JOURNAL *of the*

BOSTON SOCIETY OF CIVIL ENGINEERS

WILDER

1848 BOSTON SOCIETY OF CIVIL ENGINEERS

101 YEARS

1848-1949

JANUARY - 1949

VOLUME 36 NUMBER 1

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1948 - 1949

PRESIDENT

FREDERIC N. WEAVER

VICE-PRESIDENTS

EVERETT N. HUTCHINS
(Term expires, March 1949)

HARRISON P. EDDY, JR.
(Term expires, March 1950)

SECRETARY

ROBERT W. MOIR

TREASURER

HERMAN G. DRESSER

DIRECTORS

GEORGE W. COFFIN
HERMAN G. PROTZE
(Term expires, March 1949)

CHESTER J. GINDER
FREDERICK S. GIBBS
(Term expires, March 1950)

PAST PRESIDENTS

CARROLL A. FARWELL
GEORGE A. SAMPSON
HARVEY B. KINNISON

SANITARY SECTION

ALLEN J. BURDOIN, Chairman
FRANK L. HEANEY, Clerk

STRUCTURAL SECTION

OLIVER G. JULIAN, Chairman
ARTHUR E. HARDING, Clerk

TRANSPORTATION SECTION

R. NEWTON MAYALL, Chairman
WILLIAM L. HYLAND, Clerk

HYDRAULICS SECTION

JOHN G. W. THOMAS, Chairman
GARDNER K. WOOD, Clerk

SURVEYING AND MAPPING SECTION

CHARLES M. ANDERSON, Chairman
HUGH P. DUFFILL, Clerk

NORTHEASTERN UNIVERSITY SECTION

WILLIAM J. DOWNEY, Chairman
ARTHUR L. QUAGLIERI, Clerk

Librarian—THOMAS C. COLEMAN

Editor—CHARLES E. KNOX

715 Tremont Temple, Boston

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 36 JANUARY, 1949 Number 1

PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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

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

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

Subscription Price $4.00 a Year (4 Copies)
$1.00 a Copy

PATRONIZE OUR ADVERTISERS

Like most everything else in this post-war inflationary period, the cost of publishing this JOURNAL has increased several fold. The cost of publishing the JOURNAL is now the largest single item of expense in the budget of the Society. To reduce the burden on the Society, a concerted effort is being made to increase the revenue from advertising.

Every member interested in the welfare of the Society should actively engage in the soliciting of advertising for the JOURNAL. Your Committee on Advertising is doing its best to reach all potential advertisers but since the committee members must do this work on their own time and at their own expense, it is doubtful if the field can ever be adequately covered by the committee alone. If every member of the Society would try to obtain even one ¼ page advertisement, the JOURNAL would quickly be self-supporting.

The advertising in the JOURNAL serves as a directory for Civil Engineers, Contractors, and Material Suppliers in this area. The more complete the coverage of these organizations by advertisements, the more valuable is this directory.

ADVERTISING RATES

	FOUR ISSUES
1/12 page (limited to professional cards only)	$ 25.00
¼ page (4½ × 1-11/16 net)	42.00
½ page (4½ × 3⅝ net)	70.00
Full page (4½ × 7½ net)	105.00
Inside back cover	122.50
Inside front cover	140.00
Outside back cover	175.00

A sample copy of the JOURNAL *will be provided upon request.*

For further information contact the Society headquarters.
715 Tremont Temple, Boston, Mass.

Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 16, 1918.

THE HEFFERNAN PRESS
WORCESTER, MASS.

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 36 JANUARY, 1949 Number 1

THE PROPOSED REDEVELOPMENT OF THE WATER POWER OF THE CONNECTICUT RIVER AT WILDER, VERMONT

By E. G. Lee, Member*

(Presented at a meeting of the Boston Society of Civil Engineers held on September 22, 1948.)

During this year of 1948 we, of this oldest engineering society organized in America, have been looking back 100 years to the time of that first meeting on July 3rd, 1848; in reviewing the events which preceded that we see that only about 100 years earlier the first white settlements in the valley of the Connecticut river north of the Massachusetts line were being established. In 1748 the most northerly of these settlements were at Charlestown, New Hampshire and Springfield, Vermont; these are located about 33 miles south of Wilder, Vermont, the location of the site of White River Falls and the water power development which we are considering this evening.

This 33 miles seems a very short distance in these days of automobiles and good roads and airplanes but in those days it often seemed a long weary distance.

Any of you who have read Kenneth Roberts novel "Northwest Passage" will remember his account of the expedition of the Rogers Rangers against the Indians at St. Francis, Quebec. On their return from that trip, in the fall of 1759, they straggled down past Lake Memphremagog and over to the Connecticut River, where they built a raft on which they hoped to sail down to the settlement at Charlestown, New Hampshire. They got on fairly well until they reached the site of the present Wilder development, then known as White

*New England Power Service Company, Boston, Mass.

River Falls, where they had the grievous experience of having their raft broken up in the turbulent rapids and of losing most of their few remaining possessions as they struggled to the shore. Then I reckon they found those remaining 33 miles to Old Fort #4, which was at Charlestown, a very long weary distance.

The conflicts between the English settlers and the French and the Indians delayed settlements along the river north of Charlestown and Springfield until after the end of the French and Indian War. Within less than 10 years thereafter, there were more than fifteen new settlements in the region bordering on the proposed new Wilder pond. One of the earliest of these was in 1762 at Newbury, Vermont, where the famous Newbury meadows are located, at the northerly end of the proposed pond, while Hartford, Vermont, in which Wilder is located, and which is at the southerly end of the pond, was established in 1764.

In those early days the river afforded the best means of travel and transportation of loads up and down the valley and, of course, the falls and principal rapids were obstacles to be overcome by portages or by constructing canals and locks for bypassing.

In 1795, the Vermont legislature granted a charter to "The Proprietors of White River Falls" with the right to build a canal and bridge. These were not built under this charter but the New Hampshire legislature passed an "Act of 1807" entitled "An Act Granting to Miles Olcott the Privilege of Locking White River Falls". By 1810 two separate canals, with dams and locks, had been built and they probably were used considerably until they were put out of business by the arrival of the steam railroad, about 1848, the end of this 100 years, when they fell into disuse, and then were completely crippled a few years later by a freshet carrying away the dam.

The remains of the upper locks are practically obliterated but parts of the lower locks are still clearly distinguishable on the New Hampshire side of the river just south of the lower falls, although they probably are seldom seen because they are so well hidden by trees and bushes.

It is interesting to note that practically all early settlers recognized that the development of the water power available in the rivers and smaller streams was of considerable value to the communities. It is noted in Tucker's History of Hartford, Vermont, that at White River Falls mill privileges were utilized on both sides of the river

as early as 1785 when "grist, saw and fulling mills were erected with rights to draw water from the bulkhead", which was probably a wing dam near the upper end of the falls.

Although the records are rather incomplete, probably some use was being made of the power at White River Falls or Olcott Falls practically all the time after that first development except for short periods when the dams or other structures had not been repaired after being washed out or badly crippled by floods.

In 1865 a small mill had been built at the upper falls, on the Vermont side, for the manufacture of paper from straw, but this mill was destroyed by a flood in 1872 and was not rebuilt.

The water rights and property at the falls were later acquired by Wilder Brothers, paper manufacturers of Boston, and in 1882 a new timber dam was built at the upper falls, with a large pulp and paper mill, which was operated by the Wilders until it was absorbed by the International Paper Company in 1898. After that it was expanded and continued in use for paper mill purposes until it ceased operation in 1927.

In 1926 a new reinforced concrete dam was built just downstream of the old timber dam, a considerable portion of which is still intact, completely flooded by the new dam. This new dam is a hollow concrete spillway section, about 635 feet long with crest at about elevation 366.4 above mean sea level and with removable flashboards about 3 feet high. The concrete spillway has a flat crest, 11 feet 6 inches wide and was designed to be raised about 5 feet in the future. There are no flood gates in the present dam and the March 1936 flood of about 91,000 c.f.s. at Wilder raised the water level to a little over elevation 380, almost 14 feet deep over the crest and more than 10 feet above normal full pond level. The contemplated raising of the crest was never carried through.

In the present development, as shown on Figure No. 1 the water for power is taken through an intake structure at the south-westerly end of the spillway discharging into a canal from about 80 to 130 feet wide by about 12 to 18 feet deep and about 600 feet long. At the downstream end of the canal there is another intake structure, fitted with trash racks and control gates and discharging into steel penstocks leading to the individual power units.

In 1910 a small electric power plant with two 750 KW twin runner, horizontal shaft units was built to furnish 3 phase, 60 cycle cur-

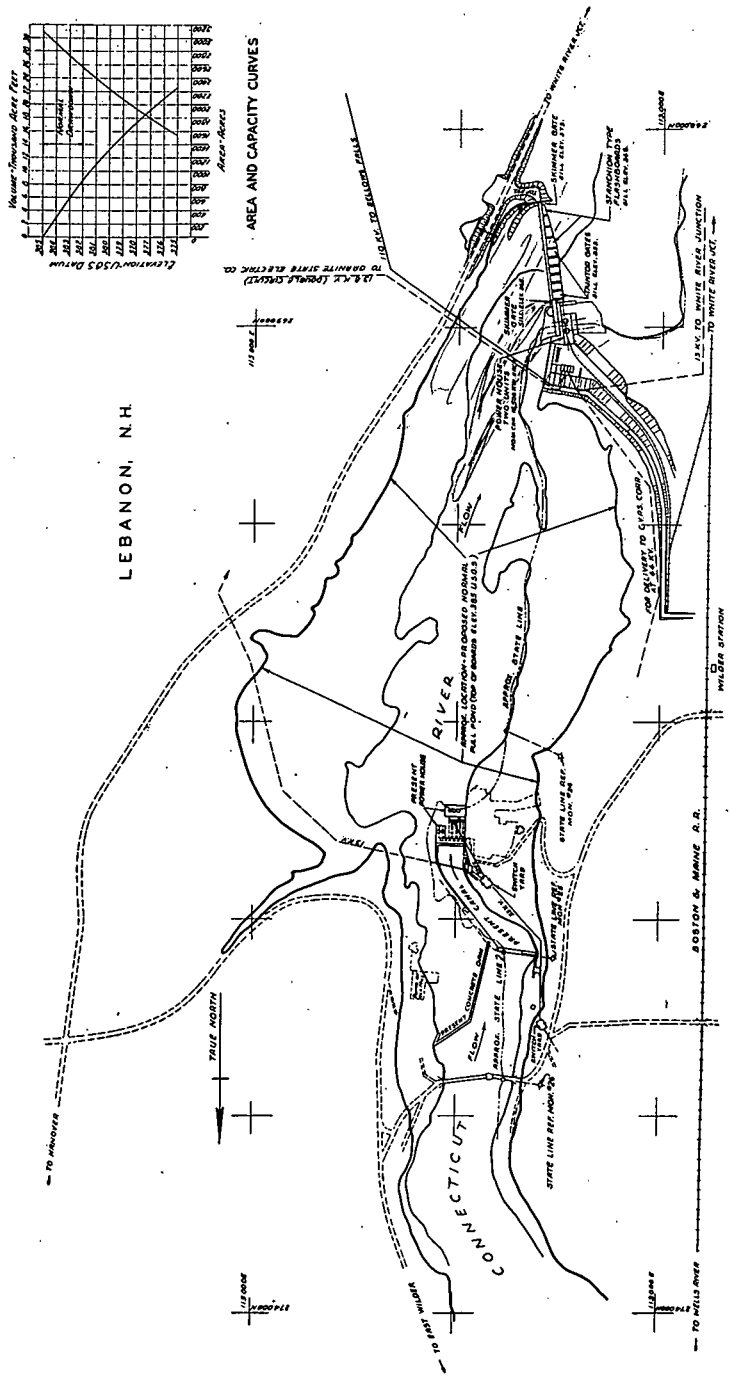


FIG. 1.—GENERAL PLAN, PRESENT AND PROPOSED WILDER PLANTS.

rent for the paper mill and the mill village. When the paper mill was shut down in 1927, this small electric power plant was continued in operation together with some of the twin runner, horizontal shaft units of the old pulp grinder room, where second hand horizontal shaft generators had been installed to replace some of the old grinders.

I understand the first commercial power development in that vicinity was the building in 1895 on the Mascoma River of what is now the Number 2 plant of the Granite State Electric Company. The lines of that Company, then the Grafton County Power and Light Company, were tied into the Wilder plant about 1923 and in 1929 a tie was made from the Wilder plant to the Taftsville substation giving a connection to what is now the Central Vermont Public Service Corporation and to the Bradford Electric Company. This provided a temporary tie to the northern part of the New England Electric System, giving a power backup during the construction period to the construction lines at the 15 Mile Falls, or Comerford development on the Connecticut river near Littleton, New Hampshire.

The Central Vermont Public Service system had been tied to the New England Electric System back in 1913 at Bellows Falls, Vermont, being connected to the lines of what was then called the Fall Mountain Electric Company. Later on this connection extended through to Wilder by way of the Taftsville connection and what is now the Central Vermont Public Service Corporation system.

There are now available, when they are in operating condition, 5 generating units in the present Wilder plant with a total capacity of about 5,000 KW. These units use a maximum water flow of about 2400 c.f.s. under a net head of about 35 feet.

This present plant is now supplying power for use locally by the Granite State Electric Company of Lebanon, New Hampshire and by the Green Mountain Power Corporation in its White River Junction area in Vermont; the remaining output, when any is available, is absorbed by the New England Power Company and an arrangement was made whereby a comparatively small amount of emergency power could be transmitted from the Bellows Falls Station to Wilder by means of the lines of the Central Vermont Public Service Corporation during periods of power shortage at Wilder.

This Wilder plant was purchased by the Power Company in 1942 and was licensed by the Federal Power Commission under a license order issued in October 1943. The Commission noted its opinion that

the existing power plant fails to fully utilize the power resources of the Connecticut River in the Wilder area and in the license which they issued for the period from January 1, 1938 to June 30, 1970, they included a stipulation that "at such time as the Commission, after notice and hearing, shall direct, the licensee shall reconstruct said project in accordance with such plans as may be found by the Commission to be best adapted to a comprehensive plan for improving and developing the Connecticut River for the use and benefit of interstate commerce, for the improvement and utilization of water power development, and for other beneficial public uses".

