the ultimate population of each of these areas. A visual survey of the City was made in which built-up and vacant areas were noted. This information was checked against aerial photographs. Park, industrial, and similar areas unavailable for construction of housing were omitted from the density calculations.

Also considered in this study were types of housing. In the early part of the present century, the multiple dwelling unit was the more common form of housing in Woonsocket. However, after the first World War, and even more markedly after the second World War, the trend has been to single dwelling units. In those unpopulated portions of the City where the available land exceeded 100 vacant acres per precinct, it was assumed that the future density would be approximately 8.4 persons per acre. This density was based on the assumption that a new small house might occupy approximately 10,000 sq. ft. of land and provide housing for 3.6 persons. Where there were less than 100 vacant acres available in a precinct, it was assumed that the future development would approximate the present density. On these bases, it was estimated that the ultimate population of Woonsocket might be approximately 70,200 persons.

The area density method of determining future population does not give one important factor required for engineering design, namely, the year in which the future population will be realized With the apparent static population of Woonsocket, there is little reason to believe that under present conditions there will be any rapid growth in the next 10 years. Under no conditions, would it appear reasonable to assume an increase of 1,000 persons per year, which rate is equal to the present natural increase in population. It was, therefore, estimated that Woonsocket would not attain a population of 70,200 persons prior to the year 2000, or 45 years hence, and barring additional industrial expansion might never reach this population.

Taking into consideration the various factors previously discussed, it was also estimated that the population of Woonsocket might be approximately 55,000 by the year 1965, an increase of about 5,000 over the present population. In general, therefore, this population was used for the design of those works which could be expanded readily, whereas the ultimate population was used in the design of those works which would be difficult or costly to expand.

Water Consumption Supply and Treatment

By far the most pressing problem facing the City is that of water supply. This supply, barely adequate for many years, is now critically inadequate. Previous studies made in 1925 and 1943 stressed the inadequacy of the supply and in the earlier report, a recommendation of additional sources of surface water which might be available to the City was presented. Unfortunately, no action was taken on these reports and the problem today remains unchanged, except that at present the need for an augmented supply is infinitely more acute than it was in 1925. The seriousness of the problem was demonstrated in 1930 and again in 1953 when the reservoir supplying water to the City was nearly depleted.

In addition to the problem of development of an additional water supply, Woonsocket is faced with a second problem relative to the quality of the supply. The color, odor, taste, and iron content of the water do not meet the U. S. Public Health Service standards recommended for drinking water. Moreover, the pollution of the watershed of the present supply is such that adequate treatment of the water should be undertaken to protect the health of the people of the City.

Quantity of Water. Before undertaking any study of additional water supplies available for development, it was necessary to estimate the future quantities of water needed. A study of the past consumption of water, both domestic and industrial, was therefore made.

Past records indicate that during the years 1925 to 1953 water consumption increased from approximately 3 mgd. to 5.3 mgd., a rise of nearly 80 per cent. However, as the population during that period remained substantially constant, the increase in consumption was the result of an increase in per-capita consumption from 52 gcd. to 80 gcd. In our opinion, this is the result of a generally improved standard living as well as the trend from apartment-type housing to single family dwellings. It was estimated, therefore, that the domestic consumption will continue to increase to 90 gcd. by 1965 and to 100 gcd. by the year 2000.

Concurrent with this study of domestic water consumption an investigation of the water needs of all industries in Woonsocket was also conducted. The industrial consumption was divided into two categories, namely, the amount required for "normal production," and that required for "capacity production." We estimated that in 1965 the industrial consumption would be equal to that required for the present "normal production" and that by the year 2000 it would be equal to the present "capacity production."

On the basis of these assumptions, we estimated that the total water requirements of the City will be 6.5 mgd. in 1965, provided industrial demands remain at their present level; and will be 9.5 mgd. when the population of Woonsocket has reached 70,200, providing industrial demands at that time equal the demand of present industries operating at capacity levels. Further details of water consumption are given in Table 1. The average daily consumption was used

TABLE 1
WATER CONSUMPTION

Item	*Year* 1954	1960	1965	2000
Population of Woonsocket	50,200	52,600	55,000	70,200
Gallons per capita per day	80	85	90	100
Industrial Production	Normal	Normal	Normal	Capacity
Water demand in mgd.				
Woonsocket—domestic	4.03	4.47	4.95	7.02
Woonsocket—industrial	1.04	1.15	1.29	2.05
Villages outside City limits	0.26	0.28	0.30	0.40
Total average consumption	5.33	5.90	6.54	9.47
Max. 4-month demand	6.2	6.9	7.6	11.0
Max. daily demand	8.6	9.5	10.5	15.2

to determine the quantity of additional water required; whereas the maximum daily consumption was used to determine the capacity of the proposed water treatment plant and the necessary additions to the major pipes of the distribution network.

Crookfall Brook Supply. The present water supply for the city of Woonsocket is obtained from Crookfall Brook, a tributary of the Blackstone River. Crookfall Brook has a watershed area of 7.9 sq. mi. and is located just south of the City in the towns of North Smithfield, Smithfield, and Lincoln. The watershed area is wooded and sparsely settled, except along the Louisquissett Pike which has been lightly developed in recent years. There is some swamp area on the upper reaches of the watershed, but on the lower portion there are large areas of swamp along the course of the brook.

When the water supply system was taken over by the City from the Woonsocket Water Company in 1885, there were two small reservoirs on the lower part of the watershed, namely, Reservoir No. 1, a small reservoir of 30 mil. gal. which is still used as a storage reservoir for the present pumping station; and Reservoir No. 2, a short distance upstream, which was destroyed and was never rebuilt. In 1895 a third and major reservoir, Reservoir No. 3, was constructed approximately 2½ miles upstream from Reservoir No. 1. Fig. 1 shows the general interrelation of these reservoirs.

Reservoir No. 3 has a safe available storage of 890 mil. gal. and is contained by an earth-fill dam with a masonry spillway. The capacity curve for this reservoir was computed from a copy of the original contour map drawn at the time of the construction of the reservoir.

During the drier months of the year when the runoff from the watershed of Reservoir No. 1 is insufficient to supply the demands of the City, water is released from Reservoir No. 3 and allowed to flow through 2½ miles of swamp to Reservoir No. 1. During the remainder of the year Reservoir No. 3 is kept full or as nearly full as possible.

The water level in Reservoir No. 1 is normally kept about 2 ft. below the spillway crest so that runoff from heavy rains on the lower watershed can be stored and used.

Safe Yield of Crookfall Brook Watershed. Safe yield is defined as the daily quantity of water which is continuously available from a supply under the lowest rainfall conditions likely to occur. The determination of safe yield is based on three factors; namely, amount of rainfall, runoff resulting from rainfall, and available reservoir storage. Many times these data are not available for a particular watershed, as was true in the case of Woonsocket, where the rate of runoff resulting from rainfall was unknown. Under these conditions, the rainfall and unit runoff from a similar nearby watershed are commonly used. In our studies, data from the Abbott Run watershed on the Pawtucket water supply, located approximately 10 miles from Crookfall Brook was used. Rainfall and runoff records at Abbott Run have been kept continuously since 1907.

Rainfall data were, however, recorded continuously at Reservoir No. 1 from 1886 to 1941. Since that time, rainfall data have been recorded daily at the Woonsocket Sewage Treatment Plant, located about 2 miles north of Reservoir No. 1.

A study of these data indicated that the average rainfall at Reservoir No. 1 exceeds the rainfall on Abbott Run by about 6 per cent. The runoff characteristics of the two watersheds are compara-

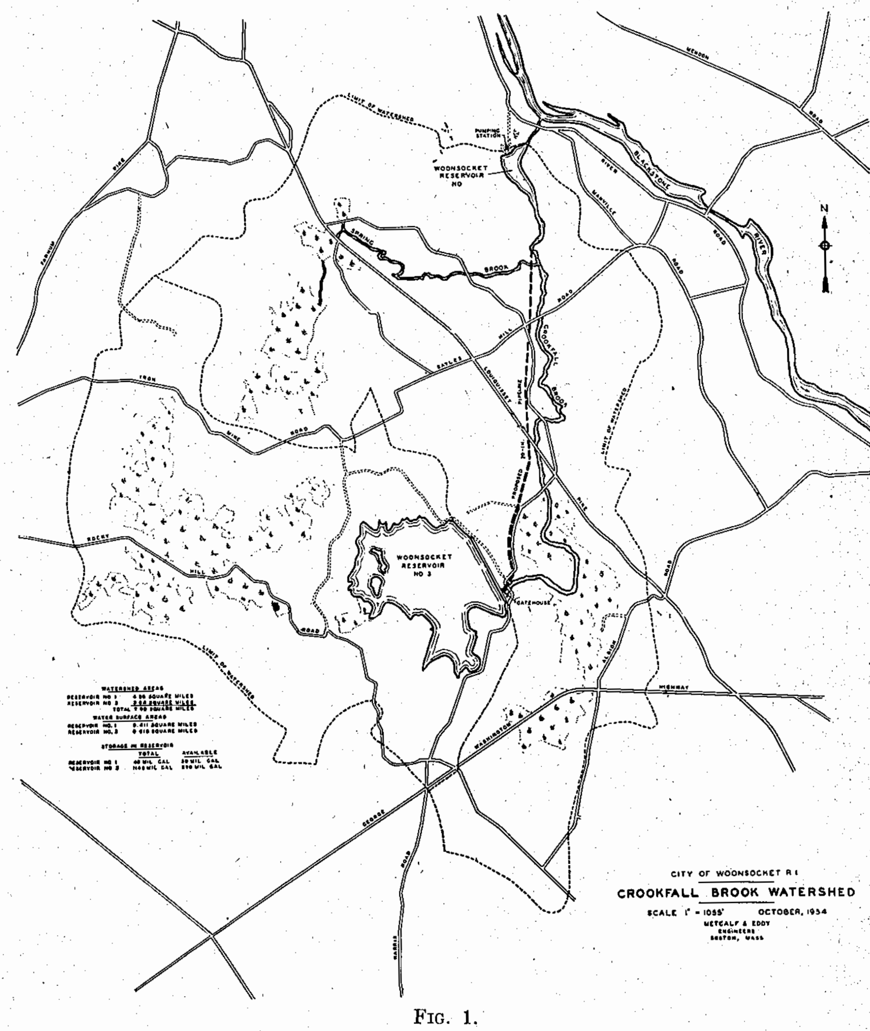


FIG. 1.

ble and it was therefore concluded that the runoff of the Crookfall Brook watershed should be at least equal to that of the Abbott Run watershed. In addition, it was concluded from a study of rainfall frequency at Reservoir No. 1 that an annual rainfall of 32.67 in., the 1930 average rainfall and the lowest annual rainfall at Reservoir No. 1 on record, can be expected only twice in every 100 years.

Utilizing all of these data and diagrams of safe yield prepared by the New England Water Works Association, it was estimated that the safe yield of the Crookfall Brook watershed was 4.5 mgd.

However, in 1930 when the average daily consumption was 3.3 mgd., the water level in Reservoir No. 3 dropped to a dangerously low point. During the preceding winter, the rainfall was below average and Reservoir No. 3 was not full at the beginning of the dry season. During the spring and summer of 1930 the rainfall continued below normal and the quantity of water in storage continued to drop to a dangerous level, although the supply was not entirely depleted. Because of the apparent discrepancy between the safe yield as computed, and the apparent safe yield, as demonstrated by the conditions in 1930, a study was made to justify or discredit our method of computing safe yield. As a result, it was concluded that approximately 1 mgd. of water was lost during the passage of the water through the 2½ miles of brook and swamps from Reservoir No. 3 to Reservoir No. 1. This loss, the result of evaporation, transpiration, and percolation into the swamps along the brook, was demonstrated by comparing the actual water in storage at Reservoir No. 3 during the summer of 1953 with the computed quantity in storage based on actual water consumption, computed runoff, and reservoir losses.

Our first recommendation, therefore, relative to improvement of the Crookfall Brook supply was the installation of a concrete pipeline leading from Reservoir No. 3 to Reservoir No. 1 to eliminate this loss. In addition, installations for measuring the quantity of water discharged from Reservoir No. 3, for recording the water level in that reservoir, and for remote control of the reservoir valves were also recommended.

Groundwater Investigations. As indicated before in Table 1, the future demand is estimated to be 9.5 mgd. The deficit in water supply is, therefore, the difference between this demand and the safe yield of the Crookfall Brook supply or 5.0 mgd. Studies were made to determine the most economical method of increasing the water supply.

An investigation was first carried out to locate possible sources of groundwater which would be capable of rapid development. A seismic search was made within the City limits and in areas adjacent to the City to determine areas suitable for groundwater supplies. This initial work was followed by drilling 2½-in. test wells in those areas where the results of the seismic tests were favorable and in several other areas which also appeared promising after further geological investigation. If test wells yielded less than 30 gpm., they were abandoned because of their limited capacity.

Only two of the fifteen areas so investigated had a sufficient depth of water-bearing sand-and-gravel deposits to warrant further investigation. In these two areas, 8-in. test wells were installed and full-scale pumping tests were made. The safe yield from these two wells was 550 gpm. and 290 gpm., respectively, or a total safe yield of 1.2 mil. gal. Laboratory tests of the water from one of the wells indicated the presence of 1.6 ppm. of manganese and slight bacterial contamination. No manganese and less bacterial contamination were present in the samples taken from the other well, although it appeared reasonable to expect that under prolonged pumping this well might also show some manganese. Both wells contained in excess of 25 ppm. of free carbon dioxide but neither well showed the presence of iron.

It was recommended that water from both wells be conveyed by a common discharge force main to new aeration facilities at Reservoir No. 1 for removal of free carbon dioxide. The aerated well water and the surface water in Reservoir No. 1 would then be blended, treated in a rapid sand filter plant and pumped to the distribution system. It was anticipated that the quantity of iron and manganese in the raw mixture of surface water and well water would slightly exceed the U. S. Public Health Service standard of 0.3 ppm. However, it was our opinion that the water would be acceptable after chlorination during the short period prior to construction of the proposed water treatment plant.

Harris Pond. A study was made of all surface water sources near Woonsocket to determine if any of these sources were possible for economic development. It was our conclusion that Harris Pond on the Mill River met all requirements. The pond and its water rights are privately owned. It lies mostly in Massachusetts, has a safe yield of approximately 6.5 mgd., and was contained by a dam constructed in 1869. Subsequent to the date of our report the dam was breeched by the flash-flood of August 19, 1955 and requires repairs.

Harris Pond water is similar in analysis to Crookfall Brook water and requires only treatment before it can be used by the City. It was, therefore, our recommendation that the water rights and property upon which Harris Pond stood be purchased by the city of Woonsocket.

It was also recommended that a pumping station be installed at Harris Pond and that the pond water be pumped to the same pipeline which carried well water to Reservoir No. 1. This plan, which was also the most economical plan, will allow all water to be treated at a single plant and will ensure that all parts of the system receive water of uniform quality.

Other Surface Water Supplies Investigated. A recommendation made in older reports for a water supply on Tarkiln Brook, southwest of Woonsocket, was reinvestigated. It was determined that this supply would not be sufficient in itself to supply all of Woonsocket's needs, and that it would be necessary to keep the Crookfall Brook supply as an auxiliary supply. The development of the Tarkiln Brook supply would be far more complicated and costly than the development of the Harris Pond supply because, in addition to the facilities required at Harris Pond, it would be necessary to construct and operate treatment plants at both Tarkiln Brook and Reservoir No. 1. Also, the treated water had to be pumped a distance of over 4 miles to reach the northern edge of the City's water distribution system, and about 3 miles of a major roadway passing through the site of the Tarkiln Brook Reservoir would have to be relocated. Consequently, it was determined that development of Tarkiln Brook was not as favorable economically as was development of Harris Pond.

A favorable site on the Mill River above Harris Pond was also investigated for development of a supply. Although the river at the point of investigation could be developed to provide a safe yield of about 14 mgd., the supply would lie wholly within Massachusetts. Its development, therefore, would require legislative action by both Rhode Island and Massachusetts. In addition, there was the problem of compensation for diversion of water involving both the owner of Harris Pond and the owners of the mills lower down on the Blackstone River and the problem of a long pipeline to reach this site. As in the case of Tarkiln Brook, it was our opinion that it was far less expensive to acquire the water rights and develop the Harris Pond supply than to construct a new reservoir on the Mill River in Massachusetts.

Water Treatment Plant. The physical and chemical qualities of Crookfall Brook water are unsatisfactory, exhibiting excessive color, tastes, odors, and iron. These deficiencies were recognized as far back as 1895 when Reservoir No. 3 was constructed. As treatment of water based on scientific principles had only recently been introduced, it was decided to postpone construction until filtration principles had been more fully investigated. Unfortunately, no works were ever constructed although chlorination of the water was initiated in February 1921.

In addition to the Crookfall Brook water, the water from Harris Pond will also require treatment because of the similarity of the two waters and the well water will require treatment to remove excessive manganese and free carbon dioxide. It was therefore decided to aerate the Harris Pond and well supplies at Reservoir No. 1 and to blend the aerated mixture with Crookfall Brook water before treatment. Coagulation tests indicated that the water was readily treatable with conventional alum and lime treatment. It was

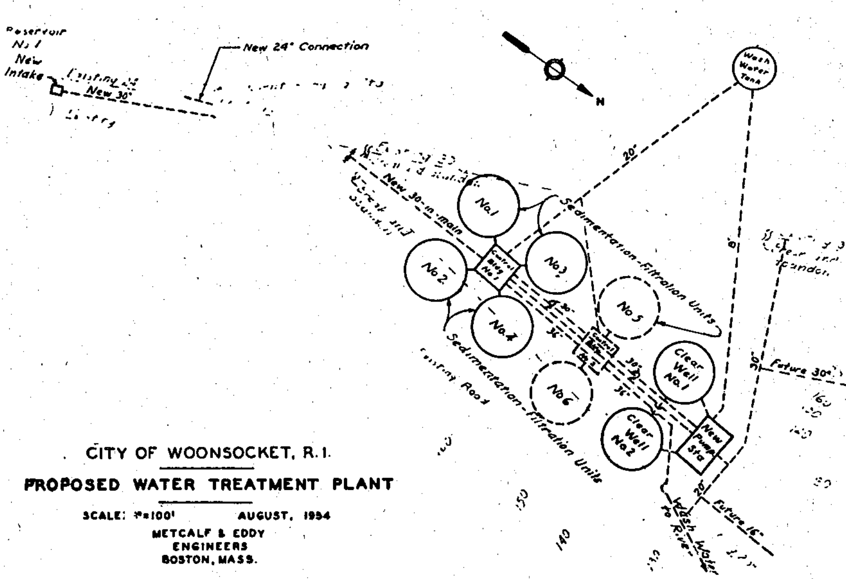


FIG. 2.

recommended that a 10-mgd. treatment plant, with provision for expansion to 15 mgd., be constructed at Reservoir No. 1. The type of plant recommended was of the "Aldrich" type utilizing concentric treatment units. An initial saving of $50,000 per million gallons was indicated in the use of this type plant over a conventional rapid sand filter plant. A layout of the proposed water treatment plant is shown in Fig. 2.