The Company filed an application in 1944 for an amendment to the existing license to provide for a redevelopment of the water power at Wilder to meet the requirements of the Federal Power Commission for a comprehensive power development of that section of the Connecticut River. Extensive studies were made of redevelopment with headwater levels at elevation 396, 390 and 385. The 396 level was selected as the highest we could reasonably consider with the purpose of obtaining the maximum head and pondage available. We found that the costs of flowage damage and of railroad and highway changes would be excessive, so great, in fact, as to make the redevelopment on that basis uneconomical.

The study with headwater at elevation 390 was sufficiently attractive so that 390 was selected as the elevation to use in the application; however, before the application was submitted, a further reduction was made to headwater at elevation 385, to decrease the area of tillable lands to be flooded to an extent which we believed would remove any reasonable argument that we were taking too much land out of agricultural use. The plans as filed called for a normal headwater level at elevation 385 above mean sea level or between 15 and 16 feet above the top of the flashboards as maintained on the present Wilder dam. A public hearing on this application for an amendment to the license was held at Hanover, New Hampshire, in October 1944 before the Federal Power Commission and the New Hampshire Water Control Commission in joint session.

After further public hearings before the New Hampshire Water Control Commission, the Vermont Public Service Commission and another before the Federal Power Commission in September 1946, the latter Commission on November 1, 1946, directed the Power Company to proceed with the redevelopmpent on the 385 headwater

level basis. The decision to actually proceed with construction was again delayed through the passing by the United States Senate of a resolution which proposes to set aside the decision and order of the Federal Power Commission as issued on November 1, 1946. This phase of the matter is still not settled.

The proposed redevelopment includes a new transmission tie to the New England Electric System by a new 110,000 volt line direct from Wilder to the present 110,000 volt Bellows Falls - Pratt Junction line, at Bellows Falls. There will also be two 13,000 volt circuits to the Granite State Electric Company in Lebanon, New Hampshire, one 13,000 volt circuit to the Green Mountain Power Corporation at Hartford, Vermont and a 44,000 volt tie to the Central Vermont Public Service System so that the entire region can be served by the proposed new plant, and this will make possible the added advantage of steam relay from the New England Electric System being available there at times when local river flows are too low to supply the local power needs.

The proposed redevelopment at Wilder contemplates the construction of a completely new concrete and earth dam and concrete and brick power house at the so-called "Narrows Site", Figure 2, at the lower end of the Olcott or White River Falls and more than one-half mile downstream from the present Wilder dam, which is near the head of the falls.

At this Narrows Site as shown in Figure 3 the river makes a sharp bend to the west crossing a ledge reef and then turns sharply again to the south. There is ledge foundation for all of the concrete structures of the dam and power house. This ledge is a slightly calcareous schist of fairly massive appearance in some places but with well developed laminations in others. One core boring which was drilled diagonally under the river bed for 61 feet about normal to the strike and the dip, showed a stratum 19 feet thick of hard hornblende schist, next a stratum 10 feet thick of soft hornblende schist with layers of pyrite, which Prof. Goldthwaite, our Geologic Consultant at that time said was sound and showed no evidence of the penetration of surface waters, next a stratum 21 feet thick of hard black mica schist, and then a stratum 3 feet thick of soft black mica schist with veins of calcite, then a stratum 8 feet thick of hard white quartz and veins of calcite. The ledge appeared sound throughout the 61 feet of the drill hole. The ledge on the shores is badly

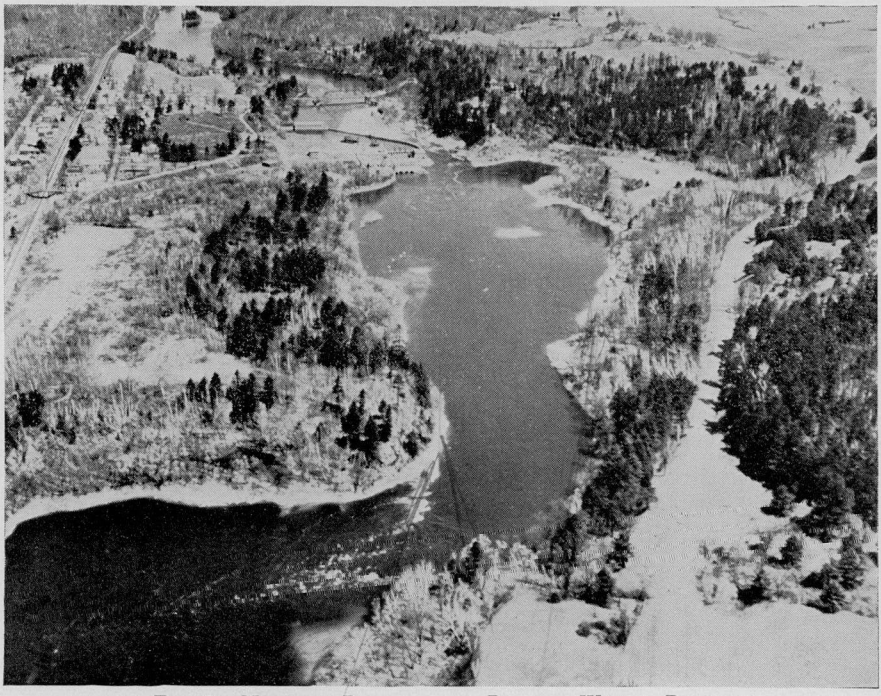


FIG. 2.—NARROWS DAMSITE AND PRESENT WILDER PLANT.
(Fairchild Aerial Surveys, Inc.)

fractured and may need some grouting to make it reasonably tight, also we may find when we proceed with excavation in the ledge that some of the fractures extend below the present river surface and may need grouting.

The borings which we had made in the heavy glacial overburden of the bank at the northwesterly end of the proposed dam showed that some 400 to 500 feet westerly of the ledge reef on which we plan to place the concrete structures of the proposed dam, the ledge surface is some 45 feet lower than the ledge reef across the river, indicating that there are marked ridges and valleys in the ledge surface or that there may possibly be a buried gorge lower than the present river bottom at the reef and some distance west of the present river location upstream from the reef.

Borings number 8 and 9, which were located on the New Hampshire shore east of the line of the aforementioned ledge reef indicated that the ledge surface was dropping in that direction also, and in

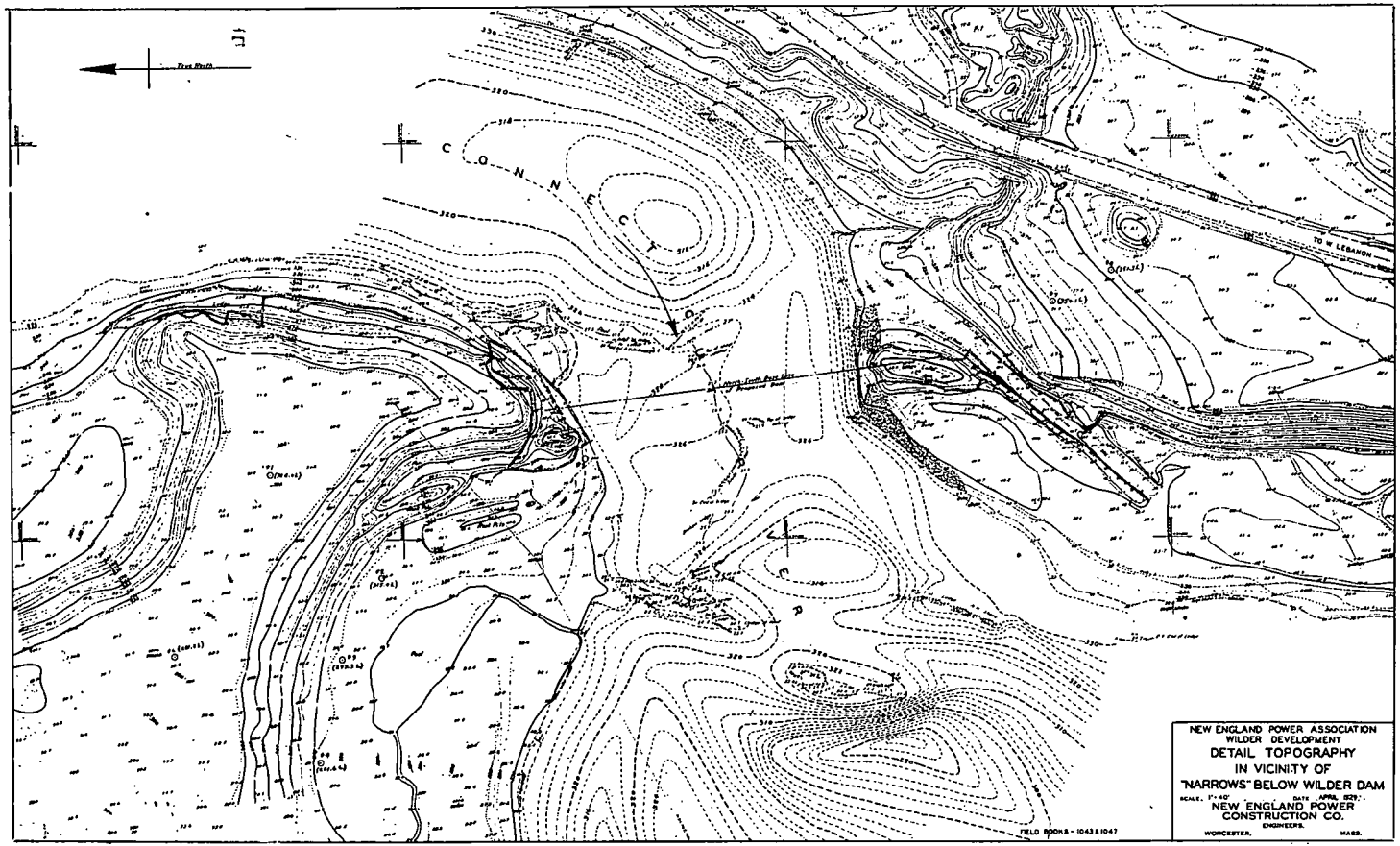


FIG. 3.—TOPOGRAPHIC MAP OF WILDER NARROWS DAMSITE.

the course of the investigation for an alternate dam location which would have less obliquity of approach of the flowing stream to the spillway than in the accepted location, we had a hole started on the New Hampshire side about 150 feet easterly of the line of the ledge reef in the river, when this hole had been sunk to nearly 20 feet below the level of the ledge surface in the river without encountering any indication of approaching ledge we stopped the boring and called off further consideration of that site and of others near it. The information obtained indicated that the ledge surface on the easterly side of the river dropped too much to have that location used as a damsite.

The present natural bank at the northerly end of the proposed damsite has a flat top at about elevation 390 and an earth escarpment some 50 odd feet high on the southerly side, with some present slopes steeper than one vertical on one and one-quarter horizontal. The escarpment on the northerly side of the bank is only about 30 feet high and is not so steep as that on the south. I will have more to say about this bank later on.

The structures of the redevelopment are all comparatively simple and of usual accepted design, the only radical difference from the general design of most of our developments is the omission of pin type flashboards and the use of large flood gate capacity in the spillway. This change is made to avoid the necessity of allowing the water level at the dam during small or approaching large floods from rising above the level of the top of the flashboards or gates or above the level of what we call the normal high pond level.

The proposed new normal high pond level will be at elevation 385.0 feet above mean sea level, and will be between 15 and 16 feet higher than the top of flashboards on the present dam.

At the damsite on the New Hampshire side, State Highway No. 10 will be raised from 0 to 9 feet above its present level and for a length of more than 900 feet. It will also be necessary to replace the present culvert under the highway in that location by a new and considerably larger culvert.

The south embankment of the new dam will start at the side of the raised highway and will extend about 180 feet to the concrete south abutment wall which forms the southerly end of the spillway. This earth embankment will rest on a carefully cleared and grubbed site and will be constructed of properly placed selected materials, with

a minimum top width of 30 feet at elevation 393.0 and with an upstream slope of one on three and a downstream slope of one on two. On the upstream face of the embankment there will be a berme of varying width at elevation 386.0, one foot above normal full pond level; this berme will provide a working space which will facilitate the removal of driki and debris that may tend to collect in the south end of the pond, and the entire face of the embankment below the berme will be protected with rip rap. The maximum height of this embankment will be about 13 feet and the top will be gravelled to provide a roadway to the south abutment wall.

The water passing structures of the spillway will be a 10 foot wide skimmer gate, four 50 foot bays of stanchion flashboards, six 36 foot wide tainter gates and a 15 foot wide skimmer gate, which with the piers will occupy a total length of 521 feet from the face of the south abutment to the face of the pier between the large skimmer gate and the power house. On the plan, Figure 4, that pier is shown as a fishway pier and in the plans we submitted to the Federal Power Commission with our application for a revision of the license, we included a layout of a fishway or fish ladder using a 16 foot wide pier. We have been informed that the design we have submitted will not be acceptable to the Fish and Wildlife Service of the U. S. Dept. of Interior, so we have prepared a new layout which we hope will be accepted but we expect to build only the underwater foundations and the corresponding parts at the upstream part of the dam in the initial construction. The completed fishway probably will not be constructed until successful fishways have been installed at the developments down river from Wilder.

In granting our license the Federal Power Commission put in a stipulation that provision be made in the initial construction for the possible future installation of a third unit so we are leaving space for such a possible future unit between the north end of the spillway and the power house. The plan shows a semi-circular bulkhead or permanent cofferdam, over the upstream end of this space, but our latest studies indicate that we will probably build a standard intake structure, without the gates, across this space and place concrete plugs in the gateways, said plugs to be removed if and when the third unit structure is built.

The next structure is the two unit power house and the control room, which together with the possible future unit space will occupy a distance in the dam structure of about 200 feet.