Included in the cost of this new plant was the construction of a new high-service pumping station to replace the existing pumping facilities at Reservoir No. 1.

Summary. A summary of the water supply costs and a recommended program of development is given in Table 2.

TABLE 2
WATER SUPPLY COSTS

Immediate Construction	
1. Development of well supply	$298,500
2. Improvement to Crookfall Brook	176,800
Subtotal	$475,300
Within 5 years	
3. 10-mgd. water treatment plant	1,167,000
4. Development of Harris Pond	242,000
Subtotal	$1,409,000
Future	
5. 5-mgd. water treatment plant addition	306,000
Total	$2,190,300

WATER PUMPING, STORAGE AND DISTRIBUTION

Pumping. The pumping facilities at Reservoir No. 1 consist of two electrically driven pumps installed in 1918 and 1919 and one standby gasoline-engine-driven pump installed in 1946. All pumps are housed in a building constructed in 1895. Tests conducted on the two electric pumps indicated pumping rates of 6.05 and 6.72 mgd., at a head of 223 and 226 ft., respectively. The pump efficiencies were 78.5 and 79.0 per cent, respectively. The gasoline engine pump supplied 8.5 gpm. at a head of 228 ft.

A number of deficiencies exist in the Pumping Station requiring piping changes, replacement of chlorination equipment, installation of new recording instruments, and replacement of "open-front" switchgear and open wiring with modern safe equipment. These changes were recommended if the construction of the new Pumping Station at the Treatment Plant were to be delayed beyond 1956. When the new Pumping Station is constructed, it was further recommended that the three existing pumps be moved to the new station and that the present building be converted to an office and shops for the Water Department to replace their present totally inadequate quarters.

Based upon requirements of the National Board of Fire Underwriters, it was determined that a third electrically driven pump of 8-mgd. capacity is required immediately for fire protection in the event of breakdown of present pumping equipment. In addition, it was determined that the two existing electrically driven pumps should be replaced with two new pumps having a combined capacity of 15.5 mgd. when the maximum daily water consumption exceeds 12 mgd.

Storage. Storage for the water distribution system is provided in five ground storage tanks located on Mt. St. Charles, approximately in the center of the system. Four of these tanks are connected to the low service system and one tank, a standpipe, is connected to the high service area. The total capacity of the four low service tanks is 3.50 mil. gal. and that of the high service tank is 1.55 mil. gal. Water is pumped by a small pumping station located at the tanks from the low to the high service system.

Four of the five storage tanks are of wrought iron or steel construction and are badly in need of painting. The exterior of the fifth tank of concrete construction, has badly disintegrated and requires immediate repairs.

The present high-service Pumping Station at the storage tanks has been in continuous operation for over 20 years. At the present time, the single pump installed in this station is in poor repair and the electrical controls and wiring are obsolete and unsafe. Because the station is too small to house a second pump, it was recommended that a new high service pumping station with duplicate pumps be constructed at the site of the present station. It was also recommended that a positive connection equipped with a relief valve be provided between the high and low service systems. This connection will permit the use of high service water in the low service system during an emergency without danger to the low service piping as a result of excessive pressures.

In the future, it will be necessary to construct additional elevated storage tanks as the need for water in the outlying portions of the City increases. Only one additional tank will be required on the low service system. However, three new high-service booster stations and elevated tanks will be required.

Distribution System. An analyses of the system based on present and future estimated flows, including fire flow, was made on all pipes 12 in. or larger. It was concluded that with several exceptions the principal water mains are of adequate size for present conditions.

Under future conditions, however, a new river crossing near the water treatment plant will be required to supply water to the eastern portions of the City, and in addition, fourteen additional new reinforcing pipelines will be required. A program of these reinforcements was presented to the City.

Also presented was a program of future extensions covering all streets now without City water but requiring water immediately or in the near future.

Summary. A summary of water pumping, storage and distribution costs is given in Table 3.

Stream Pollution and Sewage Treatment

The present Sewage Treatment Plant was built in 1931, for an average sewage flow of approximately 3.3 mgd. Since that time, the flow of sewage and industrial wastes to the plant has nearly doubled. Many items of plant equipment have exceeded their useful life, and it is now impossible for the plant to effectively handle the present load. In order to meet the requirements of the State Department of Health, extensive additions and alterations are required.

Gagings were made and samples collected over a 5-day period of all sewage and industrial wastes reaching the Sewage Treatment Plant. The total volume of sewage and wastes thus gaged was 5.9 mgd., of which 3.0 mgd. was estimated to be infiltration. The per capita contribution of sewage during the gaging period was determined to be 63 gcd.

In addition to sewage and wastes reaching the plant, many industries within the City still discharge their wastes without treatment to the river. If these wastes are to be accepted into the sewerage system, provisions for their treatment must be incorporated in the design of the plant. Consequently, a personal survey of 45 industries was made by our engineers to obtain information regarding the quantity and quality of the industrial wastes. On the basis of the information thus obtained, it was estimated that approximately 0.37 mgd. of industrial wastes are presently being discharged to the sewerage system and that 1.6 mgd. are discharged untreated to the river. If these untreated wastes are intercepted, the total volume of sewage and wastes currently requiring treatment is 7.5 mgd.

TABLE 3

SUMMARY OF WATER PUMPING, STORAGE, AND DISTRIBUTION COSTS

Within 2 years	
Repairs to present pumps	$ 1,700
High-Service Pumping Station	32,000
Repairs and painting storage tanks	58,800
Reinforcements to Distribution System	89,000
High-Service Pressure Reducing Valve	5,000
Repairs to Main Pumping Station	33,000
Subtotal	$219,500
Within 10 years	
Reinforcements to Distribution System	$277,000
Extensions to Distribution System	288,000
Subtotal	$565,000
Future	
Low Service Elevated Tank	$ 60,000
Three high-service districts	564,200
Reinforcements to Distribution System	191,700
Subtotal	$815,900
Total	$1,600,400

Further estimates of the quantity and composition of the sewage and wastes are given in Table 4. The first main heading, namely, "Present" includes only those industrial wastes now discharged to the sewers, whereas the second heading "Design" assumes all industrial wastes have been intercepted.

Stream Survey. To determine the capacity of the Blackstone River to absorb pollution and as a consequence to determine the degree of treatment of the sewage and wastes that may be required, a detailed stream survey was conducted. River sampling stations were established at five points as shown in Fig. 3. Samples were taken at these stations over a 2-day period to determine existing river conditions. Flows in the river during the time of sampling were also determined. As shown in Fig. 4, the critical point with regard to dis-

TABLE 4
QUANTITY AND QUALITY OF SEWAGE AND INDUSTRIAL WASTES

	Present		*Design*	
Population	50,200		55,000	
Population served	40,000		55,000	
Industrial production	Normal	Maximum	Normal	Maximum
Flow, mgd.	5.91	6.09	9.36	10.76
5-day, 20 deg.C., B.O.D., lb./day	7,585	8,285	22,155	29,815
Suspended solids, lb./day	7,680	8,055	31,475	40,775

solved oxygen in the stream occurs at Manville Dam located approximately 2.5 miles below the Sewage Treatment Plant. These data and the time of flow between stations were used to estimate the various river reaeration coefficients.

On the basis of a study of river discharges over the past 25 years

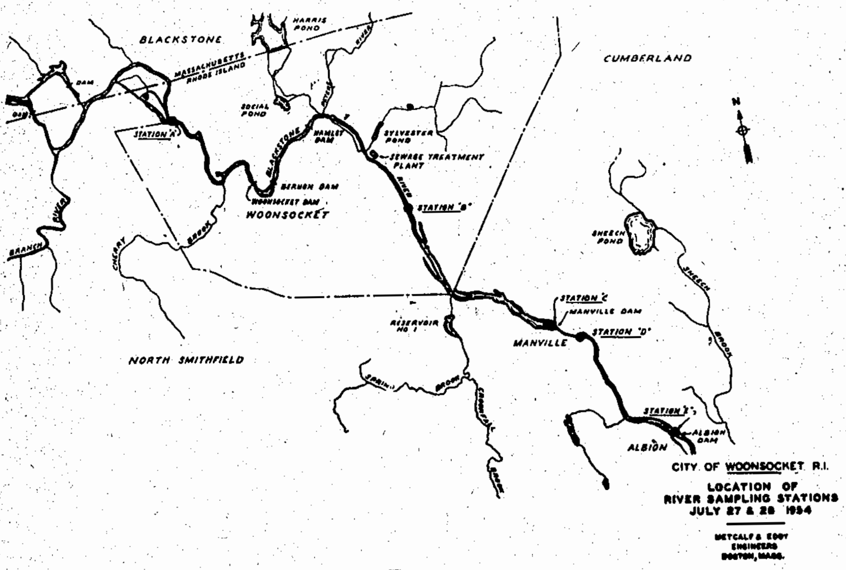


FIG. 3.

of record and the previous data, it was estimated that a satisfactory dissolved oxygen level at Manville Dam can be maintained in the river during the summer months by removal of 85 per cent of the present B.O.D. load, which removal must be increased to 90 per cent by the end of the design period.

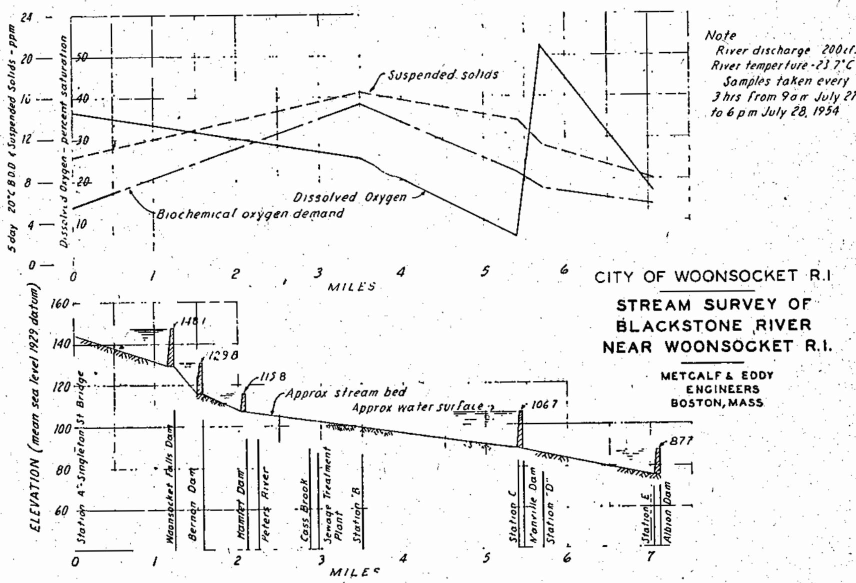


FIG. 4.

Treatment Plant. It was therefore recommended that the present treatment plant be modified to a treatment plant employing the "step-aeration" and "biosorption" processes. These processes, a type of high-rate activated sludge treatment, were selected because only one-half of the aeration tank capacity needed for conventional activated sludge treatment is required; because they are less affected by industrial wastes; and because the sludge is more readily compacted before digestion.

The air requirements of the extremely strong sewage and wastes were estimated to be approximately 2.2 cu. ft. per gallon based on 0.0065 cu. ft. of air per gallon of raw sewage per part per million of 5-day, 20-deg. C. B.O.D. removed in the over-all plant. This quantity of air is also equal to 950 cu. ft. per pound of B.O.D. applied to the tanks, which quantity is comparable to that required by other industrial wastes.

The proposed treatment plant is shown on Fig. 5. Specifically, the additions required to convert the present plant to a 10.8-mgd. plant include a new sewage pumping station, new screening and grit-removal facilities, two new primary settling tanks which will also be used to remove up to 75 per cent of the grease reaching the plant, new sludge pumping facilities, modification and expansion of the existing four aeration tanks, new blower facilities, two new secondary settling tanks, conversion of the existing final settling tanks to chlorine contact tanks, new sludge thickening and digestion facilities, new chlorination facilities, and rehabilitation of the existing sludge drying beds and conversion of several old sand filter beds to sludge lagoons.

The estimated cost of this new plant is $2,966,500. However, construction of the new aeration tanks and of two of the sludge digestion tanks could be deferred until some time in the future. The cost of the plant without these facilities is estimated to be $2,298,500. It was further recommended that this construction be divided into primary treatment facilities constructed within two years at a cost of $1,203,300 and secondary treatment facilities within 5 years at a cost of $1,095,200.

Sewerage System

The sewerage system of Woonsocket is divided into two systems; namely, a "high level" system serving the northeastern portion of the City, and a "low level" system serving the remaining 75 per cent of the City. In general the problems relative to sewerage involved determination of the major sources of infiltration and corrective measures required; modifications or reinforcements to the system required to handle increased flow or to eliminate present overloading; and extensions to the system required to provide service to areas not now served by sewers.

Infiltration. The main trunk sewer of the "low level" system follows the Blackstone River. The sewer, with a maximum diameter of 36 in., is of brick construction and lies for most of its 5,100 ft. length below river level. Inspections indicated that the entire sewer was subject to possible infiltration, particularly in the upper end where heavy infiltration was indicated. It was recommended that the upper portion of the sewer in River Street be cleaned and that a more detailed inspection be made to determine the approximate locations of the excessive infiltration. It was also recommended that, if necessary, this portion of sewer be relaid.

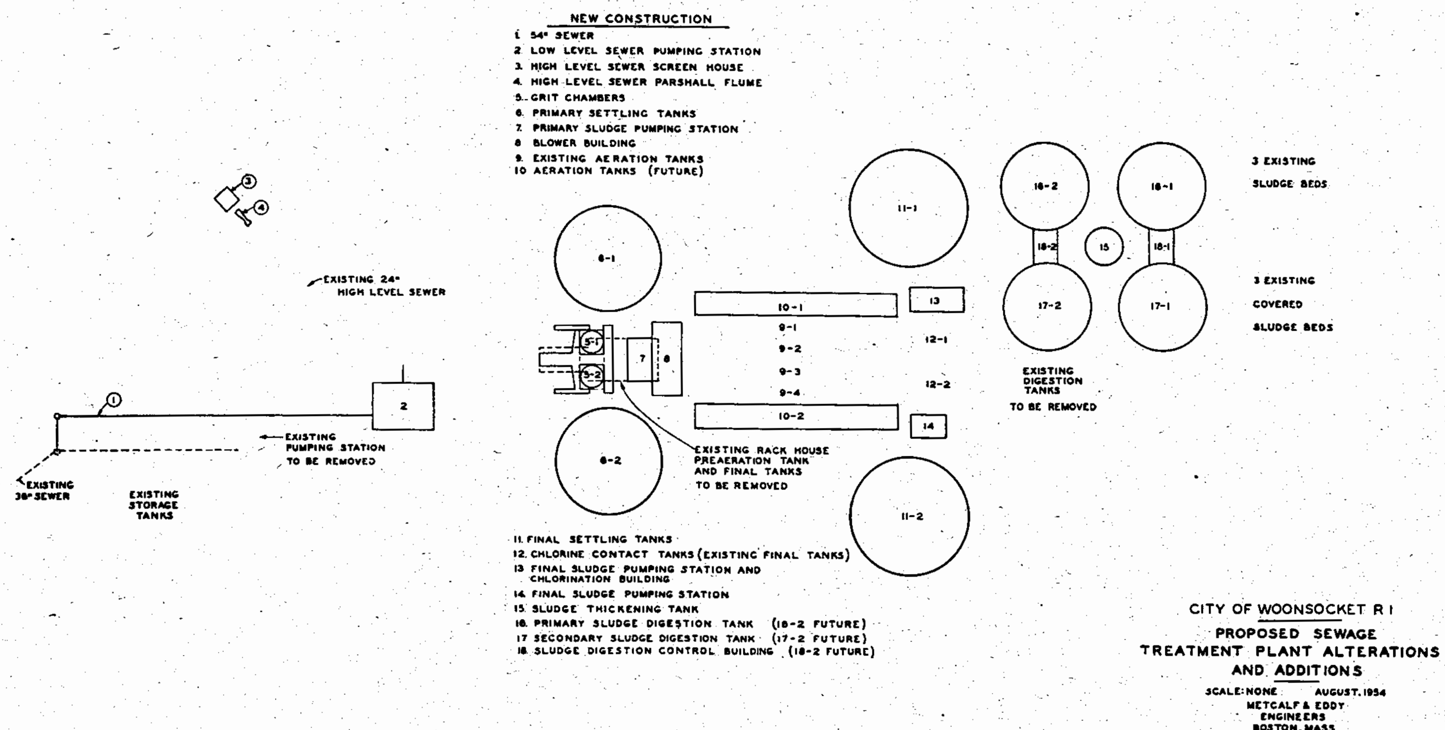

NEW CONSTRUCTION
1 54" SEWER
2 LOW LEVEL SEWER PUMPING STATION
3 HIGH LEVEL SEWER SCREEN HOUSE
4 HIGH LEVEL SEWER PARSHALL FLUME
5 GRIT CHAMBERS
6 PRIMARY SETTLING TANKS
7 PRIMARY SLUDGE PUMPING STATION
8 BLOWER BUILDING
9 EXISTING AERATION TANKS
10 AERATION TANKS (FUTURE)
11 FINAL SETTLING TANKS
12 CHLORINE CONTACT TANKS (EXISTING FINAL TANKS)
13 FINAL SLUDGE PUMPING STATION AND CHLORINATION BUILDING
14 FINAL SLUDGE PUMPING STATION
15 SLUDGE THICKENING TANK
16 PRIMARY SLUDGE DIGESTION TANK (16-2 FUTURE)
17 SECONDARY SLUDGE DIGESTION TANK (17-2 FUTURE)
18 SLUDGE DIGESTION CONTROL BUILDING (18-2 FUTURE)
EXISTING 24" HIGH LEVEL SEWER
EXISTING PUMPING STATION TO BE REMOVED
EXISTING 36" SEWER
EXISTING STORAGE TANKS
EXISTING RACK HOUSE PREAERATION TANK AND FINAL TANKS TO BE REMOVED
EXISTING DIGESTION TANKS TO BE REMOVED
3 EXISTING SLUDGE BEDS
3 EXISTING COVERED SLUDGE BEDS
CITY OF WOONSOCKET R I
PROPOSED SEWAGE
TREATMENT PLANT ALTERATIONS
AND ADDITIONS
SCALE: NONE AUGUST, 1954
METCALF & EDDY
ENGINEERS
BOSTON, MASS

FIG. 5.

Other studies and methods of correction were recommended for points of lesser infiltration in other portions of the City.

Modifications and Reinforcements to Trunk Sewer. On the basis of a study of population density and industrial wastes volumes, recommendations were made relative to modifications and reinforcements.

The study indicated that the major trunk lines of the system had adequate capacity to handle present and future flows with the following exceptions:

a. Low Level Trunk Sewer. Additional capacity must be provided in this sewer when all of the industrial wastes now being discharged to the river are diverted to the City's system. The recommended plan involved complete replacement of the older downstream brick section with new pipe of greater capacity and construction of a parallel sewer in the remaining sections to provide additional capacity for future flows.

b. Diamond Hill Trunk Sewer. It was determined that a relief sewer will be required to reinforce the existing sewer as northern areas of the City are sewered.

c. Cass Avenue Trunk Sewer. It was determined that when the eastern areas of the City are sewered, a new relief sewer will be required to prevent overloading of existing sewers. The cost of this sewer was not estimated as its capacity and location were too indefinite.

Extensions. The unsewered areas of the City were divided into seven sewer districts. Each district was studied and recommendations made relative to the size and cost of sewer required in each street. Also recommended was the construction of a total of three small pumping stations.

Summary. A summary of costs of sewerage facilities is given in Table 5.

Rubbish and Garbage Disposal

Household rubbish and ashes are collected by a contractor engaged by the City. Collections are made weekly throughout most of the City although in sparsely settled areas, collections are limited to once in every 2 weeks. All rubbish is deposited on an open dump operated by the City and located along the river near the center of the City. The dump capacity is now nearly exhausted. Either additional dumping areas or alternative new methods of rubbish disposal will be required in the near future. Although efforts have been made in recent years to control the operation of the dump, it remains a source of smoke, odors, debris and provides a breeding place for rodents and insects.

TABLE 5
SUMMARY OF SEWERAGE COSTS

Immediate	
Repair or relay River Street sewer	$ 39,000
Within next 5 years	
Replace and reinforce low-level trunk sewer	791,000
Reinforce Diamond Hill trunk sewer	30,000
Within next 10 years	
Sewer extensions	612,700
Future	
Sewer extensions	1,313,600
Total	$2,786,300

Garbage is collected twice weekly by another contractor engaged by the City for disposal at his piggery in Cumberland, Rhode Island. Although the piggery is clean and well maintained, the future of this method of disposal is questionable because of recent legislation requiring the cooking of garbage prior to its use as a hog feed. Consequently, study was given to alternate methods of garbage disposal.

Methods of Refuse Disposal and Quantities of Refuse. Studies were made of the several methods of garbage and refuse disposal which could be adopted by Woonsocket and it was concluded that the most economical method of disposal was by means of incineration.

To obtain a basis for design, all rubbish delivered to the dump was weighed for a period of 1 week and all garbage collected on an average day was also weighed. It was thus determined that approximately 1.6 and 0.4 lb. per capita of rubbish and garbage, respectively, were collected in Woonsocket on an average day. This information was then compared with the per-capita weight of combustible rubbish in several other cities and as a result it was decided to base the furnace design on a weight of 2.5 lb. per capita per day, the approximate average of the other cities. It was our opinion that the low per-capita contribution in Woonsocket was partially the result of widespread unemployment in the City.

TABLE 6

SUMMARY RECOMMENDED PROGRAM OF EXPENDITURES

	First Cost of Projects				
	Immediate	*Within 2 years*	*Within 5 years*	*Within 10 years*	*Future (as required)*
Water					
Supply and Treatment	$475,300	0	$1,409,000	0	$ 306,000
Pumping, Storage and Distribution	97,500	$ 89,000	365,700	$ 199,300	815,900
Sewerage					
Sewage Treatment	0	1,203,300	1,095,200	668,000	0
Sewers and Pumping Stations	39,000	0	821,000	612,700	1,313,600
Refuse Disposal					
Incinerator	0	520,000	0	0	0
Total Costs	$611,800	$1,812,300	$3,690,900	$1,480,000	$2,435,500

Incinerator. Several sites and sizes of furnaces were studied. It was ultimately recommended that a side hill site near the sewage treatment plant be obtained, and that an incinerator housing two 80-ton furnaces be constructed. These furnaces would provide sufficient capacity to destroy all of the present rubbish in a single 8-hr. shift with both furnaces in operation. At the end of the design period both furnaces would be needed for two 8-hr. shifts.

The cost of the incinerator plant was estimated to be $520,000.

Summary of Recommendations

Because of the large capital expenditures required to carry out the complete program and because many of the items will not be required for a number of years, a step construction program was devised based upon the time when it was estimated that the various facilities would be required. A summary of all of the recommended facilities in this step program is given in Table 6. It should be recognized that this program is not inflexible and that items may be changed from one category to another as the need changes. However, all of the items scheduled within the next 10 years should be completed as expeditiously as possible within the financial capacity of the City to provide an adequate system of sanitary facilities and further to provide a firm base upon which future improvements may be added.

Acknowledgments

The author wishes to acknowledge the interest and cooperation of all of the municipal officials and employees of Woonsocket, particularly Mayor Kevin K. Coleman, and Mr. Charles G. Hammann, Director of Public Works. He also wishes to acknowledge the assistance given by Fr. Daniel Linehan, S.J., Director, Seismological Station at Weston College, and Mr. William B. Allen, U. S. Geological Survey, Groundwater Branch in connection with the groundwater exploration program. Assistance of State officials, executives and employees of the many industries and of public utilities companies in Woonsocket is also gratefully acknowledged.

THE MACKINAC BRIDGE — CONQUERING THE IMPOSSIBLE

By D. B. Steinman*

[Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on November 9, 1955.]

"HELP ME, LORD, TO BUILD MY SPAN"

Anchored firm in solid rock,
On Thy foundation let me build—
Strong to bear each strain and shock,
An arch of dreams and faith fulfilled.

Help me, Lord, to build my span
Across the chasm of the years;
Firm in purpose, true in plan,
Above the drag of doubt and fears.

Help me to build on Thy high road
A bridge to serve the common good;
To smooth the way and lift the load,
A link of human brotherhood.

Introduction

The thought of connecting two sections of the State of Michigan by a physical link across the Straits of Mackinac has challenged the imagination of engineers and the public for the past three-quarters of a century. The difficulties, both physical and financial, appeared insurmountable. Various plans and designs were proposed from time to time during the past forty years. Some of the schemes would have been impossibly fantastic in cost, but the promoters did not know it. One official design for the proposed bridge would have collapsed before completion, but the officials did not know it.

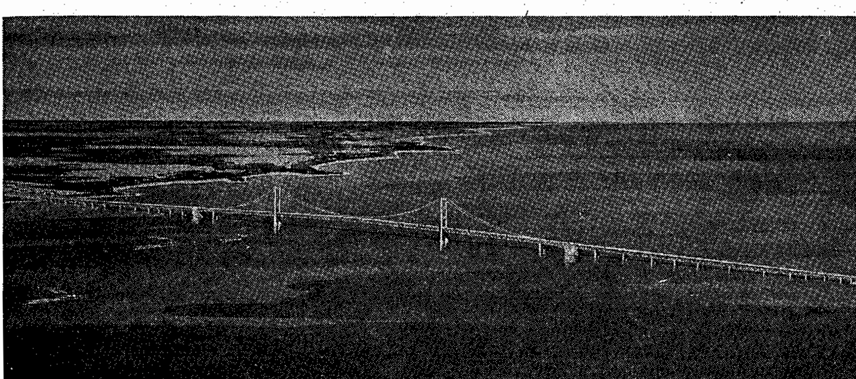
People (who were not engineers) said that the project was impossible; that the cost would be prohibitive; that it could not be financed; that the bridge could not be built; that the foundation prob-

*Consulting Engineer, New York City.

lems could not be solved; that the wide glacial gorge under deep water in the middle of the Strait could not be spanned; that the bridge, if built, would not stand up; that it would be destroyed by the elements; that no foundation piers could withstand the pressure of ice from the Great Lakes in winter; that no span could withstand the storms and wind forces at the site.

Despite all obstacles and difficulties, both natural and man-made, the project has now been successfully financed; all of the engineering problems have been successfully, economically and safely solved; the difficult foundations have been successfully conquered; and the construction of the bridge, commenced in July 1954, is well under way to meet the scheduled completion date of November 1957.

The Mackinac Bridge is five miles long. In the middle of the five miles, in the deepest water, to span the wide, submerged glacial canyon, a record-breaking suspension bridge is being built; the length, 8614 feet from anchorage to anchorage, makes it the longest suspension bridge in the world. The central span of this suspension bridge,



PERSPECTIVE DRAWING OF MACKINAC BRIDGE.

from tower to tower, is 3,800 feet; this is 300 feet longer than the span of the George Washington Bridge, and is exceeded only by the 4200-foot span of the Golden Gate Bridge. The difficult foundations under the two main towers of the suspension bridge, one at each rim of the submerged gorge, were carried down to rock, reaching the remarkable foundation depths of 205 feet and 210 feet, respectively, below the water surface. The suspension bridge cables are carried on artistic steel towers 550 feet high, each containing 6250 tons of structural steel; and the suspended trusses, carrying the roadway, have a normal clear height of 155 feet above the water.

The total cost of the bridge, including the bond-interest during construction, is $99,800,000; this is the amount of the bond issue. The cost figure establishes a new record for the magnitude and difficulty of a bridge project, and will certainly be a long-time record for a bridge carrying only four lanes of highway traffic and no railway loading.

Without careful economic design, the cost would have been many millions of dollars greater and the financial feasibility of the project would have been defeated. Scientific design made the bridge possible, while at the same time assuring a high margin of strength and safety in generous measure.

By spending a few million dollars more, the span could easily have been made the longest in the world. (In fact, the foundation problems would have been easier.) But the writer feels strongly that an engineer is violating his obligation if he seeks personal glory at the expense of his clients, in this case the traveling public.

The Need for the Bridge

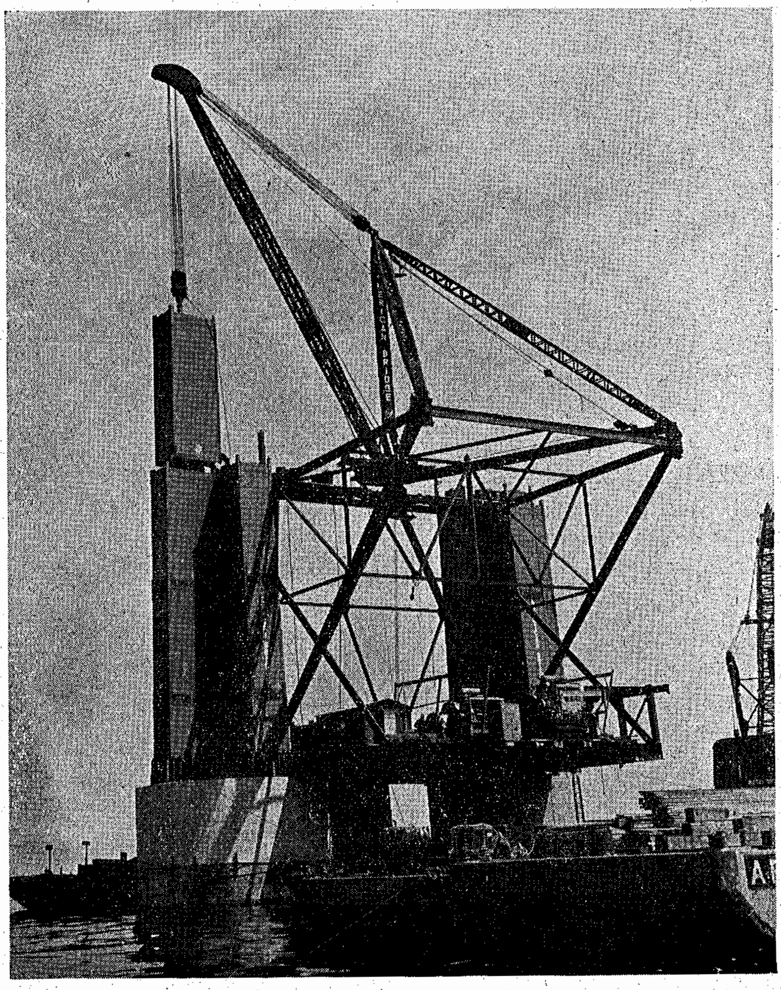
The Strait of Mackinac (pronounced "Mackinaw"), four miles wide, joins Lake Michigan and Lake Huron. These waters divide the State of Michigan into the 41,700 square mile Lower Peninsula and the 16,500 square mile Upper Peninsula.

The far greater part of the population is concentrated in the highly industrialized Lower Peninsula and its large cities such as Detroit; but the Upper Peninsula is possessed of immense natural resources which, when further developed, will attract additional population and industrial activity.

The Upper Peninsula is 400 miles long and is nearly equal to the combined area of four New England states. The principal industries at present are forestry, mining, agriculture, and recreation. Part of this area is world-famous as "The Copper Country." The area is also known as a "Vacation Paradise," drawing tourists and sportsmen from many states for hunting, fishing, camping, sailing, and winter sports.

The Mackinac Bridge will replace the existing State-operated highway ferry system in order to provide an all-year, all-weather, direct, time-saving connection between these two great Peninsulas of Michigan.

It is recognized that the project which will contribute most to the further development of the Upper Peninsula is the Mackinac Bridge. But in a larger measure, it will contribute to the advantages of Michigan as a whole and of the entire Great Lakes area as well as of the Province of Ontario in Canada. In the words of Governor



ERECTION OF BASE OF MAIN TOWER.

G. Mennen Williams, the builders of the Mackinac Bridge "are participating in Empire-building."

People doubted the possibility of financing the Mackinac Bridge because it does not directly connect two large cities or population centers. But modern highway uses have enlarged our vision and our perspective. Within a radius of 500 miles from the Straits of Mackinac there resides a population of 30,000,000 people in the United States and Canada who will benefit from the construction of the Mackinac Bridge and who, in turn, insure the economic practicability of the project.

The major highways of Michigan converge at Mackinaw City on the south and St. Ignace on the north of the Straits of Mackinac. Thus the Mackinac Bridge will funnel traffic from the Lower Peninsula into the Upper Peninsula and then into Canada by way of Sault Ste. Marie, 50 miles north of Mackinac Straits. Furthermore, the Mackinac crossing will provide a shorter east-west route for bonded traffic between the western provinces of Canada and populous southeastern Ontario. At this key location, the Mackinac Bridge will be an essential connecting link in the national and international highway system, and of high strategic importance in our Continent's defense program.

Truck traffic on the Mackinac Straits ferries has been increasing rapidly, and already amounts to 12 per cent of the total vehicular traffic.

The Mackinac ferry rates were increased 45 per cent in 1953 and, in spite of this increase in rates, traffic for the ensuing months increased 12 per cent above the same period of 1952.

The five-mile ferry crossing takes over one hour; the bridge will reduce the crossing time to ten minutes. But, more important, the bridge will save the time now lost in waiting in line for the ferries. During the summer months, this waiting-in-line time amounts to 3 to 4½ hours; and on holidays and during the deer-hunting season, cars have had to wait in line as long as 14 to 17 hours. The lines of waiting cars have extended along the highway as far back as 20 miles from the ferry. Parking fields are provided for the waiting cars, and the occupants find overnight accommodations to resume their place in line in the morning.

Photographs, stereo-views, and movies of these traffic conditions at the Mackinac ferries were used to convince bankers and investors before the bridge bonds were sold.

The proposed toll rates on the bridge will average 10 per cent higher than the present rates on the ferries; the time-saving will be the governing advantage to the motorists. At an average toll rate of $3.08 per vehicle ($2.10 for a passenger auto, more for trucks), the estimated traffic of 2,000,000 cars and trucks in 1958 will yield a revenue of over $6,000,000 in the first year of operation, with progressive increase thereafter. According to the traffic experts, the bridge will pay for itself in 18 years (retiring all bonds), and can then be made toll-free.

From Dream to Reality

In 1920, the Michigan highway commissioner suggested a submerged floating tunnel for the Mackinac Straits crossing.

In 1928, the State highway department recommended a bridge, but the subsequent depression put a stop to the project.

In 1934, a Bridge Authority was created by the State Legislature. The Authority retained three successive consultants, who presented respective diverse plans in 1934, 1935, and 1940. World War II stopped all planning.

In 1950 the present Mackinac Bridge Authority was created by the Michigan State Legislature. The Authority promptly appointed a Board of Consulting Engineers: O. H. Ammann, G. B. Woodruff, and the writer. In 1951 the three-man Board of Consultants reported that construction of the bridge was feasible. The traffic-engineering firm of Coverdale and Colpitts was retained to make the survey of traffic and prospective revenue.

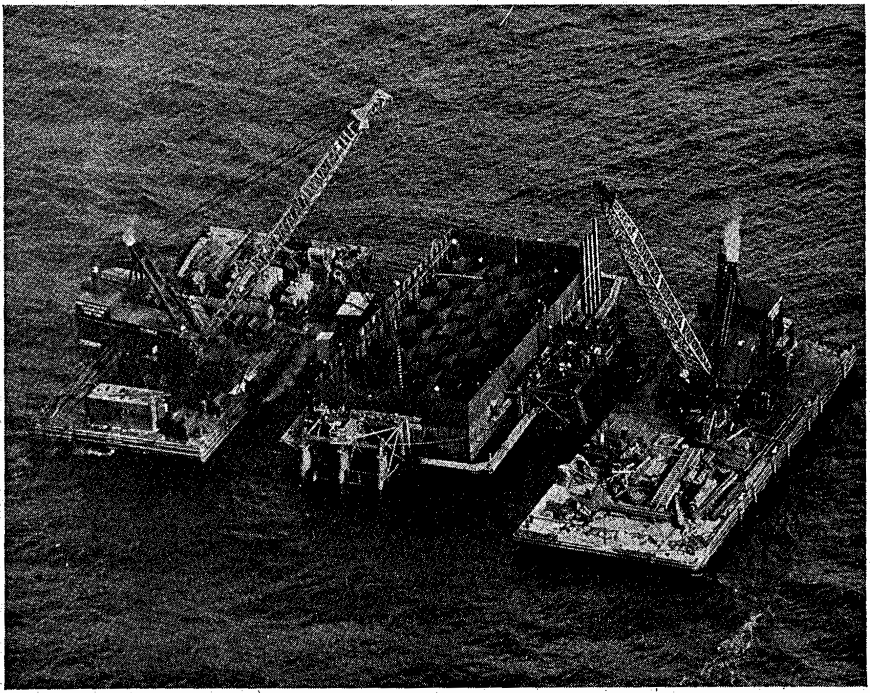
In January 1953, the Authority selected the writer to design and supervise the construction of the bridge, and the writer engaged Glenn B. Woodruff as his Associate Consultant. Within two months, in March 1953, preliminary contract plans and estimates of quantities were ready and the substructure and superstructure contracts were negotiated and awarded for prompt commencement of construction as soon as the bonds could be sold. All plans were rushed to get construction started in the spring of 1953.

Two attempts to sell the bonds were made in April and June, 1953, but the bond market was unfavorable. A new syndicate of investment bankers was formed and, in December 1953, this group of bankers purchased the $99.8 million of bonds to finance the project, at interest rates of 4 per cent for $79.8 million of first-lien bonds and 5¼ per cent for $20 million of second-lien bonds.

On January 18, 1954, the $79.8 million of first-lien bonds were offered by the banking syndicate and sold to the bond houses and investing public in one day. The second-lien bonds were offered to the public but were held by the underwriters for their own accounts.

Through the spring of 1954 the contractors proceeded to order materials and to mobilize equipment. During the next few months, $5 million of floating construction equipment was assembled and in place along the line of the bridge for the substructure contract, said to be the largest and finest floating equipment ever assembled for a construction contract.