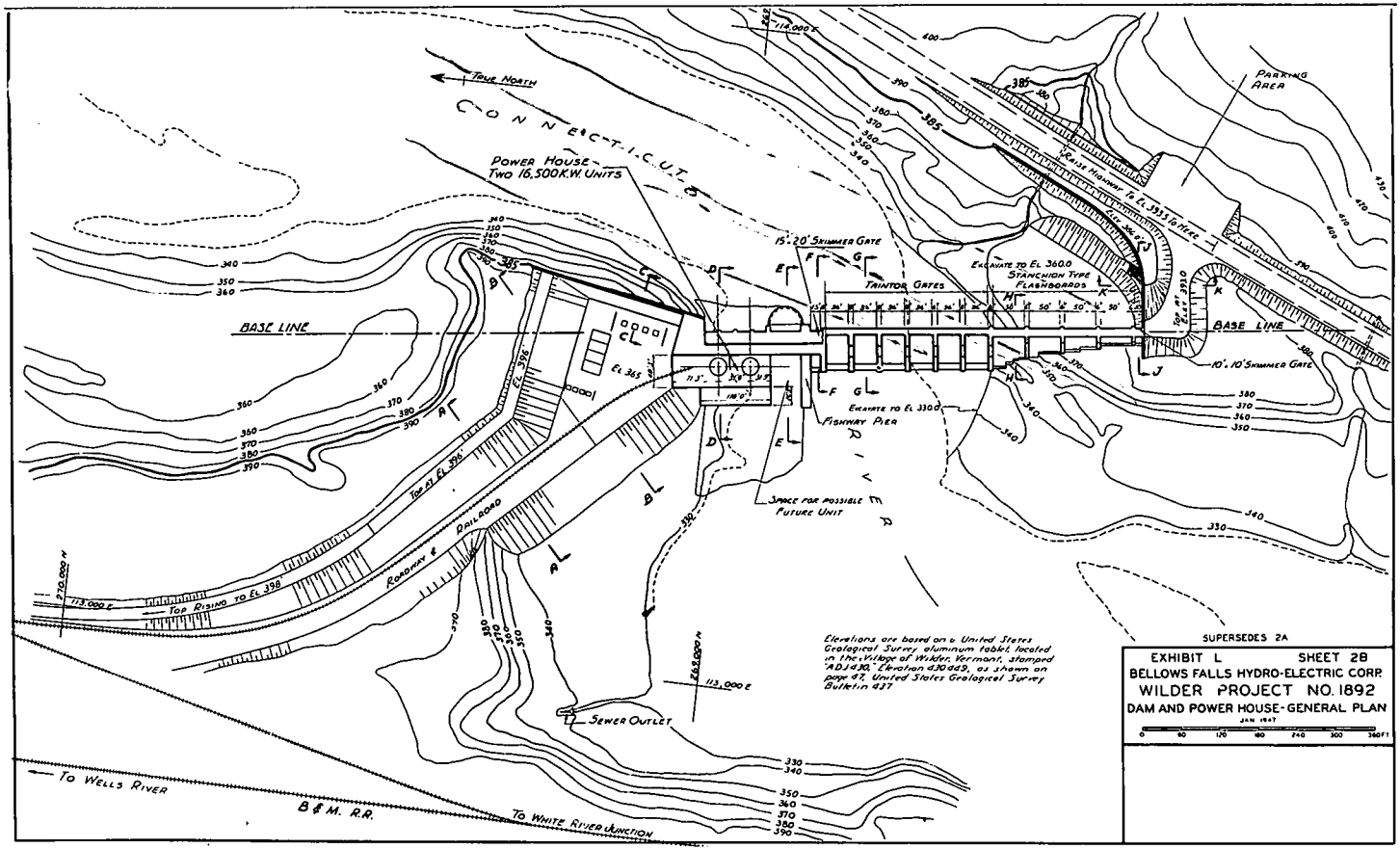


FIG. 4.—GENERAL PLAN, PROPOSED WILDER DEVELOPMENT.

From the north end of the power house a concrete north bulkhead wall extends about 220 feet to the center line of the north embankment. The total length of the concrete structures of the dam is about 940 feet.

The north embankment will extend from the end of the north bulkhead wall up to and around the end of the "Y" in the spur track connecting the Boston and Maine Railroad tracks with the power house. The north embankment will be about 1,800 feet long and the total length of dam will be about 2,920 feet.

On this general plan you will note that the high tension switch and transformer yard will be located on the Vermont side between the power house and the north embankment. A temporary substation with high tension transformers and control apparatus has been built up on the plateau near the road leading down to the present Wilder Plant. A temporary tie has been made at this substation to the new 110,000 volt transmission line from Wilder to Bellows Falls. This will provide a temporary power backup for this whole region during the construction of the new Wilder plant.

Final detail designs of the various structures of the proposed redevelopment have not been completed but general designs have been completed and many details have been decided for the major structures. Of course, we expect some changes will have to be made during construction, all such changes in design will be subject to the approval of the proper Commissions.

The south abutment wall will be a gravity section concrete wall founded on ledge and will have a maximum height of about 30 feet with the top at elevation 393.

The small skimmer gate, 10 feet wide by 10 feet high, will be next to the south abutment and will be mounted on a concrete gravity ogee section, about as shown by Section J-J on Figure 5. This gate will be of the dropping or skimmer type and will be controlled with an electrically operated motor driven twin screw hoist. Its primary function will be passing the debris or driki which may collect in the southerly end of the headwater pond. It will be necessary to excavate some 8 feet to 10 feet of ledge to accommodate this gateway and the discharge channel from it.

Between the small skimmer gate and the next section of the spillway there will be a six foot thick gravity concrete pier with the top at elevation 393.0.

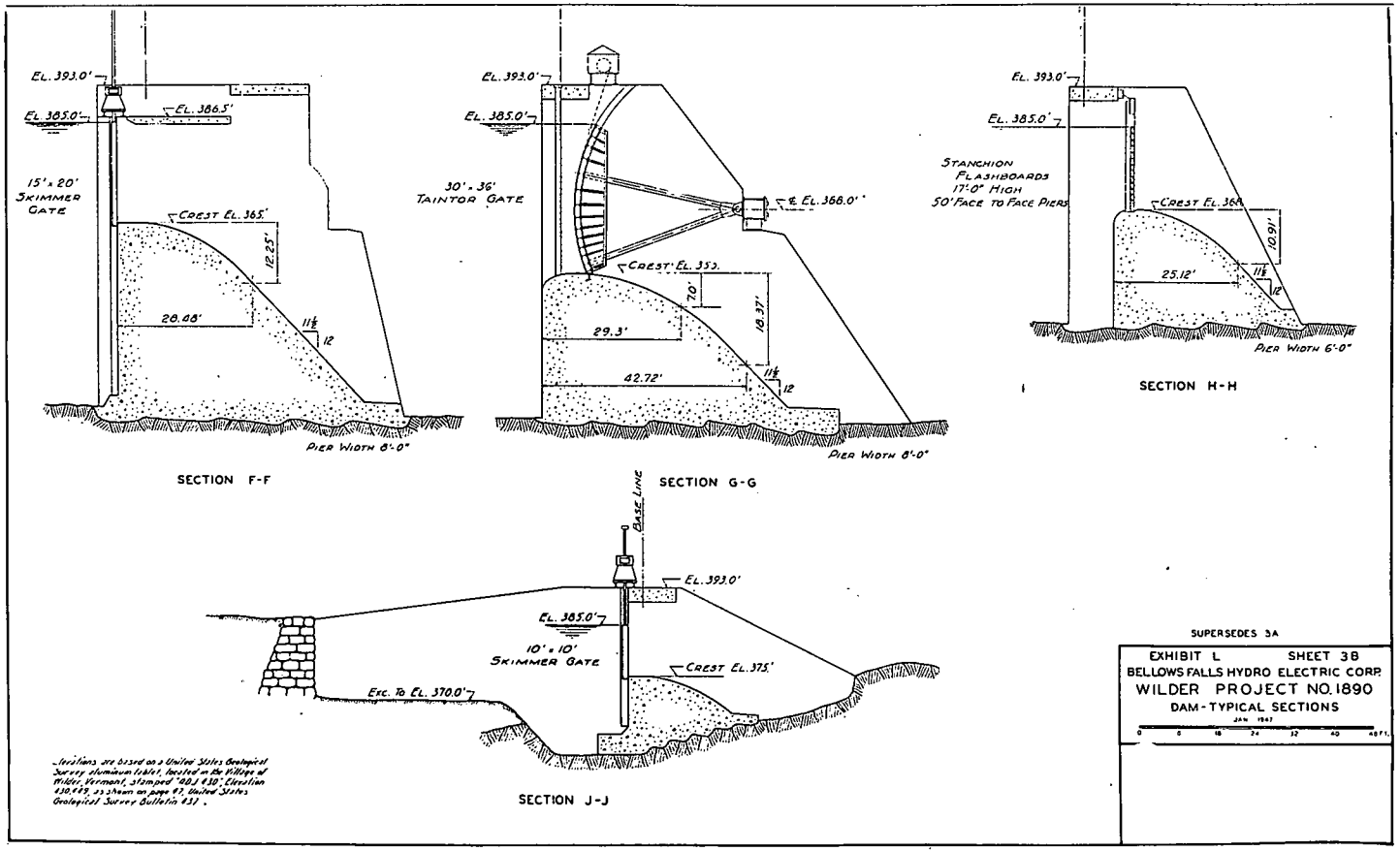


FIG. 5.—CROSS SECTIONS, SPILLWAY DAM AND GATES.

Proceeding northerly, next in the spillway will be four bays, each fifty feet long, of stanchion type flashboards, seventeen feet high, with six foot thick gravity concrete piers between bays. The dam in these bays will be a gravity concrete ogee section, on ledge, with crest at elevation 368.0 and with profile designed for an overflow depth of about 21 feet. The tops of the piers and of the floor of the bridge which will extend across all bays will be at elevation 393.0. The stanchion flashboards will consist of vertical steel beams or stanchions on about five foot centers, with the bottoms set in sockets in the concrete crest and the tops attached to the overhead bridge structure and so constructed that they can be readily released by one man in case of emergency. Between the steel stanchions there will be horizontal timber stop logs and when the emergency release is made both stop logs and steel members are usually lost in the flood. Sometimes in operation to meet unusual conditions part of the stop logs are removed and replaced without releasing the steel stanchions. This can be done with this type of flashboards without causing the dangerous condition which ensues if some flashboards are removed from a pin type flashboard layout, where such removal would prevent the pins from failing at the desired water level.

Between the fourth stanchion bay and the next section of spillway there will be an eight foot thick concrete gravity pier with top at elevation 393.0, then there will be six bays, each with a steel tainter gate thirty-six feet long by thirty feet high, with eight foot thick concrete gravity piers between the bays.

The dam at these tainter gates will be a gravity concrete ogee section, on ledge, with its crest at elevation 355.0 and with the profile designed to give proper nappe conditions for overflows up to thirty-six feet deep.

These tainter gates are made up of a steel face, which is a segment of a cylinder with a horizontal axis, having a heavy arm at each end connecting the face with a trunnion located at the center of the face circle, the trunnion supports being fastened to the adjacent piers through heavy prestressed bolts attached to anchors set deep in the piers.

Each gate will have a fixed, electrically operated, motor driven hoist and the ends, the lower edge and the lower part of the face of the gate will be provided with electric heating to prevent ice formation from interfering with the proper operation of the gate. There

are also spaces for electric heaters back of the steel seal plates in the faces of the piers at the ends of the gates. There will be rubber seals on the gates to contact the steel seal plates.

In addition to the heating provisions for preventing ice from forming on the gates there will be a compressed air bubbler system to prevent ice from forming in the water in front of the gates.

The top of each pier and of the floor of the bridge which will extend over all the tainter gate bays will be at elevation 393.0.

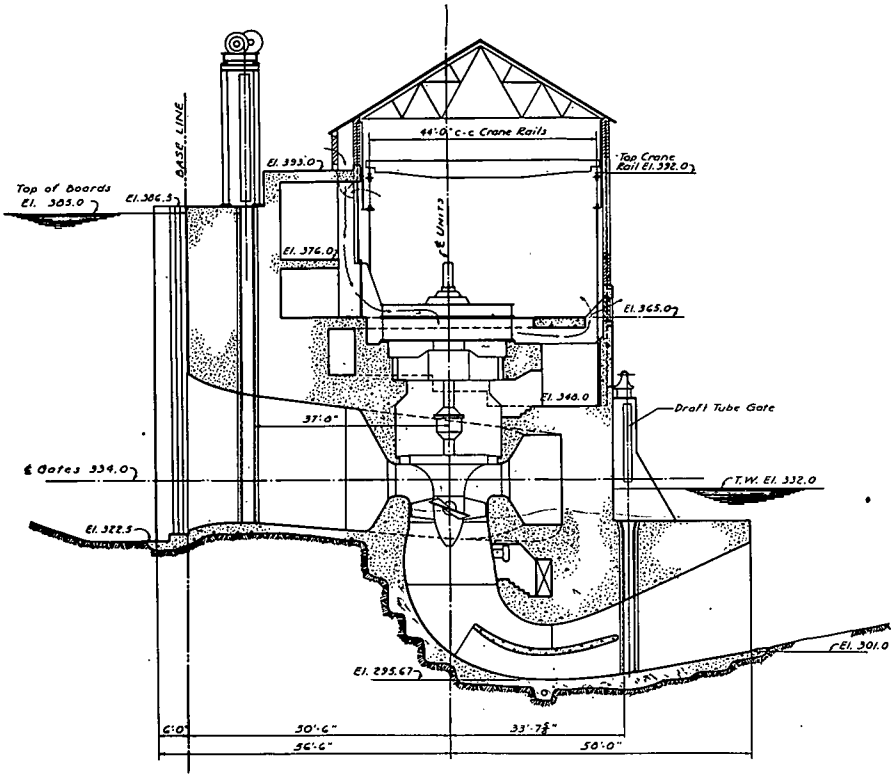
The design of the pier at the northerly end of the northerly tainter gate has not been finally determined but it is probable that it will be the so-called fishway pier and will be a gravity section about eleven feet thick with provision for an intake to and a support for part of the proposed future fishway.

Between the fishway pier and the six foot thick concrete gravity pier which forms the southerly side of the space provided for a possible future unit there will be a large skimmer gateway. This will consist of a concrete gravity ogee section with its crest or gate sill at Elevation 365, fitted with a dropping or skimmer type gate with a clear width of fifteen feet and provided with an electrically operated motor driven double stem screw hoist. This particular hoist is to be provided with a removable extension frame so that the gate may be used during construction to regulate the flow through a temporary opening some thirty feet lower than the permanent skimmer gateway. The entire downstream face of the gate will be planed to provide close contact with the gate frame.

An engine driven auxiliary generator will be installed to provide power for any of the gate hoists and other emergency operations in the event of the failure of the regular power supply.

The next structure, going northerly, is the space for a possible future unit. The final design of this structure has not been determined but we will do all ledge or other excavation required for the power house and tail race and will probably build a large part of the intake structure although we may not install head gates, using instead heavy concrete plugs which would be removed at the time of building the third unit.