On July 10, actual excavation was commenced for the subaqueous foundations. Over 750 men were engaged on the work at the site, working 20 to 24 hours a day. It was a race against time and a battle



South Cable Bent Pier—Constructed in Rectangular Steel Caisson with 21 Circular Dredging Wells.

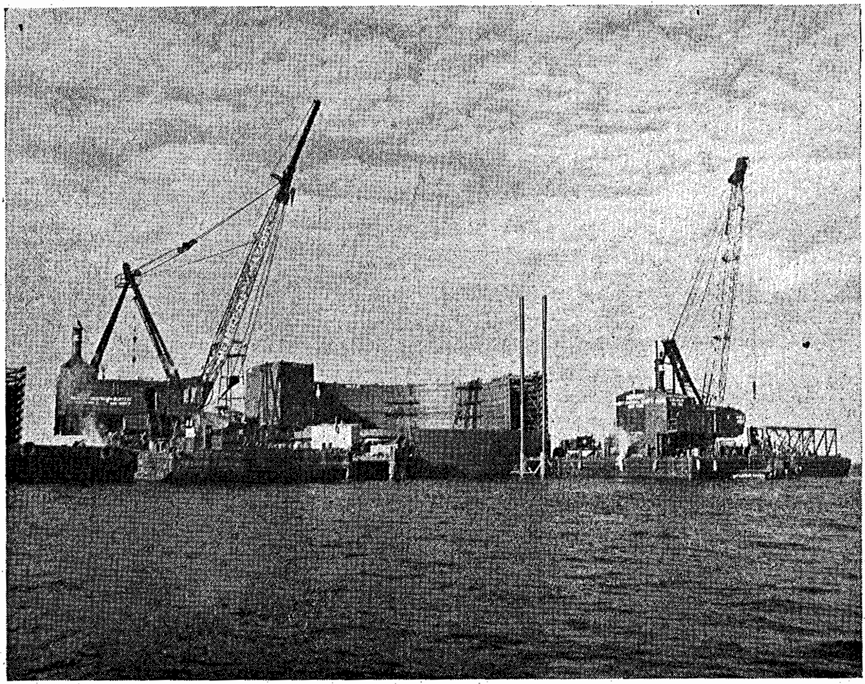
against the elements. The winter ice conditions at the Straits limit the normal working season to eight months. Word went down the line, from the president of the Merritt-Chapman & Scott Corporation to every man in the organization to spare no effort or expense to meet the engineer's schedule and to get all the suspension bridge piers and anchorages down to rock before the freezing of the Straits. To make up for time lost by impossible weather conditions, the men continued working in the rough water of the Straits through the winter cold, snow, and storms until freezing of the Straits finally forced the work to stop on January 14, 1955; but the two main-span piers were safely down into bed rock under the Straits, and the side-span piers and anchorages were already completed as scheduled.

An Ultra-Safe Bridge

Because of the unusual brecciated formation, people said that the rock underlying the Straits could not support the weight of the bridge. To resolve any doubts, outstanding geologists and soil-mechanics authorities were retained. Exhaustive geological studies, laboratory compression tests, and "in-place" load tests on the rock under water at the site established, without a doubt, that the rock under the Straits can safely support more than 60 tons per square foot. This is four or more times as great as the greatest possible load that will be imposed on the rock by the structure, including the combination of dead load, live load, wind load and ice pressure. The foundations were proportioned to keep the maximum possible resultant pressure below 15 tons per square foot on the underlying rock.

Because the public had been alarmed by unscientific claims that no structure could withstand the ice pressure at the Straits, we added a further generous margin of ultra-safety. According to the most recent engineering literature on the subject, the maximum ice pressure ever obtained in the field is 21,000 pounds per lineal foot of pier width, and the greatest ice pressure producible in the laboratory under controlled conditions for theoretically maximum pressure is 23,000 pounds per lineal foot. We multiplied this higher figure by five, and we designed the piers to be safe for a hypothetical, impossible ice pressure of 115,000 pounds per lineal foot, in addition to all of the usual, conventional factors of safety followed in the best engineering practice.

With the maximum possible ice pressure multiplied by five, and the safe foundation pressure divided by four as a basis for design, the combined factor of safety is twenty for the design of the piers against any possible ice pressure. For still further safety against any possibility of ice damage, the concrete of the piers is protected by steel sheet piling, steel caissons, and armor plate.



ERECTING RING 5 ON CAISSON OF SOUTH MAIN TOWER PIER.

The massiveness of the foundations and the resulting stability against the most severe wind reactions, ice pressure, or any other conceivable loads or forces, are represented by figures like the following:

In each of the main piers, which are 116 feet in diameter, the concrete alone weighs 145,000 tons, and this weight is augmented by 30,000 tons by the reaction of the steel tower superimposed upon the pier, making a total of 175,000 tons.

The total pull of the two cables upon each anchorage is approximately 30,000 tons. To resist this pull, the weight of concrete alone, in each anchorage, is approximately 5½ times as much, or 170,000 tons. This resisting weight is further augmented by the reactions from the adjacent truss spans.

Similarly, because the public had been told that no structure could resist the force of storms at the Straits, the design was made ultra-safe against wind pressure. The greatest wind velocity ever recorded in the vicinity is 78 miles per hour; this represents a wind force of 20 pounds per square foot. We multiplied this force by 2½ and designed the bridge to be ultra-safe against a hypothetical, improbable wind pressure of 50 pounds per square foot, in addition to providing all of the usual, conventional factors of safety established in the best engineering design practice.

The Most Stable Suspension Bridge

The main span at Mackinac is a suspension bridge, which is inherently the safest possible type of bridge. The stiffening trusses are 38 feet deep, or 1/100th of the span length. This is the same ratio adopted (after years of exhaustive aerodynamic tests) for the proposed Severn River Bridge in England, and 68 per cent greater than the ratio of the Golden Gate Bridge.

Even without this generously high depth-ratio, the Mackinac suspension span would have more than ample aerodynamic stability. In fact, by scientific design, utilizing all of the new knowledge of suspension bridge aerodynamics, including the writer's discoveries, analysis, and design principles, the Mackinac Bridge has been made the most stable suspension bridge, aerodynamically, that has ever been designed.

This result has been achieved, not by spending millions of dollars to build up the structure (in weight and stiffness) to resist the effects, but by scientific design of the cross-section to eliminate the cause of aerodynamic instability. The vertical and torsional aerodynamic forces tending to produce oscillations are eliminated.

An important feature contributing this high degree of aerodynamic stability is the provision of wide open spaces between the stiffening trusses and the outer edges of the roadway. The trusses are spaced 68 feet apart and the roadway is only 48 feet wide, leaving open spaces 10 feet wide on each side, for the full length of the suspension bridge. The effectiveness of this feature was demonstrated to the profession by the writer in 1940, and this feature has since been used in the construction or reconstruction of all large suspension bridges.

For further perfection of the aerodynamic stability, the equivalent of a wide longitudinal opening is provided in the middle of the roadway. The two outer lanes, each 12 feet wide, are made solid, and the two inner lanes and the center mall (24 feet of width) are made of open-grid construction (of the safest, most improved type). Wind-tunnel tests have confirmed the high aerodynamic stability of this design of cross-section, combining the two outer openings with an opening in the middle of the roadway.

In addition to the foregoing design features yielding assured aerodynamic stability, maximum torsional stability has been secured by providing two systems of lateral bracing, in the planes of the top and bottom chords, respectively. (This feature has recently been added to the Golden Gate Bridge at a cost of $3,500,000.)

The Mackinac Bridge represents a triumph of the new science of suspension bridge aerodynamics. The design was predetermined scientifically in final form, without spending years in groping, cut-and-try experimentation. Now, two years after determination of the design and award of construction contracts, extensive wind-tunnel tests have finally been completed on a large-scale dynamic model of the bridge. No modification of the design has been found necessary or desirable. The wind-tunnel tests show conclusively, as predicted by the writer, that the Mackinac Bridge, as designed, has:

1. Complete and absolute aerodynamic stability against vertical oscillations at all wind velocities and all angles of attack.
2. Complete and absolute aerodynamic stability against torsional oscillations at all wind velocities and all angles of attack.
3. Complete and absolute aerodynamic stability against coupled oscillations (combining vertical and torsional) at all wind velocities and at all angles of attack.

Professor F. B. Farquharson states in his report that: "Tests at angles of attack up to 20 degrees and over the full range of velocities available (191 miles per hour) have failed to develop any indication of instability."

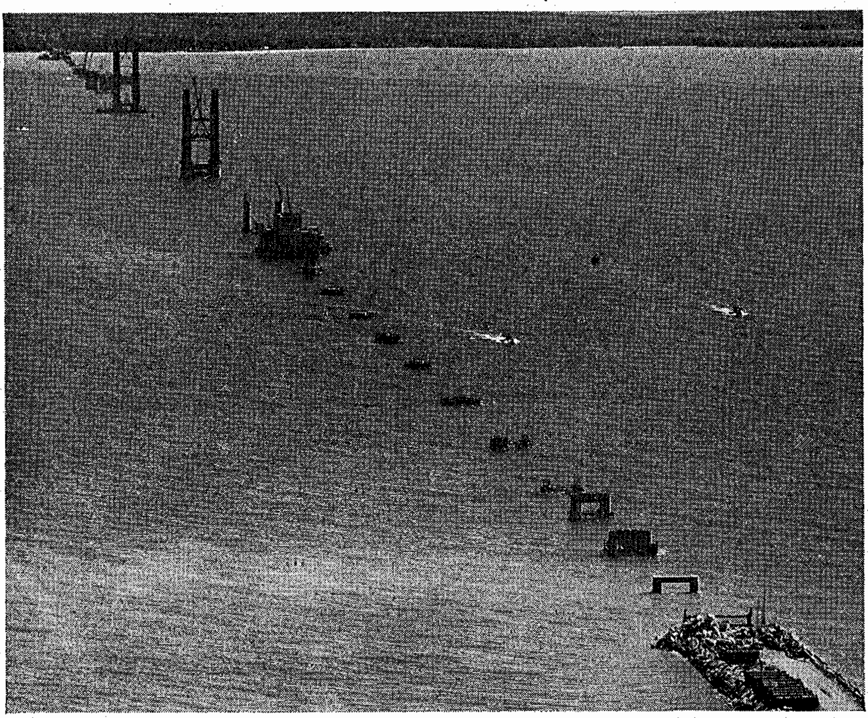
What Makes It Big

The total length of the bridge (including the approaches) is 26,444 feet (5 miles and 44 feet).

The total concrete in the substructure (anchorages, piers, and foundations) is 440,000 cubic yards. Of this amount 350,000 cubic yards are placed under water.

The total weight of the steel superstructure (cables, structural steel, and roadway) is 66,500 tons.

The 33 spans are carried on 34 piers. The two main piers are carried down to depths exceeding 200 feet. (Pier 19 through 140 feet of water and 70 feet of overburden, and Pier 20 through 100 feet of water and 105 feet of overburden.)



AERIAL VIEW OF CONSTRUCTION—MACKINAC BRIDGE.

The substructure contract (lump sum) is $25,735,600, awarded to the Merritt-Chapman & Scott Corporation.

The superstructure contract (structural steel and cables) is $44,-532,900. This is the largest single contract the United States Steel Corporation has ever received; in fact it is the largest single contract in the history of bridge engineering.

Although the suspension bridge appears to dwarf the other spans, continuous-truss spans of notable length are used over a secondary gorge in the crossing. Twenty spans over the deep portions of the waterway range in length from 560 to 330 feet. These span lengths were economically determined by the deep and massive piers required to withstand the ice pressure (with the large factor of safety adopted.)

The use of the Prepakt method for placing concrete in the foundations for the Mackinac Bridge has enabled a new world's record to be established for underwater concrete placement from a single floating plant: 6,250 cubic yards in a 24-hour day.

The wonderful progress that has been recorded on the work despite all difficulties has been made possible by the cooperative teamwork of officials, engineers, and contractors. On this project we have been fortunate in working with the Mackinac Bridge Authority, under the chairmanship of Prentiss M. Brown and under the leadership of Governor G. Mennen Williams. This Bridge Authority has been outstanding for caliber, for competence, for judgment, for integrity, and for ability to put a project through despite all obstacles. Under this inspiring leadership, no effort is being spared to produce the finest, safest and most beautiful bridge that money, skill and brains can build.

THE CHALLENGE

Nature said: "You cannot."
  Man replied: "I can."
From shore to shore, above the tides,
  He built a gleaming span.

Nature said: "You dare not."
  Man replied: "I dare."
He launched his winged ship aloft
  And boldly sailed the air.

Nature said: "You shall not."
  Man replied: "I will."
He caged the thunderbolts of Jove
  And made them serve his skill.

Nature said: "You must not."
  Man replied: "I must."
He split the atom. Now he holds
  A godlike power in trust.

A REPORT ON THE PROGRESS OF CONSTRUCTION OF LITTLETON DEVELOPMENT

By David R. Campbell*, Member

[Presented at a joint meeting of the Boston Society of Civil Engineers and the Hydraulic Section, BSCE, held on November 16, 1955.]

Description and History

The Connecticut River is born amid trout pools and beaver dams in three lakes at the extreme northern tip of New Hampshire, almost at the Canadian border. These bear the unromantic names of First, Second and Third Lake, and the first two have been raised by the New England Electric System to impound 88,000 acre feet of water. A fourth artificial lake was formed just downstream of these by the completion of the Murphy Dam in Pittsburg, New Hampshire, by the New Hampshire Water Resources Board in 1940, and this when full adds an additional 100,000 acre feet to the storage. From Third Lake to the mouth of the river over four hundred miles away, the Connecticut drops twenty-two hundred feet to the sea, but before arriving there it turns countless wheels and lights innumerable homes through the power generated by its falling waters.

A glance at the profile (figure No. 1) of the river gives some conception of the completeness of its development for power purposes. The total installed capacity of the power stations on the main stream including the Littleton Development which I will discuss today will amount to approximately 530,000 kilowatts and if the tributary streams are included the installed capacity will total about three-quarters of a million kilowatts.

For the first eighty miles from its headwaters at Pittsburg, the Connecticut drops steadily and uneventfully, but beginning at Dalton, New Hampshire, there is a succession of rapids extending almost twenty miles which with typical understatement the natives called Fifteen Mile Falls. In this stretch the river falls some 340 feet. This region had long been recognized as having great power possibilities but down through the years it remained unused except for a comparatively small industrial power development at the upper end of the

*Engineer, New England Power Service Co., Boston, Mass.

rapids at Gilman, Vermont, and another near the foot of the rapids at McIndoes, Vermont. During the first quarter of the present century numerous studies were made for various owners in an attempt to design a feasible power development for use either at the site or within reasonable transmission distance. However, the characteristics of both the site and the river made impractical the development

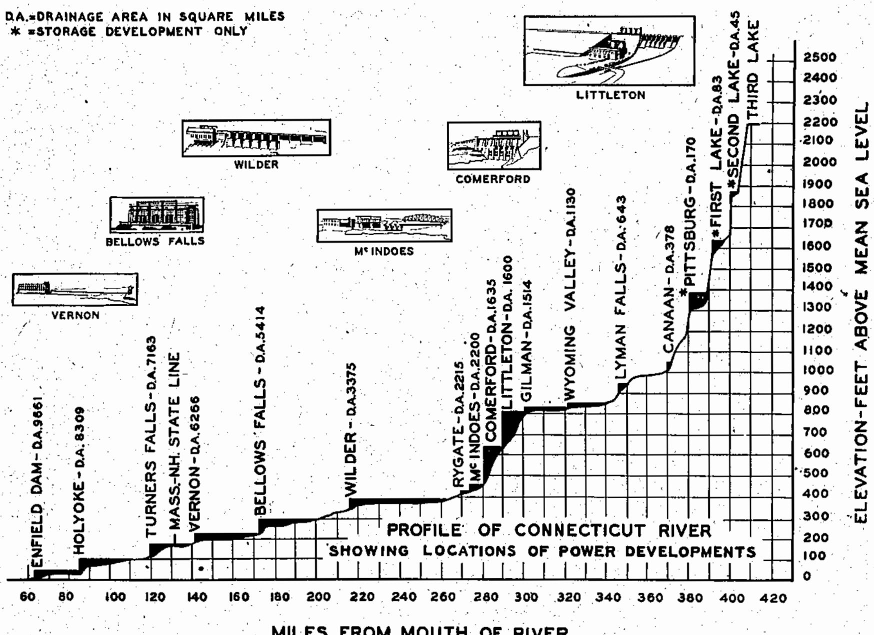


FIG. 1.—PROFILE OF CONNECTICUT RIVER.

of these powers for use in local industries and small utility systems alone.

The New England Electric System, meanwhile, had completed in 1909 a hydroelectric development on the Connecticut at Vernon, Vermont, and during the next twenty years it was successfully demonstrated that the most economical power for New England was obtainable from an interconnected power system including both tidewater steam plants and inland hydroelectric stations.

By the late twenties our power system had developed most of the feasible sites on the Deerfield River and had acquired extensive steam electric resources in southeastern New England. It was apparent even then that hydro power realized its greatest worth when designed to serve the so-called peak loads of the power system and it was at that time that the Fifteen Mile Falls properties were acquired and their development as peak load plants began.

In 1930 the lower section of the falls was harnessed by the completion of the Comerford hydroelectric station with an installation of

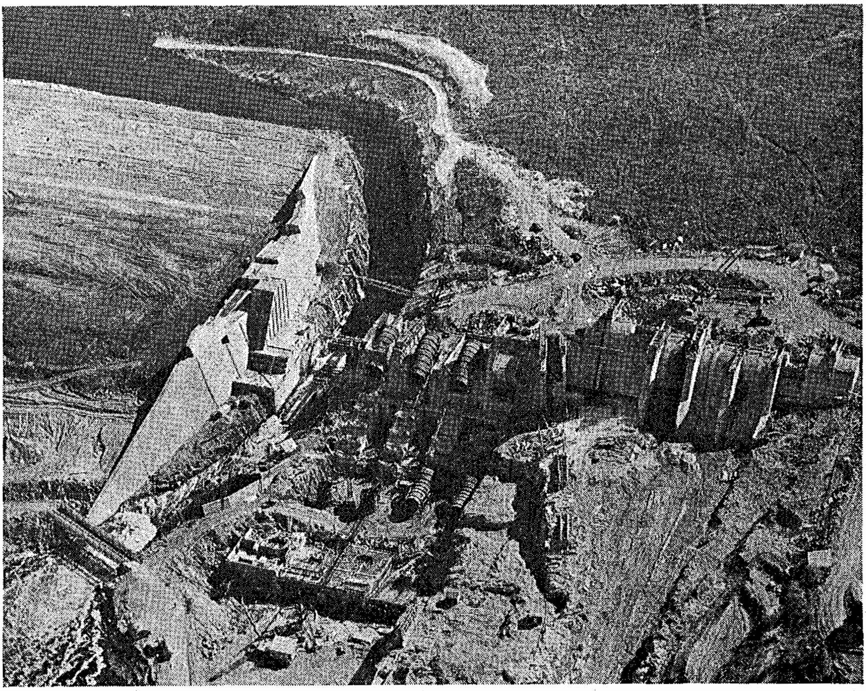


FIG. 2.—LITTLETON DEVELOPMENT—AERIAL VIEW.