The next structure will be the power house, Figure 6. The power house substructure and the intake structure will be built integral with the dam and will be of substantial plain and reinforced concrete construction, on ledge, providing proper waterways for the proposed



CROSS SECTION THRU POWER HOUSE

FIG. 6.

two generating units and a foundation for the steel frame and brick superstructure.

The intake will have two gateways or inlets for each unit. At the trash rack, which will have welded flats on six inch centers, the waterway will be twenty feet wide by thirty feet high with the top thirty feet and six inches below the normal full pond level at Elevation 385. The waterway at the head gate will be twenty feet wide by twenty-four feet three inches high. There will be a head gate for each inlet, one flat steel sliding gate and one flat steel wheel type gate for each unit. Each gate will have an individual electrically driven fixed hoist and the wheel gate and its hoist will be designed to open or close under full head with no back pressure.

There will be two generating units with a nominal capacity of 16,500 kilowatts each. The three phase, 60 cycle, 13,800 volt generators will be driven by vertical shaft, adjustable blade, Kaplan, propeller type turbines of 23,700 horsepower capacity at 112.5 revolutions per minute under a net operating head of 49 feet.

The draft tubes, which will be about fifteen feet in diameter at the throat ring, will be of the concrete elbow type, with horizontal steel and concrete splitters and vertical center piers in the lower and end sections.

The center lines of the turbine blades will be set about four feet below normal full plant tailwater level to reduce the probability of pitting or other troubles from cavitation. There will be a compressed air system for depressing the tailwater level in the draft tubes to below the blades during motoring operation when the units are being run as synchronous condensers and draft tube gates with individual electrically driven hoists and an adequate unwatering system will be provided for each unit to permit maintenance work on the turbines.

Considerable could be told about details of construction and operation of the Kaplan turbines and their control equipment, however this has been well told before and any who might wish to pursue this subject further I would refer to the paper on the McIndoes Falls Development presented before this society by Mr. Harry E. Popp and published in the September 1932 JOURNAL.

The next structure of the dam will be the north abutment wall, a heavy gravity concrete bulkhead section, on ledge, extending about 240 feet from the power house up the northerly bank to the north embankment. At the power house the top of this wall will be at elevation 393.0 but as you proceed north the top will be sloped up to a top elevation of 396.0 when the north embankment is reached.

The next and final structure in the dam is the north embankment which is practically all a natural bank. The top of the natural bank will be cleared of vegetable matter and a low embankment will be built, 20 feet wide on top, with a one on three slope on the upstream face and a one on two slope on the downstream face. The top will be at elevation 396.0 from the north abutment wall a distance of 500 feet, then the top elevation will be uniformly increased until it reaches elevation 398.0 at the end where it runs out into the natural grade.

This north embankment natural bank is mostly a fine silty sand

but there are some gravel strata so we are planning to place a blanket of low permeability earth on the upstream face of the bank and then protect this blanket with a cover of gravel.

We also plan to install a rather extensive drainage system in the downstream part of this bank or in the yard fill which also forms part of the embankment, and will construct a large inverted filter, with proper drains to the river, in the embankment near the downstream toe.

Between the power house and the north embankment there will be a yard, with surface elevation of about 365, the level of the main floor in the power house. This yard will be largely used as an outdoor switchyard and will contain a four bay 13,800 volt bus structure with oil circuit breakers and one bank of 13,800 volt to 44,000 volt transformers and one bank of 13,800 volt to 110,000 volt transformers.

In the power house there will be a low voltage station power supply distribution bus, with connections for all motor driven auxiliaries, such as water and oil pumps, station crane, gate hoist motors for both power house and spillway, and station and yard lighting and for charging equipment for the station control battery.

Switchboard equipment will be installed in a separate room in the northerly end of the power house, and will include all necessary control switches, indicating and integrating meters, signal lamps, protective relays and other equipment for the operation and control of all major apparatus.

Underground cables will transmit the current from the generators in the power house to the generator switch positions in the outdoor bus structure.

There will be five outgoing transmission or distribution circuits connected through oil circuit breakers to the outdoor 13,800 volt bus. Two 13,800 volt overhead circuits will run approximately east across the new pond and connect to the Granite State Electric Company system to replace the circuit connecting them with the present Wilder plant. A third 13,800 volt overhead line will run down the river on the Vermont side to supply the load in the White River Junction area of the Green Mountain Power Corporation. A fourth circuit will feed the low tension side of the 13,800 - 44,000 volt transformer bank providing the tie to the lines of the Central Vermont Public Service Corporation.

The fifth circuit will go through a 36,000 KVA, 13,800 - 110,000 volt transformer bank to a new 110,000 volt transmission line extending easterly across the new pond and thence southerly to the high tension switchyard at the Bellows Falls generating station. This new 110,000 volt transmission line, which has been completed, is about 40 miles long and has 13 wood pole structures per mile. These structures are the so-called pole arm type which consist of two vertical wood poles spaced about 28 feet apart, with a third horizontal pole serving as a cross arm from which the conductors are suspended. One of the vertical poles extends about 15 feet above the horizontal cross-arm and carries the protective ground wire.

The occurrence of what are here considered to be abnormally high flows on all or parts of the Connecticut River in November 1927, March 1936 and September 1938 combined with some other extreme local high runoff conditions here in New England has tended to make us rather flood conscious and has caused us to raise our estimates of required spillway flood capacities to figures considerably higher than we would have considered adequate some twenty odd years ago.

Figure 7 is a picture of the 1936 flood at Wilder at very near the peak. The pond level above the dam was about 380.1 or nearly 14 feet over the top of the concrete dam. The tail water level rose considerably more than the headwater level as water stood nearly 16 feet deep over the floor of the grinder room power house. We note in this picture that the paper mill buildings are still intact although the mill ceased operation in 1927.

The water level in the river at the extreme right in this picture is about elevation 385, the proposed new normal full pond level and the picture shows clearly that the clearances for the railroad track, which runs along the Vermont shore, are more than ample.

The proposed new spillway at Wilder, with all gates wide open and all flashboards off and with the water level at the dam held at elevation 385, normal full pond level, will have a discharge capacity of about 160,000 cubic feet per second, or more than 75 percent greater than the March 1936 flood, which was the greatest flood of record at Wilder. With this flow the freeboard at the lowest earth embankment would be 8 feet.

With all gates wide open and all flashboards off and with the pond level at the dam at elevation 380, the discharge capacity will be about 118,000 cubic feet per second which compares very favorably

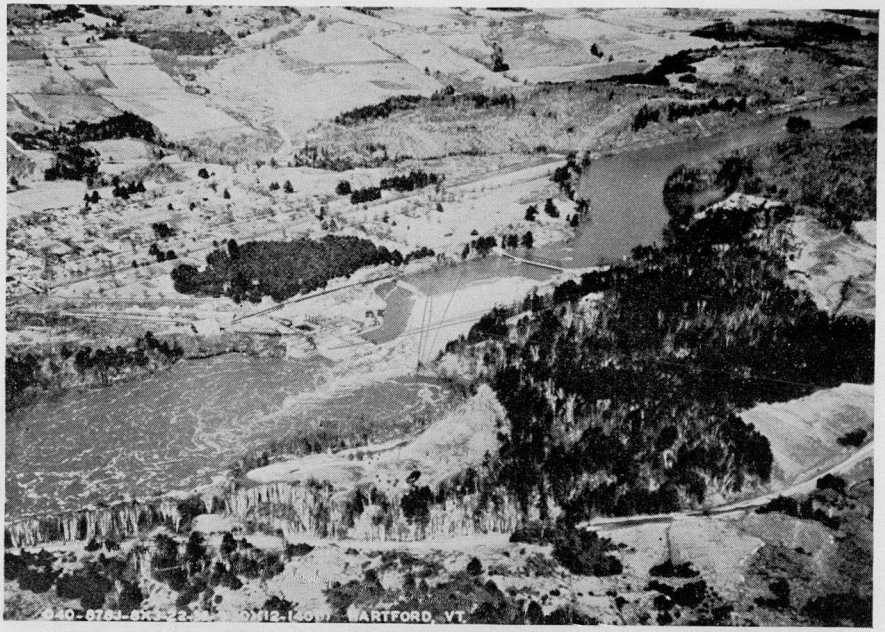


FIG 7.—1936 FLOOD AT WILDER. (8th Photo Section, A.C., U. S. Army)

with the discharge over the present dam, in the March 1936 flood, of 91,000 cubic feet per second with a headwater level of 380.1. With the new dam built and operated as proposed, if a flood of exactly the same size as the March 1936 flood should occur, the water levels throughout the length of the new Wilder pond would be practically identical with the levels reached during the actual March 1936 flood and for floods of that size the construction and operation of the dam as proposed would make practically no change whatever in the water levels upstream.

The proposed general operating conditions will be such that the water level at the dam will be below elevation 385, the so called normal full pond level, for the major portion of the time and the expectation is that this 385 elevation at the dam will never be exceeded unless we have a flood greater than about 160,000 cubic feet per second. Should a greater flood occur, which event appears extremely improbable, the excess will be passed at a somewhat higher pond elevation, but such higher pond elevation would be lower than would have occurred in a similar flood passing the present dam.

The surface drop in the pond as it approaches the part of the dam nearly parallel to the highway bridge is very marked where it goes over the old submerged timber dam.

In order to get the full upstream benefit of the increased flood passing capacity of the new dam, we propose to remove parts of the old concrete and the old timber dams, the canal headworks and the power plant headworks and parts of the present power houses.

In determining the discharge capacity of the proposed new dam, we computed an allowance for decrease due to the obliquity of approach; in addition to the usual allowances for end contractions, backwater effect and velocity of approach. The formula for the coefficient of obliquity which we used was: $Co = 1 - (0.0012 \text{ times angle a times } \frac{H}{D})$ in which "a" represents the angle in degrees between the thread of the stream and a line normal to the dam, "H" represents the head on the opening and "D" represents the vertical distance from the sill of the opening to the bottom of the river at that

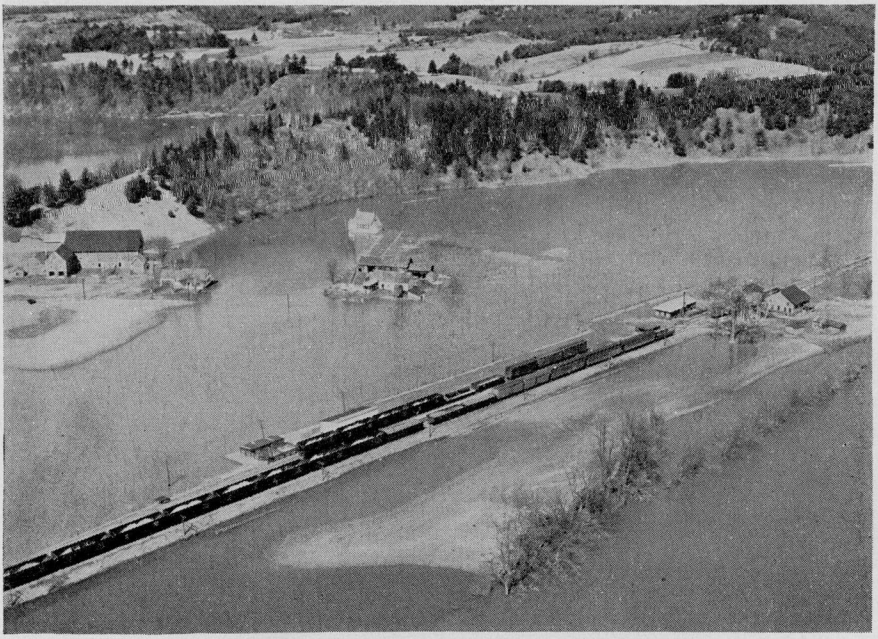


FIG. 8.—1936 FLOOD AT KENDALL STATION. (Fairchild Aerial Surveys, Inc.)

location. The decrease in discharge of the tainter gate section due to obliquity was figured as about 10.7 percent.

The total drainage area at Wilder is 3,375 square miles, of which only a minor part is controlled by storage reservoirs.

Figure 8 is a picture of the 1936 flood conditions in the vicinity of Kendall railroad station, formerly Pomponoosuc station, near the mouth of the Ompomponoosuc river. At the time this picture was taken the water level was about elevation 391, or about six feet higher than the proposed new normal full pond. The maximum water level here during that flood was a little over elevation 394.

Figure 9, is a picture of the 1936 flood conditions in the Bradford-Piermont district, taken by the U. S. Army 8th Photo Section, A.C. on March 22nd, 1936, at about the peak of the flood. This was taken from above the Piermont bridge looking northerly up the river valley. The maximum water level at the bridge was about elevation 412, or about 27 feet above the proposed new normal full pond level. At

FIG. 9.—1936 FLOOD, FROM PIERMONT BRIDGE NORTH.
(8th Photo Section, A.C., U. S. Army)

the peak of the flood the Bradford railroad station yard and track were submerged when the water level went to about elevation 413, or about 28 feet above the proposed new normal full pond level. In the upper right background we can see the flood waters covering the Newbury meadows where a flood lake from one half mile to more than a mile wide was formed. As previously mentioned, if a future flood of the same magnitude as the March 1936 flood should occur after the construction of the new dam, the river and flood conditions would be practically exactly the same as occurred in 1936.

Figure 10 shows a duration curve of the computed daily discharge of the Connecticut river at Wilder. The average flow for the period covered by the records used was 5,910 cubic feet per second and the curve indicates that this average flow was exceeded only about 26% of the time. The full water capacity of two units in the proposed plant will be about 9,000 cubic feet per second. This flow was equalled or exceeded for only 17 percent of the time while the 90 percent flow was only 1,450 cubic feet per second. The present old plant capacity of about 2,400 cubic feet per second is available for about 70 percent of the year.

The order of the monthly average flows is an arbitrarily selected arrangement which shows August as the driest with an average flow of only 1,020 cubic feet per second, September next with 1,600 cubic feet per second, July third with 2,000 cubic feet per second, February fourth with 2,360 cubic feet per second, October fifth with 2,800 cubic feet per second, June sixth with 3,250 cubic feet per second and April the twelfth or maximum month with an average flow of 25,300 cubic feet per second. Of course this arbitrary method of assigning a certain location on the duration curve to each month does not give a true monthly average flow as all floods, flash or otherwise, occurring in the low flow months are allocated to higher flow months and low flows during the higher flow months are allocated to the low flow months so all the averages below the median month are probably slightly low and those above the median month correspondingly high, but the approximation is good enough for most flow computation purposes and does give a true idea of about what average flows can be expected during the different months. The most significant fact about them is that during the two heavy growing summer months of July and August the average flow is not over about 2,000 cubic feet per second and in the other principal crop-growing month of June the average flow is less than 3,500 cubic feet per second.

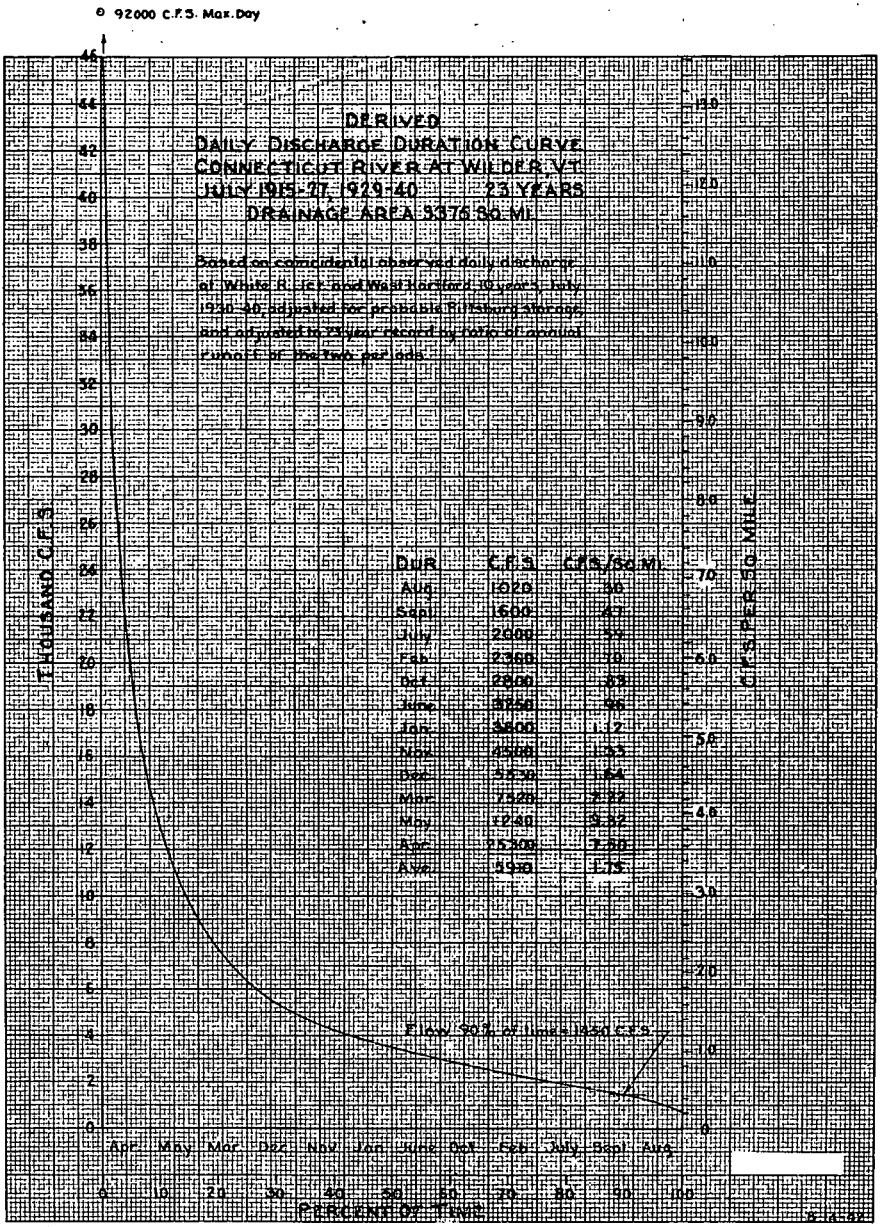


FIG. 10.—DURATION CURVE, DAILY DISCHARGE AT WILDER.

With the proposed new plant the expected average annual usable output is about 124,000,000 kilowatt hours.

With the expected method of operation there will be some drawdown of the pond for overnight and weekend pondage during most of the time when the river flow is less than 9,000 cubic feet per second, which is full plant capacity. This pondage drawdown will be something less than one foot each day, and each night it will fill nearly to the previous day's level. The lack of filling might amount to from 1½ to 2 inches daily so that the cumulative effect of this lack of filling combined with the Friday and Saturday drawdowns might amount to a total of about two feet for the weekend. This drawdown would be filled up again by the following Monday morning. The maximum operating pond drawdown would be five feet, to elevation 380, which will occur only rarely for repairs or some other emergency or under certain flood conditions.

The present Wilder pond extends upstream about 19 miles or nearly to the Fairlee-Orford bridge and the proposed new dam, with normal full pond level at elevation 385, will form a pond which will extend about 46 miles upriver along the natural course of the river or about 33 miles in a straight line northerly from the power house location. This is a very long pond for the amount of head involved but the future fluctuations of the pond water levels for various river flows will be of considerably lower magnitude than under present conditions.

At the Ledyard bridge, in Hanover, for a change in flow from 2,000 cubic feet per second to 41,000 cubic feet per second the present condition range of water levels is about 8½ feet, from about elevation 370 to about elevation 378.5 while the range after redevelopment will be only about 2½ feet, from elevation 385 down to about 382.5 as the headwater level of the new pond will be drawn below 385 for a flood of this magnitude. Similarly for a change in flow from 2,000 cubic feet per second to 91,000 cubic feet per second, equal to the March 1936 flood, the present range is about 19 feet, from elevation 370 to about elevation 389, while the proposed future range will be only about 5 feet, from about elevation 384 to about elevation 389.

We obtained a set of aerial photographs of the river on April 15, 1947, at about the highest flow for the spring of 1947, when the flow at Wilder was about 34,000 cubic feet per second which is appreciably less than the figure of 41,000 cubic feet per second, which we had

selected as the magnitude of spring flood that should be expected to be equaled or exceeded every other year or at least in 5 out of 10 years.

With this 41,000 second foot flow at Wilder and with the pond level at the new dam held at elevations 380, as expected, the backwater level will not reach elevation 385 for a distance of about 8 miles up the river to about the mouth of the Ompomponoosuc river. With this flood, the greatest increase in proposed over present backwater levels, above elevation 385, will occur at about mile 12, where the present backwater condition with this flood is elevation 385 and the proposed new backwater condition will be about 387, or 2 feet higher. As we proceed upstream, this differential between the old and new backwater conditions will decrease continuously until it reaches zero at about the South Newbury bridge, where the old and the new backwater levels will be practically exactly the same, at elevation 399.8. Up-

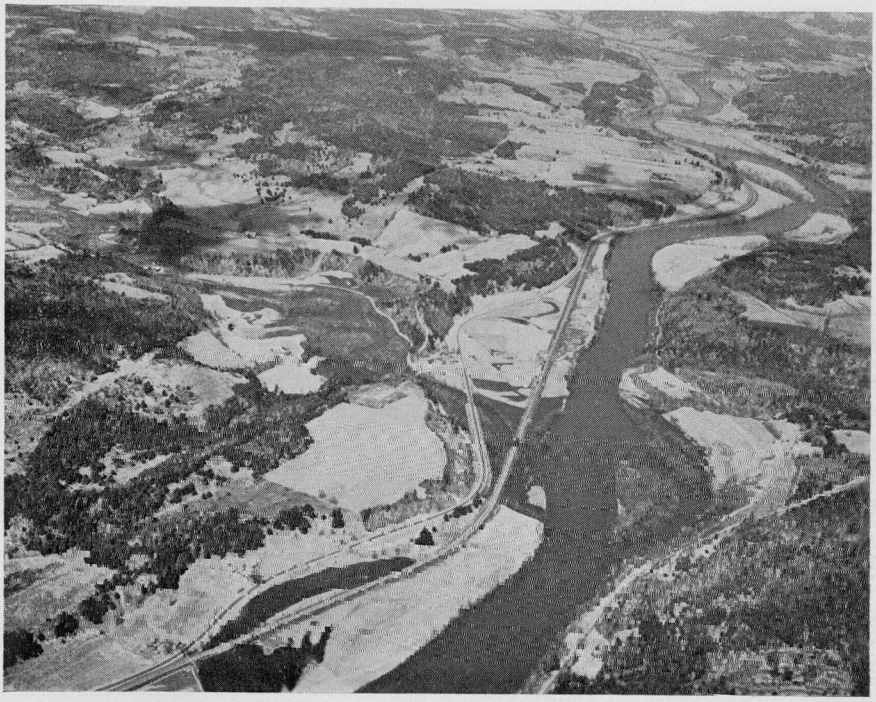


FIG. 11.—1947 FLOOD AT KENDALL STATION, SEE FIGURE 8.
(Fairchild Aerial Surveys, Inc.)

stream of South Newbury bridge and for floods of 41,000 cubic feet per second or more, the backwater levels will not be any higher with the new pond conditions than they would be under present conditions.

Figure 11 is a picture of the 1947 spring flood conditions in the vicinity of Kendall railroad station near the mouth of the Ompomponoosuc river. The water level about opposite the railroad station was about elevation 379.5 or 5½ feet lower than the proposed new normal full pond. A considerable amount of the meadow land in the foreground between the highway and the river bank and in the center of the picture between the highway and the railroad track will be flooded by the new pond at elevation 385, and during a flood of about 41,000 cubic feet per second the water level in this region will be just about elevation 385, the normal full pond level.

Figure 12 is a picture of the 1947 spring flood condition in the vicinity of Ely railroad station, near the Thetford-Fairlee town line.

Fig. 12.—April 1947 Flood Near Ely Station.
(Fairchild Aerial Surveys, Inc.)

The water level was about elevation 385.5 or ½ foot above the proposed new normal full pond level. You will note that in general the river is definitely within the present rather steep river banks although just above the center of the picture, at the outlet of Morey Brook, there is an appreciable area of rather low lying meadow which will be flooded by the new pond. In the upper right corner you will see the Fairlee-Orford bridge which is at about the northerly limit of the present Wilder pond during normal low flows.

Figure 13 is a picture of the 1947 spring flood conditions at Bradford and the mouth of Waites River. The water level here was about elevation 393, or eight feet above the proposed new normal full pond level. At the mouth of Waites river a flood of 41,000 cubic feet per second at Wilder would produce a backwater level of about 396.9 and the same flood after redevelopment would produce a backwater level of about 397.1.

FIG. 13.—APRIL 1947 FLOOD AT BRADFORD, SEE FIGURE 9.
(Fairchild Aerial Surveys, Inc.)

Figure 14 is a picture of the 1947 spring flood conditions on the Newbury meadows. The South Newbury or Bedell bridge, one of the few remaining old wooden covered bridges over the Connecticut river, shows just above the center of the picture. At that bridge the water level was about elevation 396.5 or 11½ feet above the proposed new normal full pond level. Substantial parts of these beautiful meadows are under water every year and this condition predisposes the farmers to fear trouble from the proposed new development. It is difficult for them to believe the truth of the statement that in the Newbury-Haverhill region, during periods of flows up to 5,000 cubic feet per second, which includes 66 percent of a normal year, not one acre of what is now really good meadow land will be injured, all injury will be confined to low-lying land which is not producing good crops even under present conditions and that during flood, flash flood and high flow conditions the actual future water levels will be

Fig. 14.—April 1947 Flood at Newbury Meadows.
(Fairchild Aerial Surveys, Inc.)

so nearly the same as under present conditions that any increase in damage due to the new development is truly negligible in that region. Of course in the lower valley where the rise in normal pond level is about 15 feet, in the towns of Norwich, Thetford, Lyme, Fairlee and Orford, there is some good meadow land that will be put out of agricultural use.

The present water area, under normal low flow conditions, in the stretch of river which will be occupied by the proposed new pond is about 2,100 acres and the total new pond area at elevation 385 will be about 3,245 acres, which indicates a land area of 1,145 acres as flooded by the new pond when full and with river flows up to about 2,200 cubic feet per second, which would include most of the normal flows in the three months of July, August and September.

Of this 1,145 acres of flooded land, 538 acres are in Vermont and 607 acres are in New Hampshire, and our classification of this land is, for Vermont, tillable meadow 189 acres, river bank and non-tillable land 349 acres, and for New Hampshire, tillable meadow 145 acres, river bank and non-tillable land 462 acres, or a total of tillable meadow in both states of about 334 acres. There will be no major farm buildings flooded out in the entire basin.

As the flow in the river increases above the 2,200 cubic feet per second, mentioned above, the backwater effect or rise in the water surface becomes greater, but when the flow has increased to about 5,000 cubic feet per second, which the duration curve indicates would normally occur during the months of March, April, May and December or possibly November, the backwater effects or increase in water level in the proposed pond from the new dam to the South Newbury bridge would be only about one half of one foot, which is a very small rise for a pond distance of about 36 miles.

Under present conditions, with a flow at Wilder of 5,000 cubic feet per second, the difference between the water level at the present dam and at South Newbury bridge is about 10 feet. This reduction from about 10 feet to about 6 inches in the relative water levels between Wilder and the South Newbury bridge, with the flow of 5,000 cubic feet per second, results primarily from the greatly increased water areas with the proposed new pond giving reductions in average velocity of about 60 per cent and corresponding increases in the hydraulic radii averaging about 100 per cent.

Considering the length of the proposed new pond, there are com-

paratively few changes in the railroad and the highways. The Boston and Maine Railroad parallels the river the entire length of the basin and runs along the river bank for a considerable distance but only about three quarters of a mile of track just south of the mouth of the Ompomponoosuc river will need to be raised, and that part only up to a maximum of two feet in height. However, there will be quite a number of changes and improvements to culverts and also at the many places where the track is immediately adjacent to the river bank there will be heavy rip rap placed to about one foot above pond level for bank protection.

For highway work, I have already mentioned the only major change in a State highway in New Hampshire, that is the raising on Route No. 10 at the New Hampshire end of the proposed new dam. The approaches to the Ledyard or Hanover-Norwich bridge may need to be modified. In New Hampshire there are several stretches of town and farm roads which will need raising or relocating. There is a section at the Hanover-Lyme town line which will need to be raised, and there are three other places in the town of Lyme where the river road or the road going up the valley of Grantbrook will probably need to be raised. There is also a short stretch of gravel road in Lebanon, leading to the present Lebanon-Wilder bridge which will need to be raised unless the bridge is discontinued.