150,000 kilowatts, and the Littleton Development (figure No. 2) now under construction at the upper end of the Comerford Pond will complete the utilization of the potential power in this region.

We are now concluding the third year of active construction of the Littleton Development which will be completed in 1956 and dedicated as the Samuel C. Moore Hydroelectric Station. The development will have power capacity identical with the Comerford Station and like Comerford, is designed to serve the extreme peak loads of a large interconnected power network. The development will require over 12,000 cubic feet per second of water when operated at nominal capacity with a full pond. With 1600 square miles of drainage area at the site such a flow is continuously available only during freshet conditions on the river, at which times the plant will operate more or less continuously as a base load plant. During drier periods the plant will normally generate power only a very few hours each day during the peak load hours on the system.

The dam site is located in the townships of Waterford, Vermont and Littleton, New Hampshire, with the river here forming the boundary between the two states. Littleton, New Hampshire, and St. Johnsbury, Vermont, are the largest nearby centers.

Geology

At the site, bed rock outcrops on the New Hampshire shore and continues at or near the surface up the hillside along the axis of the dam. Under the river, however, the bed rock descends rapidly and beneath the Vermont hillside it lies about 90 feet below river level and up to 200 feet below ground surface. Above the ledge in this area the overburden consists of quite impervious glacial till which in turn is overlain by a layer of sand and gravel.

The bed rock at the site was originally volcanic tuffs and lava which have been metamorphosed to a schist which varies from a massive rock with almost no cleavage to a slaty rock with highly developed cleavage. Excavation has revealed that in certain areas the rock has been subjected to punishment possibly by glacial action and had been quite badly shattered. Rock of this nature has generally been removed.

General Layout

The general layout (figure No. 3) of the development called for a large rolled earth dam on the Vermont side and across the river about 2040 feet long and 180 feet in maximum height, a 120-foot high retaining wall set on ledge above the New Hampshire bank, a concrete non-overflow section, approximately 180 feet in maximum height and 115 feet long which spans the channel excavated to divert the river during construction and in which final closure was made, a concrete intake structure 255 feet long and a concrete spillway section 373 feet long, containing a skimmer gate and three tainter gates flanked on each side by two bays of stanchion flashboards. At the southerly end of the spillway there is a concrete structure 120 feet long extending into a small earth dam about 340 feet long. A concrete core wall, 40 feet long, will tie this structure to the earth dike. The total overall length of the dam is approximately 3150 feet or about six-tenths of a mile.

Full pond elevation will be 809 feet above mean sea level and normal tail water at elevation 650.

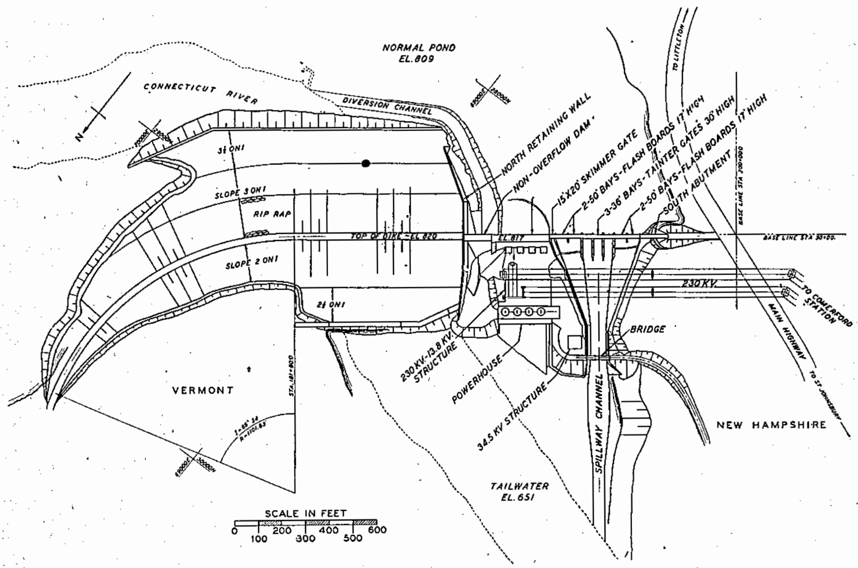


Fig. 3.—General Layout.

The power house, situated downstream of the intake structure, contains four units, developing 150,000 kilowatts at a net average head of 149 feet. Water will be conducted to the turbines by four penstocks approximately 300 feet long and varying in diameter from 21′-6″ at the upper end to 16′-6″ at the scroll cases.

A tailrace excavated in ledge and earth will conduct the discharge back to the river. The spillway will discharge into a side hill channel excavated in ledge and earth with a training wall on the powerhouse side and a short training wall at the upstream end of the opposite side.

Diversion Channel

The first major step in the actual construction of the development was the diversion of the river into an open channel on the New Hampshire side. This diversion channel (figure No. 4) was excavated during the latter part of 1953 and is 50 feet wide at the bottom, about 1600 feet long and 70 feet in maximum depth. Concrete side walls and piers slotted to take timber stop logs were provided in order to facilitate final closure.

Upon the completion of the diversion channel the river was turned from its normal course on December 20, 1953 by cofferdams

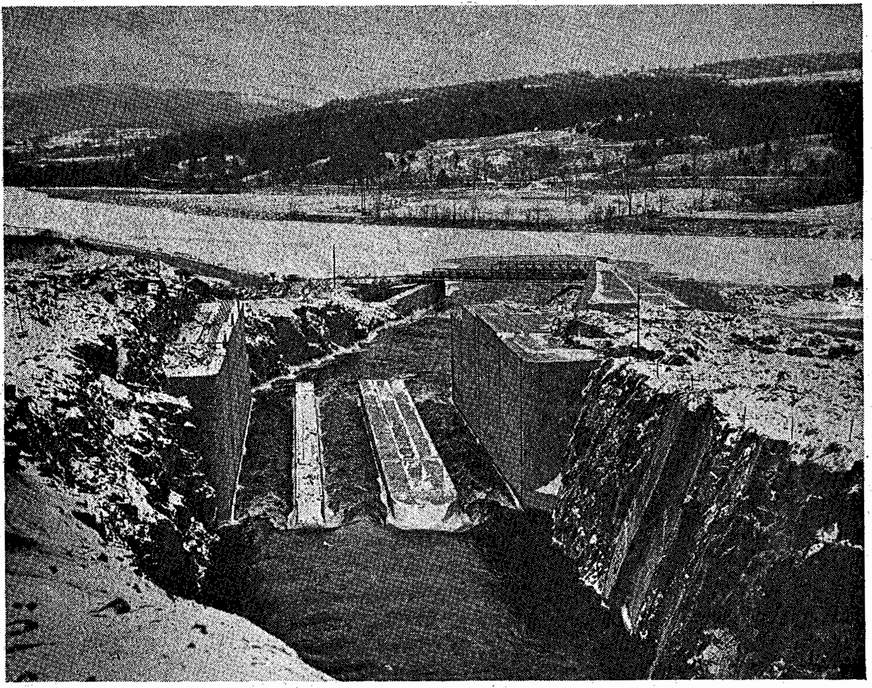


FIG. 4.—DIVERSION CHANNEL.

above and below the area to be occupied by the earth dam. These cofferdams are of the rock fill type with an impervious layer of boulder clay protected by a filter layer on the slope adjacent to the water.

At the New Hampshire end of the upstream cofferdam where the converging entrance to the diversion channel increases the velocity of the river flow, it was felt that difficulty might be encountered in blanketing the face of the boulder clay and consequently a concrete paving was placed in that area to aid in sealing the cofferdam.

We were of course taking a calculated risk in diverting the river before the passage of the spring runoff high water since it was economically impractical to build the cofferdams high enough to withstand the maximum spring flood that might occur. However, they were built to withstand a runoff that was normally exceeded only 25 per cent of the years of record and we felt that if we did not get an exceedingly high spring flood we would thus gain several weeks of valuable time for preparing the foundation of the earth dam.

Actually we lost our gamble, since in April of 1954 the river rose to 35,000 c.f.s., the fourth largest flow on record, and our cofferdam was breached. However, it was quickly repaired and we were able to unwater the site of the earth embankment at least as quickly as if we had delayed diversion until the spring runoff had ended.

Later the cofferdams became a part of the permanent rock toes of the dam.

By contrast, in the spring of 1955 we had a great deal more to lose in the event of a major flood and every possible precaution was taken to be sure that we could safely handle the maximum amount of water which we could conceivably receive. The earth dam was brought up to a height which would accommodate more than twice the largest flow we had ever had and a timber crib, reinforced with steel sheeting, and a concrete flood wall were provided on the power house side of the diversion channel. Our highest runoff amounted to 27,000 c.f.s. in 1955 and it was passed without incident.

Earth Embankment

After the area between cofferdams was unwatered in May of 1954, the river bed was stripped of boulders and loose material and the top soil was stripped from the remainder of the embankment area. A wide trench was then excavated through the overlying pervious material to boulder clay.

It was unfortunate that directly across the river from the area of good ledge outcrop on the New Hampshire side, the overburden on the Vermont side consists of pervious material to a considerable depth. Only a short distance upstream, however, a good impervious boulder clay lies near the surface and extends downward to ledge. Because of this we have moved the Vermont end of the center-line as far upstream as possible while maintaining the New Hampshire end in the area of good ledge outcrop. This has made necessary a curve in the alignment of the dam.

The cutoff trench was excavated as far upstream as practicable, where the underlying impervious material lay at a more shallow depth. This trench was backfilled with boulder clay, compacted by rolling, which connects with an inclined core of boulder clay sloping back to the center of the embankment. The main body of the embankment consists of sandy till, a somewhat less impervious material and the outer faces consist of a shell of pervious sand and gravel.

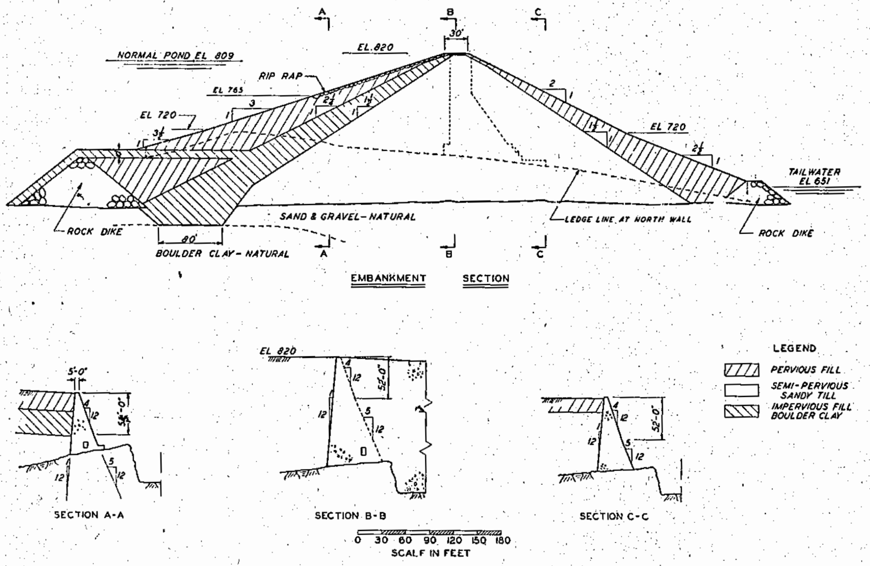


Fig. 5.—North Embankment and Retaining Wall.

The embankment (figure No. 5) has a crest width of 30 feet at elevation 820 with slopes of 1 vertical on 3 horizontal upstream and 1 on 2 downstream for the upper 100 feet changing to 1 on 3½ upstream and 1 on 2½ downstream below that. The upstream slope was protected against wave action by riprap extending 55 feet downward from the crest. The toes of the dam consist of rock fill and filters to provide easy drainage and greater stability.

During stripping of the foundation, a deposit of varved clay was found in the Vermont bank downstream of the center-line of the embankment and extending to a point downstream of the toe. After analysis it was decided to excavate a large part of this material which could be conveniently reached. The remainder was contained by flattening the surface slope with rolled fill starting at the contour at elevation 720 and sloping down toward the river on a slope of 1 vertical on 4 horizontal.

Ample supplies of embankment material were found within a short distance from the site, the principal borrow areas being on the Vermont shore just upstream of the dam.

The embankment was completed in the Fall of 1955 and contains about 3,300,000 cubic yards of material.

Studies were made of the stability of the earth dam under the supervision of Professor D. W. Taylor of M.I.T. and piezometers were placed at various locations and elevations in the embankment and foundation in order to measure pore pressures. These consist of a well point set in a sand pocket which is sealed with grout against leakage from higher elevations, with a ½ inch wrought iron pipe extending to the ground surface. Water elevations in the pipe are read by means of an electric sounding device. Readings have been made at regular intervals, most of the piezometers showing some small fluctuation with tail water elevation. Only at one point has there been evidence thus far of appreciable pore pressure or discernible fluctuation with head water. This piezometer is located at the ledge surface immediately downstream of and below the cutoff trench. Readings in this piezometer indicate pressures amounting to approximately 50 per cent of the total head differential between pond and tail water. Stability studies were therefore renewed assuming the maximum pore pressures which the piezometer readings would lead us to consider possible and these have indicated no serious reduction in safety factor, largely because of the sloping core.

Compaction, shear, and permeability tests were also made of the materials available for construction. During placement of the embankment, a field soils laboratory was set up and numerous field compaction control tests were made, one density determination being made for each thousand yards of fill in the early stages of the work with this number being reduced as experience of the materials and equipment was gained. Mechanical analyses and optimum moisture content determinations were also made.

At the height of the embankment placing operations the equipment in use included forty bottom dump and rear dump Euclids, seven Euclid and six Caterpillar scrapers, 15 bulldozers, nine shovels and a huge fleet of trucks. Maximum yardage placed in a single day of two 10-hour shifts amounted to 32,000 yards and a maximum of 744,000 yards were placed in a single month.

Retaining Wall

The retaining wall which supports the embankment is of gravity section and extends 376 feet downstream and 400 feet upstream from the axis of the dam. Fifty-five thousand cubic yards of concrete were placed in its construction which was completed in August of 1955. A grouting and inspection gallery has been provided in the base of the wall, as well as in the intake, non-overflow and spillway structures.

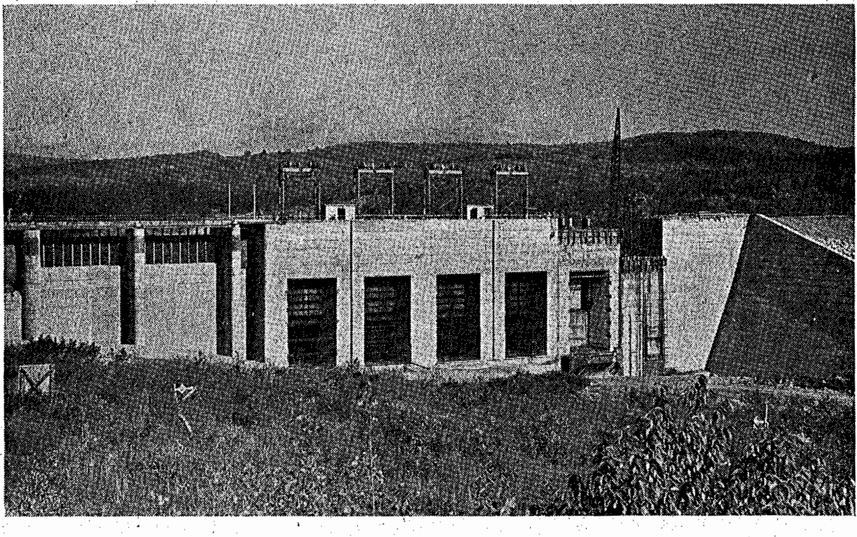


Fig. 6.—Intake and Non-Overflow Structure During Construction.

Intake and Penstocks

The intake (figure No. 6) is of gravity design and provides for four steel penstocks controlled at the upper end by wheel gates, 18 feet wide by 24 feet 3 inches high, with 60-ton electric hoists. The sills of the gates are at elevation 725 to permit plant operation under a 40-foot drawdown. The intake openings are provided with racks for the full height, composed of round edged flat bars, spaced 3 inches in the clear, and carried on steel supporting members. Racks and supports were designed for a ten-foot difference in head to allow for some clogging of the racks. A compressed air cleaning system has been installed and will be used in conjunction with a truck crane for handling trash.

The penstocks are located on 50-foot centers and weigh about 350 tons each. They are of all welded construction and are designed for a maximum stress of 13,750 pounds per square inch with 85% efficiency of welded butt joints. They were designed for the static head, plus a pressure rise above pond level varying from zero at the intake to 80 feet of water at the outlet end. To the thicknesses thus determined, one-sixteenth inch was added for corrosion, resulting in a total plate thickness of $\frac{5}{8}''$ at the upper end and $\frac{15}{16}''$ at the lower. A hydrostatic pressure test was made on each penstock to 150 p.s.i. pressure at the lower end after which the penstocks were painted inside and out and an earth cover was placed over them to prevent ice formation.

A model of the intake was made at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute and numerous tests made to determine the best shape and dimensions for the approaches to the penstocks.

Spillway

The spillway dam located on the New Hampshire river bank is 373 feet long and from 40 to 100 feet in height (figure No. 7). A bridge has been built over the crest and its piers reduce the net length of the spillway to 323 feet.

The skimmer gate, 15 feet wide by 20 feet high, is located next to the intake. It is mounted upstream of a concrete ogee section with its crest or gate still at elevation 789. This gate is of the dropping type and is controlled by an electrically operated, motor-driven double stem screw hoist. Its primary function will be passing the debris which may collect in the head water pond.

Next in the spillway are two bays, each 50 feet long, of stanchion type flashboards, 17 feet high, with a six-foot pier between bays. The crest in these bays is at elevation 792 with profile designed for an overflow of about 21 feet. The stanchion flashboards consist of vertical steel beams on about 5-foot centers with horizontal timber stop logs between beams. The bottoms of the beams are set in sockets in the concrete crest and the tops are attached to the bridge structure and so constructed that they can be readily released by one man in case of emergency.

In the center of the spillway are three bays each with a steel tainter gate, 36 feet long by 30 feet high, with 8-foot thick piers at either side of the bays. The dam crest at the gates is at elevation 779 with its profile designed to give proper nappe conditions for overflows up to 36 feet deep.