In Vermont there are several places in the town of Norwich where State Highway No. 5 and a short stretch of state highway at the approach to Ledyard bridge will need to be raised and there are two stretches of state-aid highway, one on the river road running northerly from the Ledyard bridge and the other on the road running up the Ompomponoosuc river valley from Kendall station, which will need to be raised. This last mentioned stretch is on the road which was improved by the Government to serve the Thetford copper mine. For town roads in Vermont, work will need to be done on at least three stretches in Norwich and the road in Hartford at the approach to the Wilder-Lebanon bridge will need to be raised unless the bridge is discontinued.

If the Wilder bridge is discontinued, the only major bridge change might be at the Ledyard bridge, which may need to be raised a small amount at the Vermont end. There are several small bridges and culverts which will need to be raised or rebuilt in connection with work on the town roads in New Hampshire and in Vermont.

There are various changes which will need to be made in telephone, telegraph and power lines and in sewers and drains.

Mr. Irving B. Crosby of Boston is retained as consultant on geological features and Mr. W. F. Uhl of Charles T. Main, Inc. as consultant on several special problems and United Engineers & Constructors, Inc., of Philadelphia have been engaged as contractors for the complete construction at the damsite.

If the various delaying difficulties are cleared up in time we hope to start construction work on the new development early next spring and hope to complete the development in time to have the units delivering power in November 1950. The construction of the dam, power house and all of the development, except the work involved in clearing the basin and in the railroad and highway changes, will require crews of from 350 to 400 men.

HISTORICAL DEVELOPMENT OF SUBAQUEOUS TUNNELING

By Ole Singstad*

(Presented at 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 the Boston Society of Civil Engineers, held on November 17, 1948.)

The date of the first tunnel built by man is lost in the ages long past. There are records of soft ground tunnels built by the Assyrians, and as far back as eight hundred years before Christ a vaulted passage was built under the palace of Nimrud, who was a grandson of Ham. There is also an ancient brick-lined tunnel in soft ground under the river Euphrates, but I would not class this with subaqueous tunnels for it was built in the dry after temporarily diverting the river from its course.

Water is so frequently encountered in driving tunnels, even in case of those high above sea level, that in order to distinguish the type to be discussed a definition should be made. I consider a subaqueous tunnel to be a bore through more or less open ground or seamy rock forming the bed of a river or other large body of water, and necessitating some means of controlling the inflow. Under this definition subaqueous tunneling is a comparatively modern development which became practicable only with the development of the shield, and was largely restricted until compressed air was used to hold back the water.

The invention of the shield is credited to Marc Isambard Brunel, who obtained an English patent in 1818. In his specifications Brunel describes the shield as consisting of a number of cells, and a drawing which accompanied the specifications shows a shield circular in form, with one horizontal division across its diameter and four vertical divisions, making ten separate cells in all. Each cell was provided with a hydraulic ram reacting against a framework supported by the completed tunnel in the rear of the shield, and could be forced ahead independent of the other cells as the workmen excavated at its face.

When all the cells had been advanced a sufficient distance a section of the tunnel lining could be built within the tail of the shield,

*Consulting Engineer, New York City.

and it is interesting to note that, although Brunel stated that the lining might be made of brick or masonry, he preferred, and his drawings showed, a tunnel made of cast iron and afterward lined with brick or masonry.

Brunel's invention was tested in 1825 when the construction of a tunnel under the River Thames was commenced. The shield was used for three years although found defective in the loose, water-bearing material which was unexpectedly encountered, and the work was at a standstill for seven years after 1828 until Brunel built another shield which completed the tunnel in 1843. The tunnel, which is now a railroad structure, was originally intended for vehicles and consists of twin brick-lined tunnels having a common dividing wall from which the two arches spring. The outside section of the tunnel was rectangular, and the shield was 37 ft. 6 in. wide by 22 ft. 3 in. high, and was divided into 12 narrow cellls, each the full height of the shield and having two intermediate platforms. At the top of each cell was a projecting, chisel-shaped plate forming a cutting edge. Each cell could be shoved ahead independently by screw jacks which reacted on the finished lining of the tunnnel.

In 1849 Samuel Dunn of Doncaster took out a patent for a device made in one piece for tunneling in soft mud, and in 1864, forty-six years after Brunel's invention, another Englishman, P. W. Barlow, patented a shield which he described as a thin wrought-iron or steel cylinder of somewhat larger internal diameter than the outside diameter of the iron tunnel ring which was to be erected within the protection of its tail. Within and near the front end of the cylinder were to be upright parallel plates dividing it into a number of vertical compartments. Barlow's specifications stated that the annular space left between the earth and the tunnel lining "may be filled by injecting or running in fluid cement." In 1868 Barlow took out provisional protection for another shield having a transverse diaphragm with a central opening which could be closed when desired, while two years prior to this; R. Morton had devised a shield to be propelled by hydraulic presses in constructing a tunnel to be lined with cast iron rings with rubber packing in the joints.

Brunel's shield and the later patents described in the foregoing incorporate every essential feature of the modern shield, namely, the shell with its cutting edge at the front and space at the rear in which a portion of the tunnel lining may be erected, a diaphragm separating

the face from the tail and having openings capable of being closed, and hydraulic jacks around the circumference for shoving the shield ahead. A segmental cast iron lining was also proposed.

The Dunn, Barlow and Morton devices were never used in actual tunnel construction, but in 1869 shields resembling Barlow's inventions were developed and used in tunneling in both England and the United States. In that year J. H. Greathead began the construction of the Tower Subway under the River Thames, using a shield similar to Barlow's 1868 design, and A. E. Beach built a subway under Broadway, New York, extending from Murray Street to Warren Street, using a shield having the characteristics of Barlow's 1864 patent. It is generally accepted, however, that Barlow, Greathead, and Beach developed their ideas independently of one another.

Beach's shield was built partly of wood, with a cast iron cutting edge and a tail made of thin wrought iron in which the lining was erected. The face of the shield was divided into a number of horizontal shelves intended to prevent an inflow of the loose, sandy soil through which the tunnel was driven. The depth of the shelves from front to back was such that the earth might flow in upon them and reach an angle of repose before overflowing into the body of the shield.

The tunnel entrance was on Warren Street and the curved portion into Broadway was lined with flanged plates of cast iron. Brick was used for lining the straight portion, which was about 8 ft. in diameter. Beach installed a car in the tunnel, which was driven in either direction by air pressure or vacuum against its ends, supplied by a large Roots blower. An air duct, opening into the roof of the tunnel as shown in the photograph, extended across the street into City Hall Park and opened into an air well covered by a grating. This formed the necessary inlet and outlet for the blower.

The tunnel was opened to visitors, many of whom enjoyed the ride underground, but it did not achieve great popularity; and the construction of the elevated railroad rather eclipsed it.

The Greathead shield used in constructing the Tower Subway was 7 feet 1¾ inches in diameter and was advanced as a unit with screw jacks, which Greathead considered superior to hydraulic rams in a small tunnel. As in modern practice, grout was used to fill the space left by the tail of the shield outside the cast iron rings with which the tunnel was lined.

Both the Broadway Tunnel and the Tower Subway were constructed in dry ground, and the shields employed were designed for tunneling in this kind of material. Brunel had anticipated that the Thames Tunnel would be driven through impervious material and, when open ground which admitted water in large quantities was encountered, his difficulties were almost insurmountable, and the tunnel, only 1200 feet long, advanced slowly and proved very costly. Projects of a similar nature were held in disfavor until in 1868 when an Act of Parliament was finally obtained for the Tower Subway. No contractor willing to assume the risk of constructing this tunnel could be found until Greathead undertook the work in 1869.

Obviously, the shield alone had not solved the problem of subaqueous tunneling, and it was not until the use of compressed air in combination with the shield was employed that this type of construction met with any real success.

The inspiration for the use of compressed air in subaqueous tunneling doubtless came from the pneumatic caisson, which in turn was derived from the diving bell. The principle has long been known, and it is related that a certain John Taisner visited Toledo in 1538 where he saw two Greeks let themselves beneath the water in an inverted cauldron, and return to the surface without getting wet and without extinguishing a light which they carried. In 1778 John Smeaton, a British engineer, employed the diving bell in the construction of a bridge foundation and gave detailed instructions for building both the bell and a pump for supplying the compressed air. A few years later, when engaged on the Ramsgate Harbor works, Smeaton developed a diving bell essentially the same as those in use today.

In 1830, or contemporaneously with Brunel's development of the shield in connection with the Thames Tunnel, Sir Thomas Cochrane obtained a patent on a scheme for the use of compressed air in shaft and tunnel work in water-bearing ground. His description covers the use of air to expel the water and hold up the face, and proposes an air lock to permit the passage of men and material without loss of the air pressure. Cochrane did not put his scheme into execution, but a French engineer named Triger utilized it in 1839 for the sinking of a pit shaft at Châlons on the Loire.

There was evidently considerable independent thought concerning the use of compressed air as an aid in tunneling at about this

time, both in Europe and America. As an instance which traces back to the pneumatic caisson, Mr. D. C. Haskin, an American, who had observed this form of constructing bridge piers in the Missouri River, conceived the idea of tunneling under the Hudson at New York. He did not contemplate the use of a shield but thought that air pressure alone would hold up the face. A brick-lined shaft 30 ft. inside diameter was sunk on the New Jersey side of the river. The shoe of the shaft was made of wood, with a cutting edge of boiler plate. The shaft walls were 4 ft. thick at the bottom and 2 ft. 4 in. thick at the top.

The shoe was placed in November, 1874, but owing to litigation with the D. L. & W. R. R., work was stopped until September, 1879. In November, 1879, the shaft was completed, after sinking through 5 ft. of ash fill, 50 ft. of silt and 5 ft. of sand. An ordinary hand pump kept the work dry until the sand was reached.

At the time when compressed air was first put in the shaft it was intended to build one double-track tunnel 24 ft. × 26 ft. in the clear, but unsuccessful efforts to start this large tunnel through the material at the shaft resulted in a change to two single-track tunnels 18 ft. high by 16 ft. wide, of oval section, lined with brick. An expedient adopted to negotiate the difficult first stages between shaft and tunnel consisted in starting at the shaft with a small iron ring 2 ft. 6 in. then increasing the diameter of each succeeding ring by about 18 in. until the full size was attained as shown in the longitudinal sectional view.

The permanent rings were made of ⅜ in. plate 2 ft. 6 in. wide, with 3 in. angle-iron flanges. Fourteen segments comprised a ring. The silt at the face was kept as a series of steps, the crown of the arch being adjacent to the highest and most advanced step. Air pressure of 18 lbs. per sq. in. was used. After a bad blowout Haskin's superintendent devised a method of driving a small pilot tunnel in the middle of the bore, considerably in advance of the main tunnel. This was lined with iron and radial struts were extended from it to support the light iron lining of the tunnel proper. About half of the excavated silt was mixed with water and blown out of the tunnel through a pipe by the air pressure.

After many vicissitudes and a blowout which cost twenty lives, the work was stopped until 1889 when an English Company, S. Pearson & Son, assumed the contract. A shield was used and a heavy iron

lining was substituted in place of brickwork. Mr. Haskin lost his fortune in the first attempt, and this second one also came to a standstill because of a failure of finances. I will not attempt to tell the detailed history of this tunnel, or rather the two tubes, which were begun by Haskin, except to say that in a reduced, circular cross-section lined with cast iron, they eventually became the northerly pair of the four tunnels which comprise the existing Hudson & Manhattan Railroad.

It may be interesting to note, in connection with Haskin's use of compressed air for tunneling, that there was published in 1880 a pamphlet promoting an invention, by a Mr. Olney B. Dowd, of a shield and mechanical excavator to be used in constructing a tubular metal lined tunnel in soft, water-bearing ground without employing compressed air. Dowd, in his pamphlet, ridiculed the use of compressed air for any kind of tunneling, and in support of his opinions, pointed to the disaster in Haskin's tunnel in which twenty men were drowned.

Haskin shares the distinction of first using compressed air in tunneling with Mr. Hersent, who in 1879 constructed a cast iron lined tunnel of approximately rectangular section, 4′10″ high and 3′10″ wide, at Antwerp in connection with dock works in the Scheldt. This tunnel was driven through fine silty sand using compressed air.

In 1889, when Pearson & Son undertook completion of the Hudson tubes started by Haskin, they used compressed air in conjunction with a shield designed by Mr. E. W. Moir under the direction of Sir Benjamin Baker. Two years before this, in October, 1886, Mr. Greathead, of whom I have spoken before, began a tunnel for the City and South London Railway under the Thames. The shield was built for a tunnel 10 ft. 2 in. diameter and had a skin ½ in. thick made of two ¼ in. plates. The six hydraulic jacks were housed in cast-iron segments which reinforced the body of the shield, and in front of these there was a diaphragm having an opening in its center. Sections of the tunnel in open ground were driven with the aid of compressed air, constituting the first use of shield and compressed air together, though this use had not been anticipated. Mr. Greathead also devised a so-called grouting pan consisting of a steel cylinder equipped with stirring blades and connections for forcing the grout outside of the tunnel by compressed air. The one other appurtenance of the modern shield which I have not mentioned is the mechanical

erector arm for placing the segments of the tunnel lining. The earliest record I find of this is also to the credit of Mr. Greathead, who shows such a device in his design for the Woolwich tunnel shield.

The shield for the Woolwich tunnel was designed and constructed in 1874 by Greathead, and contemplated the use of compressed air. The contract for this work was let in 1876. Ordinary tunneling methods were unsuccessfully employed and the work eventually abandoned.

The combined use of the shield and compressed air in tunneling, planned as such, was inaugurated in 1888 in the construction of the Sarnia Tunnel, a 21 foot cast iron lined single track railroad tunnel under the St. Clair River between the United States and Canada, opened to traffic in 1890.

A 9 foot diameter tunnel under the Mersey River at Fidlers Ferry for the Vyrnwy Aqueduct was also started in 1888. During the first 18 months of this work ordinary tunneling methods were used and but little progress made. Another contractor then took over the work and completed it successfully with shield, cast iron lining, and compressed air.