Each gate has a fixed, electrically operated, motor-driven hoist and is provided with electric heating to prevent ice formation from

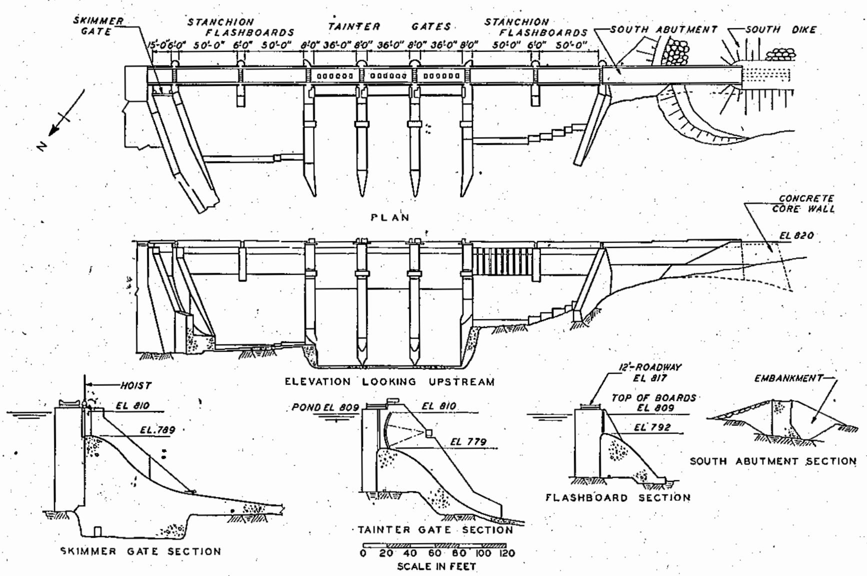


FIG. 7.—SPILLWAY—PLAN, ELEVATION, AND SECTIONS.

interfering with its operation. In addition, a compressed air bubbler system prevents ice from forming in the water in front of the gates. An auxiliary power supply has also been provided to insure our being able to operate the gates even in the event of a power failure during flood conditions.

Proceeding southerly we next have two more 50-foot long bays of stanchion flashboards.

The spillway has a discharge capacity of about 120,000 c.f.s. with the pond level held at elevation 809 which will be normal pond elevation. This leaves 11 feet of freeboard. The generating units themselves will pass better than 12,000 second feet.

As contrasted with this total flood capacity, the maximum flood of which we have record in this vicinity was about 50,000 second feet in 1936. The tainter gates alone have a greater capacity than this, so that throughout the 55 years of record at no time would it have been necessary to remove flashboards.

The adjoining structure will be a concrete section, 120 feet long, extending into a small earth embankment with a central core of boulder clay. A 40-foot long concrete cutoff wall will serve to tie this structure to the earth dam. The earth dam here has a top width of 25 feet with slopes of 1 on 2 downstream and 1 on 3 upstream.

The spillway discharges into a rather long and steep channel in which it was obvious that we must expect very high velocities. We therefore had constructed at the Alden Hydraulic Laboratory a scale model of the entire spillway, powerhouse, tailrace, spillway channel and the river below the channel and tests were conducted upon this model continuously for over a year in order to study the problem of dissipating the energy in the water, reducing turbulence and preventing any harmful erosion, and to obtain the best possible design of all the components which made up the flood passing structures of the dam.

This model proved valuable in many ways and paid for itself many times over in economies we were able to effect through analysis of the test results.

The channel is about 1200 feet long and excavated in ledge throughout most of its length. It slopes toward the river at a 15 per cent incline.

As excavation proceeded there was some concern about the seamy condition of the ledge in the channel area and it was decided to pave the major portion of the bottom and sides of the channel with a minimum thickness of one foot of concrete.

At the lower end of the channel where it re-enters the river, the ledge drops off sharply and continuing the excavation to ledge through deep overburden or attempting to construct a stilling pool deep enough to create a hydraulic jump would necessitate a very substantial and expensive quantity of excavation. We therefore decided to excavate only a pilot channel through this area and let the water do our excavation for us over the years. The model studies have indicated that there is little tendency for the eroded materials to build up in front of the tailrace to affect tailwater elevation and the eroded material will be harmlessly deposited in the Comerford pond.

POWER HOUSE AND HYDRAULIC EQUIPMENT

The power house (figure No. 8) is of steel frame and brick construction on a concrete substructure. It is approximately 256 feet long by 52 feet wide with additional width of 25 feet below the generator floor and with a 20-foot wide extension upstream over the penstocks and beneath the transformer yard.

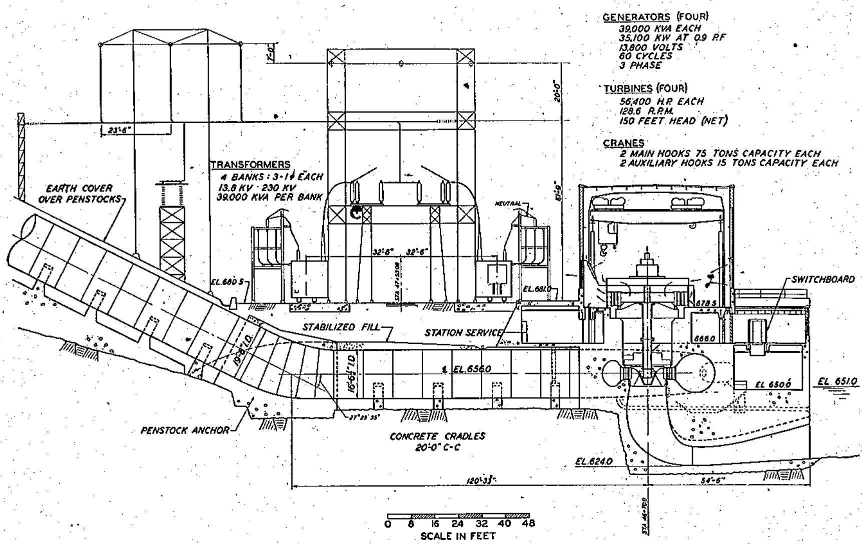


FIG. 8.—POWERHOUSE AND INTAKE—TYPICAL CROSS SECTION.

The turbines and generators are located on the upstream side of the powerhouse with the switchboard and operating room downstream. The generator floor is at elevation 678.5, with the operating floor at elevation 666, and the basement floor at elevation 650.

Access to the power house is from a station yard at elevation 680.5 which is reached by means of a bridge across the spillway channel.

Panels of glass block and light buff brick of varying shades were used for the exterior of the superstructure to harmonize with the surrounding masses of concrete. The interior walls will be of light buff tile and red quarry tile will form the surfacing of the operating floor.

An extensive study was made of the relative merits of an outdoor, semi-outdoor or fully enclosed power house but we were unable to justify use of either the outdoor or semi-outdoor type either economically or from an operating point of view. Our studies of stations of these types, however, led us to rather a unique conception of power house design. Fundamentally, we have designed our power house as if it were to be one of the outdoor types with the control room, switchboard and all other appurtenances enclosed in the substructure below the generator floor level. This has resulted in making the superstructure merely a shelter for the cranes and generators. We thus eliminated the need for an expensive outdoor gantry crane, and for hatches or generator envelopes and at the same time provided shelter for men and equipment during periods of repair at a substantial saving in cost over a more conventional design.

The four turbines are vertical, single runner Francis type wheels of S. Morgan Smith manufacture, direct connected to the generators. Turbines are rated at 56,400 H.P. each and 3970 c.f.s. under 150 feet net head at 128.6 r.p.m. This is full load rating and therefore exceeds the requirements for normal operation at best efficiency. They will be set in plate steel riveted scroll cases which are connected to the welded steel penstocks. The draft tubes are formed in the concrete of the power house substructure with plate steel liners in the upper portion.

The turbine guide bearing will be of the babbitt lined, partially immersed, self lubricated type, with oil circulation provided by viscosity pumping action through grooves in the bearing shell. Oil from the immersion reservoir at the bottom of the bearing will be forced up the oil grooves by the rotating shaft to the top of the bearing and thence will overflow back into the reservoir. The generator has a Kingsbury type thrust bearing below the rotor with the guide bearing combined on the thrust bearing collar which is forged integral with the shaft. These bearings are in a common oil pot with water circulation coils for cooling. Indicating thermometers with alarm attachments are embedded in each bearing.

The turbines will be equipped with the latest Woodward twin cabinet actuator type governors, each unit having a capacity of 259,000 foot-pounds at 300 pounds pressure. The actuators and pumping units are mounted on a fabricated steel sump base as an integral part and the whole assembly enclosed in a cabinet with all gauges and controls mounted on the front panel. The actuator and oil pressure systems of each pair of units are so interconnected that either pump or either pressure tank may be used to supply the oil requirements of either or both units or they may be operated separately. Each governor oil pump has a capacity of 200 gallons per minute at 300 lbs. pressure and is driven by a 50 H.P. motor. The governor fly balls are motor driven by the ball head motor which receives its power supply from the permanent magnet generator mounted on top of and directly connected to the main generator. The speed indicator, or tachometer, will be mounted on the actuator cabinet, and is actuated by the permanent magnet generator through a rectifier-resistor filter pack.

Electrical Equipment and Layout

The turbines will drive four umbrella type, vertical, air-cooled Westinghouse generators, each rated 39,000 KVA, 0.9 power factor, 3 phase, 60 cycles at 128.6 r.p.m. Each generator has its own direct connected exciter and pilot exciter.

A very complete system of ventilation is provided for cooling the generators and heating the power house. Outside air is drawn into the upper portion of the power house through adjustable louvers set in the downstream wall and one end wall of the power house. The air is then drawn into the individual machines, each generator requiring 80,000 cubic feet of air per minute. A total of fifty-four washable filters are mounted on top of each generator to filter the air before it enters the generator. The fans for drawing the air into the building and circulating it through the generator and exhaust air ducts are mounted on the generator rotor. After the air passes through the rotor and the stator coils it is discharged through the sides of the stator frame into a discharge air duct and thence outside of the building through another set of adjustable louvers located in the upstream wall of the power house. This eliminates any chance of short-circuiting the air circulation. Auxiliary dampers in the discharge air duct and in the generator room floor allow warm air to be discharged within the power house for heating and then re-circulated through the machines.

This system of air cooling the generators represents a departure from prevalent methods, since a great majority of modern units are of the water-cooled type. We were unable, however, to achieve any economy by using the water-cooled type. The air-cooled machines have the additional advantage of providing building heat during cold weather.

The generator room floor is served by two 75-ton traveling electric cranes, each with 15-ton auxiliary hooks. The cranes can be coupled together and by means of a lifting beam can be used to handle the generator rotor which is the heaviest piece of equipment. Each generator will be connected in a unit system with its own step-up transformer bank consisting of three General Electric single phase, water-cooled units, each rated 13,000 KVA connected 13,400 volts delta and 230,000 volts wye. Water for cooling the transformers will be supplied from the tailrace by two 1750 r.p.m. motor-driven pumps of 450 gallons per minute capacity, each under a 75-foot head.

Low voltage and 230 KV steel switching structures, including the main transformers, are located outdoors, upstream, close to the power house structure. Power supply from Littleton will be transmitted into the 230 KV system at Comerford Station by means of two single circuit 230 KV wood pole lines. Generation at Littleton will also furnish power supply into the 34.5 KV system in the Littleton area. Each of the 230 KV lines will be 6.7 miles long and will be equipped with expanded aluminum conductors having a cross section of about 0.4 square inches and an outside diameter in excess of one inch.

The first few hundred feet of the 230 KV line outlets of Littleton present a special problem because they cross over the spillway channel. It is recognized that turbulence of the water during spillway discharge in cold weather might cause excessive ice formation on any overhanging wires. We are therefore installing an insulated heating element in the conductor that by suitable switching can be made to carry all the current for such length of time as is necessary to melt or prevent ice formation.

Station service power will be obtained from two 500 KVA, 13800/480 V transformers, each connected to a main generator circuit through 15 KV fuses. Station service transformers will feed a control center in the power house to serve various 440 V station auxiliaries. A second 440 V control center will be located at the dam, fed from the power house control center to serve various motor-driven gates and hoists at the dam.

Foundation Treatment

Because of the seamy character of the ledge in some areas, a great deal of attention has been given to the treatment of the foundation of the dam. A grout curtain has been established beneath the upstream portions of all the concrete dam structures and beneath the cut-off trench of the earth dam for the distance in which it was carried to ledge. Grout holes were drilled from the toe blocks and galleries of the various structures to varying depths up to 100 feet below the surface, and grout pressures up to 130 pounds per square inch were used. Primary grout holes were spaced on twenty-foot centers with secondary holes on ten-foot centers and tertiary holes on five-foot centers.

In addition, in some areas, consolidation grouting was done by drilling patterns of holes and washing between them after which the entire group of holes was grouted.

Relief holes were also drilled extending from box drains in the base of the concrete sections.

Over 35,000 linear feet of grout holes have been drilled and over 6000 bags of cement used in the grouting operations. An additional 5000 feet of relief holes have also been drilled.

Concrete

A source of concrete aggregates was found a short distance from the site on the New Hampshire side. This consisted of an esker composed of sand and gravel which was washed, crushed and graded in an aggregate processing plant. The maximum size of stone used was six inches.

Concrete was mixed in an automatic batch type mixing plant containing six bins for storing aggregates. On a lower floor a pair of two-yard tilting type mixers batched the concrete. Adjacent to the mixing plant are 3 cement silos with screw feed and bucket elevator.

A fleet of six trucks, carrying four-yard buckets transported the batched concrete to the point of placement. Here one of two five-yard Manitowoc Speedcranes with 140-foot booms lifted the bucket and transported it to the forms. These were the first such machines to see service in New England. A stiff leg derrick has also been used to transport the concrete buckets in some areas.

Cement was Type II Portland cement and was supplied by several companies to assure a continuity of supply. It was shipped in bulk by rail and stored at a siding which was built in the Town of Littleton. The cement was inspected and tested at the manufacturers' plants prior to shipment.

The proportions for concrete were fixed by Thompson & Lichtner Co. in collaboration with a field laboratory which was completely equipped with testing apparatus and storage room. The specifications call for grades of concrete based on strength at 28 days varying from 2500 p.s.i. for the mass structures to 3500 p.s.i. for thin sections, floors, etc. An air entraining agent was added at the mixers.

Almost a quarter of a million cubic yards of concrete have been placed and on the bigger pours the rate of placement ran to 110 yards an hour and 1800 yards in a single 24-hour period.

Closure

The first steps toward the final closure of the dam began with the advent of low river flows in July of 1955. The concrete piers which had been formed in the diversion channel were raised to elevation 700 and a 10′ x 10′ sluice was formed in one pier at elevation 671, with a 10′ x 10′ Broome Gate installed at the upper end. Concrete was then placed in the three gaps between piers and sidewalls by successively stop logging one opening after another and pouring lifts of different height in such a manner that the water could always be passed through two openings while the third was closed off by the stop logs. This was necessary because the requirements of downstream water users made it necessary to pass some river flow almost continuously.

In order to guard against erosion of the ledge in the diversion channel near the base of the retaining wall by the water discharging from the sluice, we attempted to dissipate much of its energy within the structure. This was done by introducing several sharp 90-degree turns in the sluice passageway. Water entering the sluice opening in the pier was turned at right angles toward a vertical shaft, thence downward and again at right angles to a 10′ x 15′ opening which was left at the base of the structure between the two piers. A model made at the Alden Laboratory indicated that this scheme did reduce the exit velocity of the water.

A second sluice was installed at elevation 720 but since river flow has remained very low since closure began, the water had not approached the sill of the upper sluice when we finally reached the stage where we were able to use the penstocks for passing water. The upper sluice was therefore filled with concrete.

When the pond elevation rises to where we can pass down-river requirements through the penstocks, the broome gate will be closed and a concrete plug poured behind it. River flow will then be controlled by the intake gates until the pond reaches the sill of the tainter gates, after which it can be controlled by the tainter gates while work on the units is completed.

Basin Work

The dam will create a pond approximately 11 miles long and will impound 115,000 acre feet of usable storage with a maximum drawdown of 40 feet. The area of the pond will be about 3500 acres when full or about three-fourths the size of Newfound Lake in New Hampshire.

The creation of the pond has necessitated considerable work in the basin area. A main trunk line of the New England Tel. & Tel. Co. crossed the valley immediately above the dam site and this has been relocated. The main State Highway from Littleton, New Hampshire, to St. Johnsbury, Vermont, also crossed the area to be covered by the pond and about 5 miles of this highway has been relocated. Several town and county roads have also been relocated.

Three cemeteries which would have been inundated by the pond also were relocated and a total of 2400 acres have been cleared of trees and brush in preparation for filling the pond. We expect also to provide recreational areas along the shore, as we have done at other system properties, to accommodate fishing, boating and picnicking.

Organization

The work is being carried out for the New England Power Company, a subsidiary of the New England Electric System. All design, engineering and supervision for the development is being performed by the New England Power Service Company, the service organization for the New England Electric System, except the plans for the power house superstructure and access bridge which were made by Chas. T. Main, Inc., who have also been retained as Consulting Engineers. Ebasco Services, Inc. of New York has been retained as Construction Managers.

The Thompson & Lichtner Company is serving as concrete consultants and Professor D. W. Taylor of M.I.T. as soils consultant. Geological investigations are under the direction of Irving B. Crosby, Consulting Geologist, and model testing at the Alden Hydraulic Laboratory was under the supervision of Professor L. J. Hooper of Worcester Polytechnic Institute. Pittsburgh Testing Laboratories covered the inspection and complete radiographing of all penstock welding.

Contracts for the excavation of the Project and for the construction of the earth embankments have been awarded to B. Perini & Sons of Framingham, Massachusetts.

Contracts for concrete masonry construction and installation of mechanical equipment have been awarded to Morrison-Knudsen Company, Inc. of New York.

All electrical equipment will be installed by the New England Power Service Company.

Many other contractors are being employed on various parts of the work including H. B. Cummings of Woodsville, New Hampshire, on superstructure erection, Kenneth E. Curran, Inc. of Littleton, New Hampshire, on job buildings and many miscellaneous sub-contracts, Walsh-Holyoke Division of Continental Copper & Steel Industries, Inc. of Holyoke, Mass., on penstock fabrication and erection and Joy Manufacturing Company of Michigan City, Indiana, for diamond drilling.

An observation stand was built upstream of the site for the convenience of the public and over 200,000 persons have viewed the construction from this vantage point. A permanent visitors' house and promenade will also be built immediately downstream of the dam.

The development will share with our Comerford Plant the distinction of being the largest hydro-electric station in New England and, situated as it is in a beautiful natural setting of green hills and rolling meadows, can take its place proudly among the many scenic attractions of Northern New England.

A COOPERATIVE APPROACH TO A STREAM POLLUTION PROBLEM IN MASSACHUSETTS

By Ross E. McKinney*

[Presented at a joint meeting of the Boston Society of Civil Engineers and the Sanitary Section, BSCE, held on December 7, 1955.]

One of the major problems facing industry today is stream pollution. The rapid expansion of industry following World War II has resulted in increased production, as well as increased quantities of waste waters. In many industrialized areas it has been possible to dispose of the liquid organic wastes into municipal sewers along with the domestic sewage. The combined wastes are treated in the municipal sewage treatment plant before final disposal to the receiving body of water. This type of operation is the ideal solution to industrial waste disposal both from the industrial and municipal standpoint. Each contributes its share of the capital investment and the operating expenses. But as in any ideal solution to waste disposal problems, it can be abused by either or both sides failing to cooperate with one another.

There are many industrial areas where adequate municipal facilities are not available to handle the combined sewage and industrial wastes. These are the areas where the majority of waste disposal problems are occurring. Normally, the liquid organic wastes from the various industrial processes are run directly to a nearby receiving body of water for ultimate disposal. If the organic content of the wastes is small and the dilution in the receiving body of water is large, satisfactory stabilization of the organic matter can take place without the creation of nuisance conditions. On the other hand, if the organic content of the wastes is high and the dilution factor is small, the receiving body is unable to assimilate the organic matter without creating nuisance conditions such as destruction of fish and obnoxious odors. The presence of nuisances of this type leads to complaints from abutting property owners. The complaints are usually referred to the State Health Department for appropriate action. In most states the State Health Department is charged with stream pollution control; but some states have given both the State Health Department and the

*Assistant Professor of Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

Fish and Wildlife Commission authority over stream pollution depending on whether the problem is one primarily of health or the destruction of fishlife.

Once a nuisance has been created the State Health Department's first job is to determine the source of the problem. A brief stream survey is usually all that is necessary to ascertain the source of the wastes. The industry or industries creating the wastes are notified of the situation and are requested to alleviate the nuisance. It is at this point that most of the misunderstandings between the state officials and the industry result. Oftentimes the State Health Department issues an order that the nuisance is to be eliminated by a certain date and then sits back and waits for the problem to be solved. If the problem has not been solved by that date, the industry usually finds itself facing a court order prohibiting further waste discharges. This can be tantamount to an order closing the plant since the industry cannot operate without producing wastes. Some industries feel that wastes are a natural byproduct of operations and that disposal of these wastes should not be their problem. These industries usually ignore the State Health Department's order or make only a small effort to solve the problem. For the most part, these are delaying tactics since the industry knows that State Health Departments will resort to court orders only after considerable effort has been exhausted in finding an amiable solution. A mild effort towards a solution will usually stall the State Health Department even longer. By playing the game to the hilt, the industry can postpone treatment of their wastes for many years. And by using their trump card of threatening to move to another area of the country, the industry can even scare the politicians into conditions favorable to the industry. Needless to say, both examples are extremes but both reflect the attitudes which have been used in stream pollution problems in various parts of the country. There are various shades of intermediates between these two extremes; but there is only one real solution to stream pollution problems and that is cooperation between industry and state.

When an industry moves into an area, it has something to offer that community. The two major benefits, increased employment and additional taxes, tend to stabilize the community by competing for the local labor force, bringing in additional citizens, increasing the total money in circulation, increasing the demand for consumer goods, increasing the money available for community expansion and capital investments in municipal financing. In return the industry expects something from the community in good municipal government, adequate housing for employees, adequate transportation facilities, for employees and for shipping their products to markets, reasonable cost of living and acceptance as being part of the community. The success of the community and its industries is dependent upon their cooperation on all problems. Industrial waste disposal is one of these problems. It does not belong entirely to the industry nor entirely to the community, it belongs to them both. Alone, there can be no satisfactory solution to the problem but together there can be.

An example of excellent cooperation between the State Health Department and industry can be cited for the stream pollution problems existing on the Chicopee River in central Massachusetts. The problems have not all been solved but the pattern for their solution has been set and there is little doubt that the nuisance conditions now being created can be satisfactorily alleviated.

The Chicopee River is formed from the combined flow of the Ware River, the Quaboag River and the Swift River. The Ware River begins north of Barre, Massachusetts, and is joined shortly by the Burnshirt River which has its head waters just over the Massachusetts border in southern New Hampshire. The Ware River meets the Quaboag River just south of Thorndike, Massachusetts, and becomes the Chicopee River. The Quaboag River starts below Brookfield, Massachusetts, and flows past Warren and Palmer before meeting the Chicopee River. The Swift River is made up primarily of water released from Quabbin Reservoir and meets the Chicopee River at Three Rivers, Massachusetts. The Chicopee River flows past Ludlow, Springfield and Chicopee before emptying into the Connecticut River. The drainage area of the Chicopee River covers 688 square miles and yields a two (2) per cent flow of 86 c.f.s. Like most of the rivers in New England, the Chicopee River is dotted with dams which regulate the flow of the river. Many of these dams were built by industries which found that the rapidly flowing Chicopee River offered a good source of power. The Swift River and the Chicopee River fall from an elevation of 530 feet at Quabbin Reservoir to an elevation of approximately 50 feet at the Connecticut River.

The Chicopee River broadens out just east of Chicopee Falls as a result of a dam at Chicopee Falls, Figure 1. The islands help create unusual flow patterns and stagnant areas in the river. It was at this point that nuisance conditions resulted. Additional nuisance conditions also resulted below the dam and along the river to its junction with the Connecticut River. A survey of the Chicopee River and its tributaries by the Massachusetts State Health Department indicated that the small industries located along the upper reaches of the Chicopee River were not making any significant contributions to the nuisance conditions arising around Chicopee and Chicopee Falls. The nuisance conditions below Chicopee Falls were found to be caused by domestic sewage and industrial wastes from Chicopee and Chicopee Falls. This condition will be alleviated by the construction of an in-

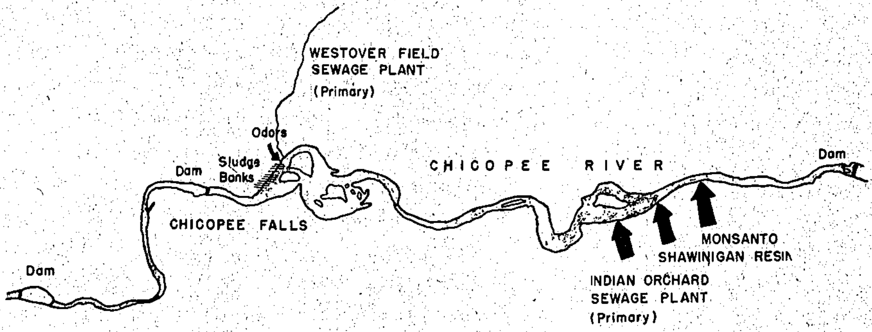


FIG. 1.—MAJOR SOURCES OF POLLUTION ON CHICOPEE RIVER BETWEEN INDIAN ORCHARD AND CHICOPEE FALLS.

tercepting sewer along the Chicopee River and a treatment plant which will discharge its effluent into the Connecticut River. The nuisance conditions above the dam at Chicopee Falls were found to be caused by the industrial wastes from Monsanto Chemical and Shawinigan Resins along with the settled domestic sewage from the Indian Orchard Sewage Treatment Plant. In the summer of 1948, both Monsanto Chemical and Shawinigan Resins were notified by the Massachusetts State Health Department that the discharge of their wastes into the Chicopee River was creating nuisance conditions and that appropriate action should be taken at the plants.

Shawinigan Resins is a subsidiary of Shawinigan of Canada and Monsanto Chemicals; but the Monsanto Chemicals plant and the Shawinigan Resins plant at Springfield are operated as separate divisions. Shawinigan Resins produces plastics from the vinyl chain. Safety glass interlayer and wire insulation enamel are the primary usages of the plastics produced at this plant from the basic raw material of vinyl acetate. The wastes from the various processes contain short chain simple organic compounds resulting from the degradation of vinyl acetate and are almost entirely soluble.

When Shawinigan Resins received notification of pollution of the Chicopee River in 1948, the Research Department took composite samples of all of their waste streams. A portion of the composite sample was neutralized to pH 8.5 with lime and aerated for two hours. A second portion was neutralized to pH 8.5 and aerated for 16 hours. Treated and untreated effluent samples were submitted to the chemist of the Springfield Sewage Treatment Plant for 5-day biochemical oxygen demand (B.O.D.) analyses. The results were as follows:

Sample	*5-Day B.O.D.*
Raw	650 ppm
Neutralized, 2 hours aeration	190 ppm
Neutralized, 16 hours aeration	160 ppm

The data showed promise for a relatively simple chemical treatment of the wastes and was presented to the State Health Department at a meeting later that year. Other methods of treatment were discussed at that meeting. The State Health Department officials suggested that biological treatment on trickling filters might prove satisfactory and offered the assistance of the Lawrence Experimental Station in making preliminary tests.

A sample of wastes from the strongest waste stream was sent to Lawrence and found suitable for biological treatment. The State Health Department suggested that further studies be made at Lawrence and that a pilot plant be built at Shawinigan based on the expected results of the Lawrence experiments. Late in January, 1949, a 50 gallon drum of wastes from the strongest waste stream was sent to Lawrence where lime treatment was found to result in a 50 per cent B.O.D. reduction. The remaining B.O.D. was readily stabilized on trickling filters. In April, 1949, two 50 gallon drums of wastes from other waste streams were sent to Lawrence to try on their trickling filter pilot plants. The State Health Department submitted a report on the Lawrence studies in September, 1949. The report recommended that a trickling filter pilot plant be set up at Shawinigan

Resins in order to determine operating characteristics which could be translated into ultimate design. Since none of the personnel at Shawinigan Resins was familiar with biological treatment methods, the State Health Department offered to advise the Engineering Department on the design and operation of the pilot plant.

By June, 1950, the pilot plant had been designed; the expenditure approved and the materials ordered. The schematic layout of the pilot plant is shown in Figure 2. The translation of the plans into

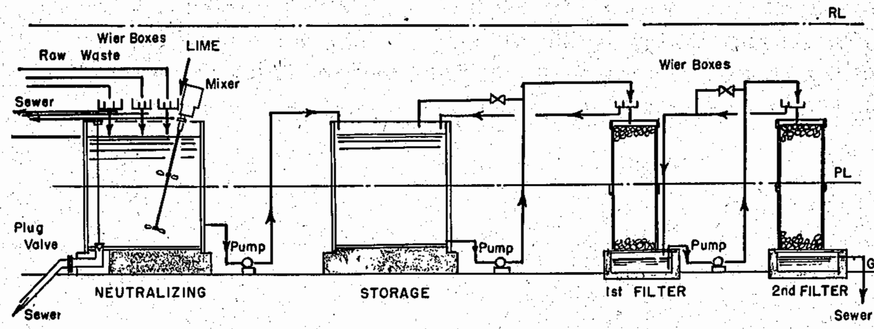


FIG. 2.—SCHEMATIC LAYOUT OF SHAWINIGAN RESINS TRICKLING FILTER PILOT PLANT.

the actual pilot plant was not an easy task and it was June, 1951, before the pilot plant was put into operation. Actually, two pilot plants were built, one having 12 inch trickling filters and the other having 24 inch filters.

The trickling filters were started on domestic sewage obtained from the Indian Orchard Sewage Treatment Plant. After 12 days operation considerable growth had built up on the stones and the filters were ready for plant wastes. The pH of the initial wastes was 3.1 and was adjusted to 6.7 with lime. Ten per cent raw wastes were fed with sewage at the rate of 13.3 M.G.A.D. on the 12 inch filter and 7.7 M.G.A.D. on the 24 inch filter. The filter growth increased rapidly as the load was slowly raised. After 8 days operation the rate of application was cut to 3.2 and 3.3 M.G.A.D. loadings and 100 per cent plant wastes were fed to the filters. As in the case with most pilot plants, troubles developed and it became impossible to obtain wastes from the strongest waste stream. But the filters operated satisfactorily on the remaining wastes. Four days after starting the 100 per cent waste feeding, the rate of application to the 12 inch filter was raised to 7.2 M.G.A.D.

The filters were operated successfully over the next two months with an average influent B.O.D. of 450 ppm and average effluent of 250 ppm on the primary filter and 170 ppm on the secondary filter. The major problem encountered was the tremendous quantity of algae and fungi which overgrew the pilot plant and created an almost continuous maintenance program to keep the filters free and in proper operation. Anaerobic conditions set in during the summer and obnoxious odors were present. In September, 1951, a portion of the strong wastes was added to the filter. After a month's operation cold weather set in and the filters froze.

Based on the data that was obtained on the filters and on strength of the raw wastes, it appeared that loading the filter at an organic load of 0.62 pounds B.O.D. per cu. yd. stone per day and a hydraulic load of 3.2 M.G.A.D. would reduce the strength of the wastes 60 per cent. Shawinigan Resins personnel did not think that this reduction would be adequate and focused attention on a preliminary chemical treatment followed by trickling filters.

Since composite samples of the plant wastes indicated that the wastes entering the Chicopee River contained 9,000 pounds 5-day B.O.D. daily, a study was made to ascertain the source of all of the wastes, their volume and their strength. It was found that the wastes contained ethyl acetate, ethyl alcohol, acetic acid, formaldehyde, butyraldehyde, ammonium hydroxide and sulfuric acid. The enormity of this task was such that it was September, 1952, before the filters were started again. They were acclimated in the same manner as before. While the filters were being acclimated, aeration studies were carried out on the strongest wastes, still house wastes, to determine if the volatile compounds could be stripped out easily. It was found approximately 32 per cent of the 5-day B.O.D. of the "B" still wastes could be removed by aeration and that an over-all efficiency of 89.6 per cent removal was possible by combined aeration and filtration.

The pilot plant had not operated on composite samples of the entire plant wastes and efforts were made to obtain continuous samples of all the waste streams during the summer of 1953. The acid content of the wastes rapidly corroded pipes and pumps while slime growths tended to clog them. It was never possible to obtain proper samples and after considerable frustration, the pilot plant was abandoned.

In the spring of 1954, Shawinigan Resins decided to hire Rolf

Eliassen Associates as biological consultants to determine the most economical treatment for their wastes. Studies conducted during the summer of 1954, showed that the wastes contained approximately 1090 ppm 5-day B.O.D., approximately the same organic concentration as in 1951. The total organic content as measured by the chemical oxygen demand test (C.O.D.) averaged 1910 ppm. The wastes contained ample nitrogen but were deficient entirely in phosphorus since the sanitary sewage was diverted to the Indian Orchard Sewage Treatment Plant. The pH of the wastes ranged from 2.5 to 4.2. Treatment of the raw wastes by plain aeration or by lime as used in the earlier studies resulted in only slight B.O.D. reductions.

Laboratory activated sludge units were used to study aerobic treatment of the wastes. A two stage aeration unit with a 30 hour aeration period was able to give better than 90 per cent B.O.D. reduction. The system was extremely stable to overloading, showing decreased efficiency at 100 per cent shock overload but rapidly recovering if the high loading persisted or if it were reduced. The mixed liquor suspended solids were maintained at approximately 1500 ppm in the first unit and approximately 4000 ppm in the second unit. A batch-fed activated sludge unit operated on a 24 hour cycle, 23 hours aeration and one hour's settling, using 1:1 diluted wastes produced an effluent under 40 ppm B.O.D. with 6400 ppm mixed liquor solids. Wasting one-third of the mixed liquor suspended solids daily to simulate high-rate activated sludge dropped the mixed liquor suspended solids to an equilibrium of 450 ppm at the start of the aeration period. The efficiency of B.O.D. removal dropped from 97 per cent to 80 per cent. These experiments showed that activated sludge treatment of the wastes should be able to produce an effluent of 50 ppm B.O.D. for ultimate disposal in the Chicopee River.

Anaerobic digestion studies were carried out on the sludge produced in the aeration studies. The excess activated sludge was readily digested at 37° C in a batch-fed digester with a 10 day retention period. Digestion of the raw process wastes was tried and found quite successful with a 10 day retention period. The gas yield at this loading was almost equal to theoretical. Efforts to shorten the retention period to more practical limits resulted in loss of activity in the digester.

While the laboratory studies were primarily carried out on activated sludge systems, the results indicated that either of the two common aerobic treatment systems, activated sludge or trickling filters would be satisfactory. Cost analyses showed that a capital investment of between $600,000 and $1,000,000 would be necessary for either system. The large quantity of recirculation required on the trickling filters tended to offset the cost of the aeration equipment in the activated sludge system.

In October, 1954, a meeting was held at the Massachusetts State Health Department in Boston to present the work to date and to plot future action. Emphasis was placed on trickling filters rather than activated sludge as the best method for biological treatment since the filters required lower maintenance and absorbed shock loads more readily. It was suggested that trickling filter pilot plant studies be made to confirm the suggested design criteria. It was also suggested that Shawinigan Resins make a reevaluation of all their processes to determine if it were not possible to reduce the total waste discharge. The State Health Department undertook the task of making stream surveys on the Chicopee River to determine the relative share of pollution caused by Shawinigan Resins and to determine the river's capacity for absorbing organic matter.

Examination of the chemical processes by Shawinigan Resins failed to show where waste reductions could be made more economically than by biological methods. Plant expansion during the year actually resulted in stronger wastes and prevented the Engineering Division from setting the Shawinigan Resins trickling filter pilot plant in operation. Instead, Rolf Eliassen Associates was retained to make the filter study. Two filters were studied, one with 2 to 3 inch stone media and one with tile media. The filters were two stage filters with intermediate and final sedimentation. Each stage was three feet deep. Recirculation of final effluent was used to reduce the organic concentration on the first stage. Both filters were designed for a hydraulic loading of 20 M.G.A.D. and an organic loading of 2 pounds B.O.D. per cubic yard filter media. Initial waste samples indicated a relatively large increase in the strength of the wastes. In order to get an accurate sample of wastes, Shawinigan Resins personnel made a 1000 gallon composite of all of their plant wastes over a week's operation. Analyses of this composite showed 2030 ppm 5-day B.O.D., 3160 ppm C.O.D., 140 ppm total nitrogen and pH 2.4. It appeared that the B.O.D. of the wastes had doubled in a year's operation.

Optimum efficiency for the filters resulted at 98 per cent for the tile filter and 96 per cent for the rock filter at a raw waste loading of 1.4 pounds B.O.D. per cu. yd. filter media. A 18:1 recirculation ratio was required. Efforts to raise the loading on the filters were fairly successful. The rock filter gave 94 per cent efficiency at 1.8 pounds B.O.D. per cu. yd. with 9.5:1 recirculation ratio and dropped only to 78 per cent at 3.9 pounds B.O.D. per cu. yd. The tile filter operated at 98 per cent efficiency at 3.9 pounds B.O.D. per cu. yd. with a 10:1 recirculation ratio and dropped only to 90 per cent at 5.0 pounds B.O.D. per cu. yd. Although the efficiency of operation was high at these loadings, septic conditions resulted in the filters and H_2S was noticeable. The demand for oxygen far exceeded the filter's ability to supply it. It appears that nuisance odors will be the primary limiter for filter loadings.

When it became obvious that the trickling filters would be larger than originally estimated, a modified activated sludge system was set up to overcome the objections of high maintenance and shock loadings. The system was designed to operate on the principle "endogenous metabolism", i.e. complete oxidation of the wastes aerobically. Equilibrium was reached at a mixed liquor suspended solids concentration of 17,500 ppm with a 90 hours aeration period and an oxygen uptake rate of only 18 mg. per liter per hour. The effluent B.O.D. was estimated at 20 ppm. Optimum operation occurred during growth of fresh solids rather than during endogenous metabolism. The mixed liquor suspended solids were 22,800 ppm and increasing rapidly with two-thirds of the B.O.D. going into fresh solids. An effluent B.O.D. of 58 ppm was produced in 21 hours aeration and the oxygen uptake rate of the sludge was only 36 mg per liter per hour. Translating the pilot plant data into design criteria and making cost estimates of the high rate trickling filter and the modified activated sludge system resulted in the following figures:

High rate trickling filters	$1,500,000
Modified activated sludge	$ 400,000

In August, 1955, State Health Department personnel began the survey of the Chicopee River. Just as the study was getting under way, floods washed the survey out. A second flood in October prevented a further attempt this year. Even though it was not possible to make the survey as planned, it was possible to make a reasonable estimate as to the river's ability to assimilate organic matter. Examination of U. S. Geological Survey records shows that the flow of the Chicopee River is controlled by a power dam a short distance above the Monsanto Chemicals Plant. During periods of low flow the dam is closed from midnight to six a.m. on weekdays and all day Sunday. In spite of this 30 to 100 cfs manage to pass. At six a.m. the dam is opened and the flow is adjusted to the power demands. The dam at Chicopee Falls below the industrial plants in question tends to absorb the flow variations by backing the river up. It is obvious that the wastes will flow rapidly down to the dam at Chicopee Falls since the river drops 50 feet in about four miles between the two dams. The dam prevents rapid discharge of the river and it is here that the critical oxygen conditions result. While the exact amount of organic matter which can be assimilated by the Chicopee River below Shawinigan Resins has not been determined, the nature of the river prevents adsorption of much organic matter. The critical portion of the river lies in a quiescent zone where there is little chance for reaeration.