Since this time there has been a continued use of the method for important tunnel projects throughout the world and many miles of tunnel have been safely and efficiently constructed by it. In the New York area, nineteen important subaqueous projects have been successfully completed, using the shield and compressed air, since 1900. Among these is the Holland Tunnel, under the Hudson River, which has become the best known of all subaqueous tunnels. There are two reasons for this. It was the first long tunnel designed for automotive traffic, and the problem of ventilating such tunnels was solved for the first time. Another reason is its striking financial success. Its total cost of forty-eight and a half million dollars, was fully amortized during the first thirteen years of operation, and at the completion of twenty years of operation on November 12, 1947, it had accommodated two hundred and forty-two million vehicles. It is financially the most successful vehicular toll project whether bridge or tunnel.

Among these tunnels is also the Lincoln Tunnel, under the Hudson River, and the Queens Midtown Tunnel, under the East River, which has the distinction of having been built through the most difficult ground of any large subaqueous tunnel, its diameter

being 31 feet. For this reason it also cost more per lineal foot than any other tunnel.

The fourth large subaqueous vehicular tunnel in New York is the Brooklyn Battery Tunnel, construction of which was started in 1940 and was temporarily suspended during the war. Work has again been resumed on it.

Although it is not part of the shield and has no direct use in the actual process of tunneling, there is another device which I should mention in this connection. This is the safety screen, which I believe was first used in 1892 in the construction of the Blackwall Tunnel in England. The screen is virtually half of a diaphragm or bulkhead and fits tightly against the upper part of the tunnel lining some distance in the rear of the shield. In case of a bad blow causing a loss of air and an inrush of mud and water, the screen forms a trap for some of the air behind it, and prevents the water from completely filling the space. In practice the screen is moved ahead from time to time, keeping perhaps within an average distance of 100 feet of the shield.

In connection with these historical references I might point out the essential and serious difference between the application of compressed air in sinking a caisson, and in driving a tunnel. The cutting edge of a caisson is usually horizontal, in consequence of which the pressure needed to balance the hydrostatic head is uniform around the perimeter. At the cutting edge of the tunnel shield, on the other hand, while the air pressure is uniform over the whole face, the hydrostatic pressure increases from top to bottom, so that for such shields as those employed in the Holland Tunnel for example, which were 30 ft. 2 in. in diameter, the hydrostatic pressure at the bottom of the shield was over 1,900 lbs per square foot higher than at the top. Many attempts have been made to overcome this difficulty, beginning with the Greathead shield which I have mentioned. This had a diaphragm behind the cutting edge, extending about two-thirds down from the top, and an air lock in back of the diaphragm. The diaphragm was supposed to form a water trap which would prevent the escape of any large volume of air in case of a blow, but owing to the small volume of air in front of the lock it probably would not have prevented a complete loss of pressure before the trap came into action, with a consequent drowning of any men in the shield. Shields have been designed with compartments in which it was pro-

posed to maintain different air pressures more or less corresponding to their vertical positions, but the fact that there is no control over the ground just ahead of the shield makes all such ideas impracticable. At the present time all that can be done in bad ground such as sand or gravel is to vary the air pressure while excavating and breasting different parts of the face. That is, the pressure will be lowered while opening up the top of the face, in the meantime permitting water to enter the lower part; then with this upper part safely protected by breastboards, the pressure will be raised and the lower part excavated. It is also now general practice to place a temporary layer of clay or "blanket" on the river bed over the tunnel while it is being driven in open ground, of sufficient thickness to produce a counteractive weight of material in addition to water, equal to or greater than the anticipated maximum "unbalanced" air pressure at the top of the heading.

While the shield and compressed air method of subaqueous tunneling was being developed, the idea of grading the river bed and laying a tunnel in pre-built sections upon it, or building the tunnel in the dry in a cofferdam, received some attention. Such a method was promoted in a pamphlet published in 1854, which proposed a cast iron tube to be laid on the river bottom after proper grading, known as "J. R. Miller's Patent Submarine Avenue."

The City of Chicago built the Washington Street tunnel under the Chicago River in 1866-69. This was not really tunneling, however, since it was constructed in the open by excavating between two cofferdams. The structure had two 11 ft. roadways 13 ft. high and a 10 ft. sidewalk. After the great fire of 1871 this tunnel formed the only link between the west side and the business district until new bridges were built. At two different times the roof of this structure was lowered to give greater depth to the river, and it now comprises part of the city subway system. The same method of construction was used in the La Salle St. and Van Buren St. tunnels under the Chicago River.

What is known as the trench method of subaqueous tunnel construction has been developed in the United States within the last forty years with the objectives of reducing construction cost, and eliminating the hazards incidental to work in compressed air. It consists of the excavation of a trench in the bed of the river and placing therein sections of completed or partly completed tunnel which are

floated into position and lowered into the trench. The joints between sections are temporarily sealed from the outside and permanent joints completed after the construction has reached a point where the tunnel can be unwatered and work prosecuted from inside.

This form of tunneling has not been widely used because of the interference produced with waterfront and shipping activities. Earlier examples of this method are the Detroit River Tunnel, built from 1906 to 1910, for railroad use, between Detroit, Michigan and Windsor, Ontario, and the rapid transit tunnels under the Harlem River in New York.

More recently, the Oakland-Alameda Tunnel for vehicular traffic was constructed by the trench method under an arm of San Francisco Bay between Oakland and Alameda, California; and the Detroit Canada Tunnel for vehicular traffic was constructed under the Detroit River, using both the trench and shield methods. The river section of the Bankhead Tunnel, under the Mobile River at Mobile, Alabama, and a vehicular tunnel under the Maas River at Rotterdam, Holland, which was completed during World War II, were both built by the trench method.

THE WATER SUPPLY TUNNELS OF THE BOSTON METROPOLITAN DISTRICT

By Karl R. Kennison, Member*

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

In the development of the District's new Quabbin supply from the Ware and Swift rivers, many works of great variety have been constructed in the last 22 years. These include 3 large earth dams for Quabbin Reservoir; 6 small dams; 48 miles of highways; 1 large cemetery; 2 municipal sewer systems; 9½ miles of trunk sewer; 25 miles of electric transmission lines; 2 hydroelectric power plants and 57 1/3 miles of aqueducts which include about 33 1/3 miles of deep rock tunnels. These are in addition to many major works, constructed following the organization of the District about 50 years ago, such as the Wachusett and Sudbury reservoirs and the Wachusett and Weston aqueducts, which included a few short sections of grade line tunnel. This paper has to do with the recent construction of the 33½ miles of deep rock tunnels and with their possible future extension.

Quabbin Aqueduct—Design

The first tunnel built by the Metropolitan District Water Supply Commission was Quabbin Aqueduct, extending westerly from Wachusett Reservoir 24.61 miles to tap the new supplies from the Ware and Swift rivers. Although this tunnel, built 1927 to 1934, cannot now compare in length with the recent tunnel construction by the New York Board of Water Supply, it and the Coast Range tunnel of San Francisco's Hetch Hetchy Aqueduct, which was under construction at the same time, were then of unprecedented length.

Quabbin Aqueduct was built as a grade-line tunnel of the so-called horseshoe section, 11′ wide between vertical walls and 12′9″ high, equivalent in area to a 12′9″ circle. This section continued throughout its length, except that a length of 500′ at each end was constructed as a grouted pressure tunnel with circular cross section. This was done at the west end next to the intake from Quabbin

*Chief Engineer, Construction Division, Metropolitan District Commission.

Reservoir on account of the unbalanced head of about 115′ from the reservoir whenever the tunnel should be drained for inspection; and it was done at the east end on account of the unbalanced head from the Ware River of about 180′ behind the closed control gates, which were located at the outlet into Wachusett Reservoir.

There were several other interesting features in the design of this tunnel. There was ample rock cover, and high ground-water level well above any hydraulic grade in the tunnel, at all points except at the easterly end. Here the original surface of the ground along the tunnel profile dropped gradually to Wachusett Reservoir and was so low that the tunnel could have been built at the minimum hydraulic grade line, discharging at the east end through a portal and an open channel in the rock. That however in addition to requiring an expensive open cut from the portal would have made it impractical to construct an electric generating plant having a tail race into the Wachusett Reservoir and utilizing the head from Quabbin Reservoir. Accordingly the actual grade of the tunnel invert was lowered about 180′ so that the tunnel had everywhere about 200 feet of rock cover and so that the control gates could be located in the power station at the top of the uptake shaft. This required a deep underground pumping station at the bottom of the shaft to provide for emergency unwatering of the east end of the tunnel, which, in any event, could be completely drained by gravity into Wachusett Reservoir for only about two-thirds of its length.

Another interesting feature is the downtake shaft from the Ware River Intake Works at about the mid-point of the tunnel. This intake is 126′ higher than Quabbin Reservoir so that the location of the control gates at the Wachusett end of the tunnel makes it possible to close these gates and to divert the Ware River westerly to Quabbin Reservoir. Such a condition of operation would put an excessive head on all works at the Wachusett end if it were not for a spillway provided at the top of the next shaft, Shaft 2, about 9900′ westerly. The discharge of flood flows from the Ware River into Quabbin Reservoir is made through an uptake shaft located about 11,100′ east of the Quabbin-end intake and discharging through one-way tide gates in such a way as to obtain long-time storage for this water in Quabbin Reservoir before it is drawn from the intake as needed into Wachusett Reservoir.

Many interesting details of the design and construction of this

tunnel have been explained in papers before this Society and in the Engineering press and are referred to here only briefly.

Construction

The method of constructing the tunnel was dictated by a number of circumstances. The first objective was to reach and tap the Ware River so as to obtain this addition to the District's supply as soon as possible. The Ware River Intake was located at Shaft 8 in Coldbrook, 13.24 miles west of Shaft 1 at the Wachusetts Outlet. On account of the urgency which then existed everything was done to hasten the construction of the Wachusett-Coldbrook section of the tunnel.

The Commission was organized in August, 1926, and following extensive preliminary surveys and subsurface explorations contracts were prepared for work which could be started in advance of the preparation and letting of the main contract. One of the construction shafts, No. 5, was 656 feet deep, much deeper than the others which varied in depth from 313 to 457 ft. Accordingly the first contract was for this deep shaft. It was executed March 30, 1927, followed on April 27 by a contract for an electric transmission line extending along the entire surface right-of-way so that there would be no delay in supplying all the shaft contractors with power. Two contracts for the remaining construction shafts, 2, 3, 4, 6 and 7, were executed June 1 and 2, clearing the decks for the main tunnel contract.

Since this tunnel was to be completed and put into early operation to divert the Ware River directly into Wachusett Reservoir, it was extended beyond Shaft 8, 0.95 mile, and a temporary heavy structural-steel bulkhead built near the west heading, so that the work of extending it later to the Swift River and Quabbin Reservoir could be carried on without interruption by the Ware diversion through the easterly end or Wachusett-Coldbrook section. This bulkhead was an interesting feature of the construction, particularly since it had to withstand a head of 260′ and be equipped with tide gates so that following the completion of the second stage, or the extension to Quabbin Reservoir to tap the west branch of the Swift River, that supply also could be made available by direct diversion into the tunnel several years prior to the actual completion of Quabbin Reservoir. In other words the diversion of the flood flows under high head from the Ware had to continue to Wachusett Reservoir in an easterly

PERMANENT TIMBER SUPPORT IN CONSTRUCTION OF QUABBIN AQUEDUCT, WEST HEADING FROM SHAFT 9.

direction only and be prevented by the tide gates in the tunnel from flowing westerly, until the reservoir could be completed ready for storage.

Bids for sinking the two waterway shafts, No. 1 and 8, and completing the entire 14.19-mile section of the tunnel were opened November 19, 1927, but the situation then became complicated by a suit brought by the State of Connecticut in the Supreme Court to prevent the proposed diversions from the drainage area of the Connecticut River. The Commission decided to delay the execution of the main tunnel contract but permitted the shaft contractors to proceed in the tunnel. They excavated in all 1.56 miles in the 12 headings from the construction shafts before these were taken over under the tunnel contract, which was finally executed on April 30, 1928, after the Connecticut case was settled in favor of Massachusetts. The need for haste also called for a close spacing of the construction shafts, which average 1.89 miles apart, the maximum distance between any two being 2.20 miles.

When the tunnel was later extended for the additional 10.42 miles to Quabbin Reservoir there was not the same need for haste; and the maximum distance between shafts was 3.17 miles and all three con-

struction shafts, Nos. 9, 10, and 11, together with the intake shaft at the Quabbin end were included in a single contract. Another waterway shaft, No. 11A, was later sunk under this same contract to tap the completed tunnel and to provide the uptake for the Ware flood flows to discharge into the East Branch of Quabbin Reservoir.

It was possible to sink all the shafts through sound ledge, from points varying from only 3 to 34 feet below the original ground surface, except at Shaft 3 where an open caisson had to be sunk through 61 feet of overburden. The four easterly shafts were in the Wachusett Watershed and the contractor was required to settle all the drainage and chlorinate the effluent. At the two shafts nearest Wachusett Reservoir, the contract required the effluent to be passed through slow sand filter beds. Actually the drainage was so small at Shaft 1 that the bed of the settling basin itself acted as a satisfactory natural filter. Generally speaking the rock encountered was a compact and uniformly sound schist varying to granite. Only 1.52 per cent of the length required permanent support. It was also uniformly dry, the leakage in the entire length, with its freely vented invert being only about 3 or 4 m.g.d. representing to some extent, I believe, a condition of rock drainage and not of stable flow.

The shafts were lined with concrete as they were sunk, all construction shafts being circular and 14 feet diameter inside the lining to provide space for the cage or skip guides and other contractor's equipment and for dropping alignment wires. The progress made in sinking such shafts is summarized briefly in the following table. Shaft 1 containing the uptake waterway and the separate access well to the unwatering pump, Shaft 8 containing the downtake waterway with cast-iron helical lining, Shaft 11-A containing the uptake waterway, and Shaft 12 containing the intake screens and stop-log controls, were not typical construction shafts and are not included in the table of progress nor in that of cost.

The tunnel was driven by the full-face method everywhere except from Shaft 5, 6, 7 and 8, where the slower heading-and-bench method was used. The quantity of dynamite used averaged about 5.7 lbs, per cu. yd. of rock. Shovel-type mucking machines were used in practically all the headings. However, in one heading the contractor used a dragline type of mucker with a 60′ boom, permitting the loading of the full muck train.