The pollution problem on the Chicopee River has not been solved at this time but a start to its solution has been made. The two industries primarily responsible for the pollution problem have recognized the problem and by a cooperative effort with their own consultants and the Massachusetts State Health Department have made an effort to alleviate the situation. It has not been an easy problem and there have been many false solutions. It has taken seven years to find a satisfactory method for treating the wastes from Shawinigan Resins, for an entire new field had to be learned. Old concepts had to be discarded and new ones found. The lessons learned from this example of cooperation between industry and state offer the real solution to stream pollution problems.

Acknowledgment

The assistance of T. Ferris, R. Hasting and H. Weymouth of Shawinigan Resins Corporation and R. Powers of the Massachusetts State Health Department in preparing the material for this paper is gratefully acknowledged.

OF GENERAL INTEREST

Inspection Trip to the Massachusetts Turnpike

Fifty-one mmebers and guests of the Boston Society of Civil Engineers met at the Maintenance Depot of the Massachusetts Department of Public Works at the intersection of Routes 9 and 129 in Wellesley on September 21, 1955. Two bus loads of people left at 9:30 A.M. and in a few minutes were traveling over about four miles of actual turnpike right of way. Though the ride was quite rough, it was worthwhile to view at first hand the preliminary construction of this part of the road. The buses crossed the turnpike in three places, and in the Rochdale-Charlton City area, traveled over three or four more miles of incompleted right of way.

The party stopped at the Publick House, Sturbridge, at 11:45. A Social hour took place in the Cocktail Lounge until 12:45 when a buffet luncheon was served. After the luncheon, Mr. McCloskey of the Turnpike Authority spoke to the group concerning the various aspects of construction of the Turnpike. The return tour followed Route 20 to Route 9 to the point of beginning. On this trip several cuts and interchanges were viewed. At each location, Mr. J. H. Reynolds explained how far each project had progressed and enumerated the problems encountered.

Though rain fell constantly throughout the day, the spirits of the travelers were high due to the widespread interest in the huge Turnpike project.

ROGER C. COLETTE,
Chairman Hospitality Committee

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

OCTOBER 19, 1955.—A Joint Meeting of the Boston Society of Civil Engineers with the Northeastern Section of the American Society of Civil Engineers and the Structural Section, BSCE, was held this date at Massachusetts Institute of Technology, Cambridge, Mass. The Student Chapters of the New England Colleges were especially urged to attend.

A dinner was held in Walker Memorial, Mass. Institute of Technology from 6:30 P. M., to 7:30 P.M. Student delegations were present from Northeastern University, Tufts University, Mass. Institute of Technology, University of Rhode Island, Brown University, University of Massachusetts and Worcester Polytechnic Institute.

The meeting which was held in the new Kresge Auditorium at Mass. Institute of Technology was called to order by President Cobb at 7:35 P.M. President Cobb extended a cordial welcome to the students and expressed appreciation of the cooperation of the officers of the student organizations and of the faculty members in making this event so successful.

President Cobb called upon the Secretary to announce the names of applicants for membership in the BSCE.

President Cobb introduced Emory Ireland, President of the Northeastern Section of the American Society of

Civil Engineers and asked him to conduct the necessary business for ASCE at this time.

President Cobb called upon Mr. C. J. Kray, Chairman of the Structural Section, BSCE, to conduct any necessary business for that Section at this time.

President Cobb then introduced the speaker of the evening, Dr. John B. Wilbur, Head of Dept. of Civil and Sanitary Engineering at Mass. Institute of Technology, who gave a most interesting talk on "Esthetic Aspects of Civil Engineering".

Two hundred seventeen members and guests attended the dinner.

The meeting adjourned at 8:45 P.M.

ROBERT W. MOIR, *Secretary*

NOVEMBER 16, 1955.—A Joint Meeting of the Boston Society of Civil Engineers with the Hydraulics Section, BSCE, was held this evening at the United Community Service Building, 13 Somerset Street, Boston, Mass., and was called to order by President Edwin B. Cobb, at 7:05 P.M.

President Cobb announced that the minutes of the previous meeting would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes would be waived unless there was objection.

The President announced the death of the following members:—

Calvin C. Sears, who was elected a member September 18, 1912, and who died May 3, 1955.

Porter O. Robinson, who was elected a member May 19, 1926, and who died April 28, 1955.

Otto Germanis, who was elected a member May 14, 1951, and who died July 10, 1955.

Albert E. Cummings, who was elected a member March 21, 1945, and who died July 20, 1955.

Albert E. Kleinert, who was elected a member December 23, 1925, and who died October 23, 1955.

The Secretary announced the names of applicants for Membership in the BSCE, and that the following had been elected to membership on November 16, 1955.

Grade of Member—Rance L. Banner, Lawrence E. Maibach, Edward J. Michalski, Robert W. Mirick, Audun Ofjord, Elwood C. Richardson, William E. Sullivan.

Grade of Student—Bruce N. MacIver.

President Cobb announced that this was a Joint Meeting with the Hydraulics Section and called upon Ralph S. Archibald, Chairman of that Section to conduct any necessary business for that section at this time.

President Cobb then introduced the speaker of the evening, David R. Campbell, Engineer, New England Power Service Company, who gave a most interesting talk on "The report on Progress of Construction of Littleton Development". The talk was illustrated with slides and movies.

A short discussion period followed after which the President announced that a collation would be served.

Forty-five members and guests attended the meeting.

The meeting adjourned at 8.30 P.M.

ROBERT W. MOIR, *Secretary*

DECEMBER 7, 1955.—A Joint Meeting of the Boston Society of Civil Engineers with the Sanitary Section, BSCE, was held this evening at the United Community Service Building, 14 Somerset Street, Boston, Mass., and was called to order by President Edwin B. Cobb, at 7:00 P.M.

President Cobb announced that the minutes of the previous meeting November 16, 1955 would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes would be waived unless there was objection.

The President announced the death of the following member:—

George C. Wallace, who was elected a member November 15, 1954, and who died November 27, 1955.

The Secretary announced the names of applicants for Membership in the BSCE.

President Cobb announced this was a Joint Meeting with the Sanitary Section and called upon Ariel A. Thomas, Chairman of that Section to conduct any necessary business for that section at this time.

President Cobb, announced that the speaker for the January 25, 1956 meeting would be Prof. John T. Howard, Dept. of City and Regional Planning, of Mass. Institute of Technology, whose subject will be "Transportation Planning", and it will be a Joint Meeting with the Surveying and Mapping Section and also the Transportation Section.

President Cobb then introduced the speaker of the evening, Ross E. McKinney, Asst. Prof. Sanitary Engineering, Mass. Institute of Technology who gave a most interesting illustrated talk on "A Cooperative Approach to Stream Pollution Control in Massachusetts".

A rising vote of thanks was given the speaker..

Thirty-eight members and guests attended the dinner preceding the meeting and sixty-three attended the meeting.

The meeting adjourned at 8:20 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

OCTOBER 5, 1955.—The meeting was called to order by Acting Chairman Darrell A. Root at 7:00 p.m. at the Society Rooms after an informal dinner at Pattens Restaurant. Twenty-five members and guests attended the dinner and fifty members and guests attended the meeting.

Mr. Root announced that Ariel Thomas, Chairman of the Sanitary Section, has been transferred to New Jersey and requested Mr. Root to take over the direction of the Sanitary Section for the remainder of the year. Chairman Root then proceeded to introduce the speakers, Charles G. Hammann, Director of Public Works of Woonsocket, Rhode Island, and John S. Bethel, Jr., Partner, Metcalf and Eddy, Engineers, who proceeded to deliver talks on the Comprehensive Sanitation Plan for Woonsocket.

Mr. Hammann gave a brief outline of the development of the City of Woonsocket from its founding up to the present and described, in general, the sanitation problems of the City. Mr. Bethel followed with a paper describing the engineering studies made of the problems and the recommendations resulting from the study, along with a program of improvements developed to meet the sanitation requirements. At the conclusion of the paper, there was an interesting discussion between the speakers and the members and guests.

The meeting was adjourned shortly after 8:30 p.m.

JOHN F. FLAHERTY, *Clerk*

STRUCTURAL SECTION

OCTOBER 19, 1955.—A joint meeting of the Boston Society of Civil Engineers, the Northeastern Section of the American Society of Civil Engineers, and the Structural Section of BSCE was held on this date at the Massachusetts Institute of Technology in Cambridge, Mass. Civil Engineering students of the New England Colleges were especially invited to attend this meeting.

Dinner was served at 6:30 P.M., in Walker Memorial, M.I.T. Delegations of students were present from Brown University, Massachusetts Institute of Technology, Northeastern University, Rhode Island University, Tufts University, University of Massachusetts, and Worcester Polytechnic Institute.

The meeting was held in M.I.T.'s new Kresge Auditorium and was called to order by President Cobb at 7:40 P.M. After the necessary business was conducted, President Cobb introduced the speaker of the evening, Dr. John B.

Wilbur, Head of Department of Civil and Sanitary Engineering, M.I.T.

Dr. Wilbur's subject was "The Aesthetic Aspects of Civil Engineering". This very stimulating talk which was illustrated by slides discussed the various factors which contribute to the beauty of a structure. He urged the young engineers present to take the lead in the development of more pleasing structures.

The talk was well received by the two hundred seventeen members and guests present. The meeting was adjourned at 8:45 P.M.

JOHN M. BIGGS, *Clerk*

NOVEMBER 9, 1955.—A regular meeting of the Structural Section was held on this date in Myers Hall, Tremont Temple. The attendance was approximately two hundred members and guests. Chairman Kray called the meeting to order at 7:15 P.M., and introduced the speaker, Dr. D. B. Steinman, Consulting Engineer, New York City.

Dr. Steinman's subject was "The $100,000,000 Mackinac Bridge—Conquering the Impossible". The overall design and the construction to date were described and illustrated by slides. This bridge which will have the longest total length of any suspension bridge in the world involved enormous quantities of material and many unusual methods of construction. The speaker's description of the aerodynamic design of the structure was of particular interest.

After question and answer period the meeting was adjourned at 8:45 P.M.

JOHN M. BIGGS, *Clerk*

HYDRAULIC SECTION

MAY 4, 1955.—A meeting of the Hydraulic Section was held at the Hydrodynamics Laboratory of Massachusetts Institute of Technology in Cambridge, Mass., on this date. The meeting was convened at 7:00 P.M. in the lecture room by Chairman Archibald, and then turned over to Dr. Arthur T. Ippen, who described the various demonstrations which were set up in the laboratory. Mr. Thomas of the Sanitary Section announced that all were invited to a meeting of that section in Lawrence and Andover, where among other interesting subjects, developments for dissipation of radio-active waste were to be described and demonstrated.

The assembly of 56 members and guests was then divided into a number of small groups who were guided to the various laboratory experiments by members of the Institute staff. Other staff members, who were stationed at the experiments, described and demonstrated the various tests.

The principal demonstration which was observed consisted of a wave tank, a model dam spillway, turbulence studies in a high velocity flume, towing of ship models in a towing tank, pressure distribution around a tainter gate, a model surge tank, operation of the cavitation tunnel and the flow of density currents in a small flume.

The tour was well organized and operated on a close time schedule so that all were able to return to the lecture hall where questions on details of the demonstrations which had been observed were answered by Dr. Ippen and his staff. The meeting was adjourned at 9:10 P.M. after a rising vote of thanks by those present for a very profitable and pleasant evening.

CLYDE W. HUBBARD, *Clerk*

SURVEYING AND MAPPING SECTION

APRIL 6, 1955.—The twenty-seventh meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:00 P.M. on this date.

Prior to the meeting the executive committee of the section met in the Society Rooms for the purpose of planning a program for the coming year.

The meeting was called to order by

Chairman Alexander J. Bone. The minutes of the meeting held on January 26, 1955 were read by the Clerk and approved. There was no new business to be taken up with the members.

The general topic for the evening, "Educational Methods, Formal and Practical, Producing the Best Results in Training Today's Surveyor". The topic was well discussed, analyzed and recommendations made by the guest speakers—Prof. John B. Wilbur, Prof. Charles O. Baird, Jr., Charles M. Anderson and Nelson W. Gay.

Professor Wilbur and Professor Baird of Mass. Institute of Technology and Northeastern University, respectively, discussed the place that surveying plays in education of the Civil Engineer and the teaching methods employed. Because of the ever broadening scope of the Civil Engineers field it was felt that surveying will occupy a smaller and smaller percentage of the Civil Engineering teaching hours. The suggestion was made to set up a new course with a new curriculum specifically for surveying.

Mr. Anderson, Chief Engineer, Massachusetts Land Court, and Mr. Gay, Chief Engineer, New England Survey Service, discussed problems of the employer in obtaining qualified party chiefs and the methods used in the practical training of competent men on the job.

The general discussion that followed the speakers, brought out strongly the fact that while the adventure and the interest attached to the surveyor's life attracted many good men and many were competent civil graduates, the general lack of attractive pay soon make many of them turn to other fields. It was felt that after a sustained acute shortage of competent surveyors, with which not even on the job training could cope, a marked increase in salaries would result in an easing of the search for qualified men and perhaps even a return to that breed of men whose studies of land law, mathematics, and surveying techniques, raised the surveying branch of Civil Engineering to a very enviable level.

Thirty-eight members and guests were present at the meeting.

The meeting adjourned at 8:30 P.M.

Ernest A. Herzog, *Clerk*

October 26, 1955.—The twenty-eighth meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:00 P.M.

The meeting was called to order by the Chairman, A. J. Bone, and the minutes of the April 6, 1955 meeting were read and approved.

Upon a motion made and recorded, the section voted to authorize the Chairman to appoint a nominating committee to submit a slate of officers for the coming year. The Chairman accordingly appointed Messrs. George W. Hankinson, C. Frederick Joy and Wilbur C. Nylander to serve on this committee.

The Chairman then introduced the speaker of the evening, Mr. Herman J. Shea, Chief Engineer, James W. Sewall Company, who spoke on the subject, "Recent Developments in Surveying Instrumentation".

Mr. Shea described the principles and application of several types of foreign and domestic surveying instruments which are being introduced and used in this country. Typical instruments contributed by section members were demonstrated.

Following the talk several members gave their experiences with some of the instruments on display.

There were 48 members and guests present.

E. A. Herzog, *Clerk*

ADDITIONS

Members

John C. Adams, Jr., 12 Hereford Road, Marblehead, Mass.

Paul J. Berger, 13 Webb Road, Sharon, Mass.

Rance L. Banner, Box 142, Keene, New Hampshire.

John G. Chalas, 3 Maplewood Street, Watertown 72, Mass.

James R. Cass, 412 Huntington Avenue, Hyde Park 36, Mass.

William R. Duffy, 34 Dartmouth Avenue, Dedham, Mass.

Peter S. Eagleson, Rm. 48-102 Mass. Institute of Technology, Cambridge 39, Mass.

Richard F. Ebens, 85 High Street, Winchester, Mass.

Arthur Hebert, 10240 Corning Oak Park 37, Michigan.

Kenneth W. Henderson, 134 Pearsall Drive, Apt. 2F, Mt. Vernon, New York.

Franklin E. Kasef, 125 Homes Avenue, Dorchester 22, Mass.

Tadeusz S. Klodnicki, 45 Chambers Street, Boston 14, Mass.

Charles F. Kennedy, 29 Moreland Street, Somerville 45, Mass.

Robert S. Loomis, 31 Loomis Avenue, Windsor, Conn.

Jack Manusowicz, 59 Brookledge Street, Roxbury 21, Mass.

Edward J. Michalski, 9 Haven Lane, Cochituate, Mass.

Robert W. Mirick, 18 Eames Street, North Reading, Mass.

Ronald E. Nece, Hydrodynamics Laboratory, Mass. Inst. Technology, Cambridge 39, Mass.

James E. Roberts, 314 Westgate W., Cambridge 39, Mass.

Paul S. Ross, 19 Central Avenue, Hull, Mass.

Elwood C. Richardson, 33 High Street, Nahant, Mass.

Roger P. Reidy, 45 Chester Street, Newton Highlands, Mass.

John P. Sullivan, 46 Woodbrier Road, W. Roxbury 32, Mass.

Reuben Samuels, 85 Wyoming Road, Paramus, New Jersey.

James Simmons, 221 East Street, Sharon, Mass.

Francis W. Taylor, 109 Watervale Road, Medford 55, Mass.

John F. Vance, Jr., 250 Park Street, Attleboro, Mass.

Juniors

Joseph J. Allegro, 126 Dakota Street, Dorchester 24, Mass.

A. Gregory Chase, 37 Prospect Street, Newburyport, Mass.

Earl H. Christopher, 93 Nahant Street, Wakefield, Mass.

Paul A. Eno, 780 Andover Street, Lowell, Mass.

Robert G. Esterberg, 188 Country Way, Greenbush, Mass.

Joseph A. Grimaldi, 48 Hichborn Street, Brighton 35, Mass.

Norman W. Crossman, 154 Union Street, Framingham, Mass.

Richard P. Ierardi, 257 Washington Street, Dorchester 21, Mass.

Uldis Kirsis, 47 Custer Street, Jamaica Plain 30, Mass.

Irving M. Kusnitz, 169-08 69th Avenue, Fresh Meadows, New York.

Robert M. Lee, 23 Kenwood Street, Brookline 46, Mass.

Robert A. McCracken, 34 Bennington Street, Lawrence, Mass.

Domenic M. Palmer, 24 Curtis Avenue, Quincy 69, Mass.

Harold J. Powderly, 36 Rosa Street, Hyde Park 36, Mass.

Joseph P. Sasso, 33 Bartlett Street, Malden 48, Mass.

James P. O'Sullivan, 138 Train Street, Dorchester 22, Mass.

Carmen C. Testa, 130 Meridian Street, Boston 28, Mass.

Norman A. Whalen, 29 Myrtle Street, Brockton, Mass.

David B. Weiner, 52 Woodlawn Avenue, Chelsea 50, Mass.

DEATHS

Calvin C. Sears, May 3, 1955
Porter O. Robinson, April 28, 1955
Otto Germanis, July 10, 1955
Albert E. Cummings, July 20, 1955
Albert E. Kleinert, Oct. 23, 1955
George C. Wallace, Nov. 27, 1955

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