PROGRESS

	Shaft Construction		*Tunnel Excavation*	
At Shaft No.	Average weekly progress sinking in rock, ft.	Average number of men in two shifts	Weekly progress per heading when going at best speed, ft.	Average number of men, total in two shifts at each shaft
2	21	35	116	119
3	19	24		
4	19	37	118	125
5	20	39	96	90
6	17	31	94	84
7	19	35	71	67
9	24	41	134	78
10	19	37	120	64
11	34	37	125	72

In the lining operations the aggregates were crushed from the tunnel spoil. For the east portion they were batched at the top of Shafts 2 and 4 mixed at the forms and fed by an air-piston concrete gun into collapsible side-wall and arch forms at rates varying from about 700 to 1000 ft. and averaging 860 ft. per week per tunnel plant; and the invert was placed subsequently in a separate operation at rates varying from about 1000 to 2000 ft. per week for the longer stretches of operation. For the west portion the concrete was mixed at the top of shafts 5 and 7, dropped down the shaft into special carriers and placed in the side-wall forms and the invert and shot by the concrete gun into the arch forms, at rates varying from about 275 to 525 ft. and averaging 370 ft. per week per tunnel plant.

The tunnel contractor was required to provide medical and hospital service. There were 15 fatal accidents in all the shaft and tunnel contracts.

COST

The costs summarized are only those which are considered typical and useful for purposes of comparison. The cost figures for the west portion or Coldbrook-Swift extension are kept separate because although they do show the actual price paid, they are believed to be somewhat out-of-line and hardly typical. Such a low bid was received for this extension, particularly for the items included in the concrete lining, that the low bidder was practically invited to with-

draw his bid and was required to justify it with detailed figures and explanation. However, he insisted on going through with it and made good, without loss so far as the writer knows.

The shaft costs are the equivalent per-foot prices which the District actually paid the contractors for sinking in rock and lining the typical construction shafts, and for excavating and lining the typical grade-line tunnel, not pressure grouted. The shaft linings were thin, only 3 in. minimum or "A-line" distance to projecting points of rock and only 12 in. additional or 15 in. total pay-line thickness, with only such grouting as necessary to cut off any inflows that would interfere with construction. For the tunnel also the minimum "A-line" distance was only 3 in. for the vertical walls and for the invert, and 8 in. for the arch, with the same 12 in. additional pay-line thickness, except only 2 in. additional for the invert.

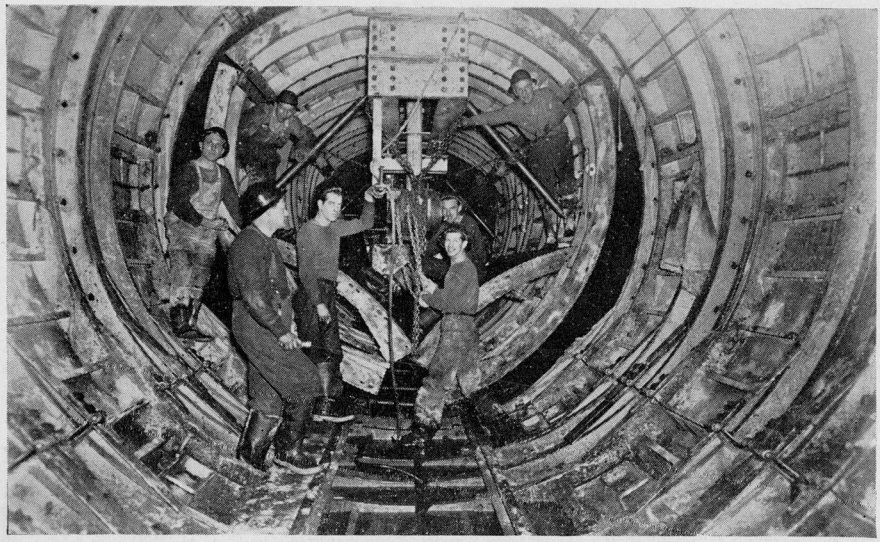
In some sections thicker lining was used and the table shows how the cost varied with the thickness of lining. The concrete mix varied with conditions but these costs are based on a uniform 1.5 bbls. of cement per cu. yd.

TOTAL COSTS PER FOOT BASED ON BID PRICES FOR ALL THE CONTRACTS

Construction Shafts in Rock 14 feet diam.		Grade-line Tunnel area equivalent to 12′9″ diam.	
3 in. "A-line"	6 in. "A-line"	3&8 in. "A-line"	8&12 in. "A-line"
East or Wachusett-Coldbrook Section (Bid in 1927)			
$184	$200	$107	$120
West or Coldbrook-Swift Section (Bid in 1931)			
$155.50	$168	$80	$88.50
An additional cost, chargeable to the shaft, over and above the per-foot cost of the tunnel, of excavating with shaft equipment each heading adjacent to the shaft, say for a length of 200 feet under typical normal conditions, was			
		$4,900	$5,300

HULTMAN AQUEDUCT—SUDBURY RESERVOIR SECTION—DESIGN

The Hultman Aqueduct extends 23.8 miles from the Terminal Chamber of the Wachusett Aqueduct to the Chestnut Hill area in Boston. It was the District's first pressure aqueduct. Its purpose was to bypass Sudbury Reservoir, to reinforce the old grade-line



TELESCOPING THE TUNNEL FORMS, HULTMAN AQUEDUCT UNDER SUDBURY RESERVOIR.

aqueduct system and eliminate sources of pollution involved in the continued use of the old works, particularly the Cochituate Aqueduct built in 1848 and the Sudbury Aqueduct built in 1878. It will carry the hydraulic grade at the Wachusett Terminal, El. 278.5 feet above mean low tide, all the way to Chestnut Hill with a loss of only about 8 or 10 feet and eliminate the present necessity of pumping at Chestnut Hill to the Southern High Service.

The west portion 18.3 miles long was constructed in 1938 to 1940 with a PWA grant of Federal aid. Actually the east portion, 5.5 miles of City Tunnel, was more urgently needed on account of its reinforcement of the distribution system. However the length of time required to construct it was too great to meet the PWA requirements. The only tunnel required in the west portion was but 3.00 miles long, 14 ft. diameter under the Sudbury Reservoir and high ground west of it, and the design could include four shafts closely spaced. This was satisfactory to the PWA authorities who agreed that a reasonable completion date could be set 16½ months from the order to the Contractor to begin work, and that it might be possible in an extreme emergency to make some use of the tunnel even before completion of the lining.

To provide for emergency unwatering, the design included, in

Shaft 4 at the east or Sudbury Dam end, a pump in a deep dry well at the bottom of the shaft similar to that for Quabbin Aqueduct described above and driven by the same 900 HP motor, which is stored at the top of the shaft and transported from one location to the other as needed.

Construction

Bids were opened December 15, 1938. The contract was executed December 30 and the work accepted as practically complete July 15, 1940, a 43-day extension of time having been granted on account of a fire which burned the plant buildings at Shaft 2, a jurisdictional strike, etc.

The shafts averaged 1.00 mile apart and it was possible to sink them all through sound ledge, from points varying from only 8 to 29 feet below the original ground surface. The work was entirely within the District's watersheds and hence the Contractor was required to settle all the drainage and chlorinate the effluent, and at the two easterly shafts to collect all plant sewage and empty it into the Marlborough sewers. Generally speaking good rock conditions prevailed. The tunnel is entirely in igneous rock, quartz diorite, quartzite and granite, and passes through several minor shear zones and faults most of which were sealed by mineralization so that a negligible amount of water was encountered. Only 4.9 per cent of the length required steel and timber support.

The contractor by repairing and replacing certain equipment at the District's Sudbury Dam power station could buy electric power from the Commission. The total so purchased was 3,954,200 KWH.

The shafts were lined with concrete as they were sunk, the same as for Quabbin Aqueduct and with the same 14-ft. diameter for the construction purposes. Shaft 4, containing the uptake waterway and the separate access well to the unwatering pump was not a typical construction shaft and is not included in the following table summarizing the progress made, nor in that of cost. The tunnel was driven by the full-face method with automatic drifting drills mounted on jumbos. An average of 35 holes, 10 to 12 feet deep were drilled in the heading and loaded with 40 per cent dynamite. Shovel-type mucking machines were used.

For the concrete lining operations the aggregates were purchased and delivered to two surface plants at Shafts 2 and 3. At each plant

PROGRESS

At Shaft No.	Average weekly progress shaft sinking in rock, ft.	Weekly progress tunnel excavation per heading, ft. Average	Maximum
1	17	94	123
2	18	117	177
3	17	121	165
4		97	129

an enclosed trap tunnel extended from the shaft head beneath a line of aggregate bins. In this trap tunnel scale hoppers discharged onto a belt conveyor emptying into a 10 in. pipe down the shaft. Cement was dropped down the shaft in a similar 10 in. pipe and the batches were made up at the bottom of the shaft and transported to the mixer at the forms from which the concrete was fed by a pistonless air gun into the top of the forms, the complete circular section being placed in one operation. 200 feet of collapsible forms were used with each mixer and 100 feet advanced each day when operating at the maximum speed. The average progress was 427 feet per week.

Grouting was carried out in two steps, first complete grouting

PERMANENT STEEL SUPPORT IN CONSTRUCTION OF HULTMAN AQUEDUCT, WEST HEADING FROM SHAFT 3.

of the entire tunnel under low pressure filling each connection to refusal under a maximum pressure of 50 lbs. per sq. in., and then a final grouting under high pressure up to a maximum of 350 lbs. Grouting reduced the leakage in the entire length of tunnel from a maximum of 740 gallons per minute to 28 gallons per minute.

During the period when all tunnel headings were being excavated the contractor's force averaged about 440 men and during the lining operations about 340, working three shifts.

The contractor was required to provide medical and hospital services. There were about 1025 accidents of all sorts, including 320 which resulted in lost time of 21,400 hours. There were no fatal accidents.

Cost

The summarized cost figures are the equivalent per-foot prices which the District actually paid the contractor for sinking in rock and lining the typical construction shafts, and for excavating and lining the pressure tunnel, all pressure grouted. In both shaft and tunnel the "A-line" distance or minimum to projecting points of rock was 8 inches, with 13 inches additional or 21 inches total pay-line thickness of lining.

The concrete mix varied with conditions but these costs are based on a uniform 1.5 bbls. of cement per cu. yd.

Total Costs per Foot Based on Bid Prices (Bid in 1938)

Construction Shafts in Rock 14 ft. diameter 8 in. "A-line"	Pressure Tunnel 14 ft. diameter 8 in. "A-line"
$295	$155
An additional cost, chargeable to the shaft, over and above the per-foot cost of the tunnel, of excavating with shaft equipment each heading adjacent to the shaft, for a length of about 42 feet, or within 50 feet of the shaft center line, was	
	$3,700

Hultman Aqueduct—City Tunnel—Design

In spite of the fact that the 5.5 mile City Tunnel was the most urgently needed portion of the Hultman Aqueduct, the war caught up with the work and finally necessitated its postponement after

Shaft 5 at the east end of the cut-and-cover section had been sunk with the help of a 98 ft. caisson alongside the Charles River and after the east and north tunnel headings had been excavated for 196 and 116 ft. respectively, all under a continuation of the Federal aid. Under the urgency of demands for its completion by both Army and Navy, the Commission cooperated with the Federal authorities, PWA, to the extent of making extensive surveys and plans for closing up the spacing between shafts so that construction could be carried on from a total of seven shafts. However, it was finally decided that in spite of the fact the need was multiplied by the war demands, there was no reasonable expectancy that the tunnel could be completed and put into service in time to benefit the war effort. In the meantime funds already depleted by the fruitless additional surveys became increasingly inadequate with the steady increase in construction costs. In 1946 the Legislature made the necessary additional appropriation. Bids would have been invited ordinarily for the entire tunnel job including the preliminary sinking of the necessary construction shafts. However it was decided to first contract for sinking Shaft 6 at about the halfway point in Newton and Shaft 7 near the Chestnut Hill end, and to postpone the asking of bids for the tunnel proper in the hope of securing firm bids with less of an allowance for future uncertainties than would be the case if bids were taken in 1946 for work that would have to be extended over a period of four years.

An interesting feature of the design of the City Tunnel is that it is a part of a comprehensive plan for the future, including a tunnel loop with uptake shafts to supply various centers of distribution, and to conserve sufficient head in supplying the Northern High Service so that it too, as well as the Southern High Service, can be supplied ultimately by gravity instead of by pumping from the Low Service Reservoir, Spot Pond. When the plans were first made over ten years ago it was expected that the tunnel loop would be extended much further to the east passing under Boston Common, Charlestown and Everett, but this more ambitious plan was rejected as financially impractical and there is no present intention of providing for more than a branch extending northeasterly from Chestnut Hill to a suitable connection with the Northern High Service, later possibly extending westerly from that point to complete the loop back to Shaft 5, also possibly a future dead end extension from Chestnut Hill to the southeast to still further reinforce the Southern High Service.

The tunnel is designed with all these extensions in mind. For example at the bottom of Shaft 5, which was completed as above stated in 1941, a branch heading driven 300 feet northeasterly in anticipation of the future closing of the loop, and a deep underground pumping station, will be completed under the present contract. This pumping station can unwater either the south or the north branch of the loop and pump the drainage into the Charles River. This possibility of two separate lines ultimately feeding the centers of consumption in the District is extended westerly by the provision of two separate waterway downtakes in Shaft 5 and of connections for a second grade-line aqueduct extending all the way from the new Norumbega Distributing Reservoir located on the Hultman Aqueduct, two miles west of Shaft 5. A similar deep underground pumping station will be located ultimately in Somerville at the northeast end of the loop and will be designed so that either of the two adjoining sections of the tunnel can be unwatered by pumping the drainage into the Mystic River.

Shaft 6, will contain an uptake to supply the City of Newton.

Shaft 7, the principal control point, is designed as a three-way shaft with all control valves accessible and embedded in the sound ledge near the surface, one shaft waterway being the uptake from the tunnel now being constructed from Shafts 5 and 6, another being the future downtake to a heading now being excavated in a northeasterly direction to provide for the future loop, and the other being a downtake to the short length of tunnel now being constructed to the uptake shaft, 7B, in the yard of the Chestnut Hill Pumping Station. A contract will later be let to connect control valves at the top of this uptake shaft to the four principal distribution force mains from the pumping station supplying the Southern High Service. There is also being constructed an extension of the tunnel a short distance beyond this uptake shaft 7B to provide for the possible future dead-end extension to the southeast to more adequately supply the Southern High Service. The present force mains out of the Chestnut Hill Pumping Station are also being tapped about ½ mile further west by a surface line directly from the main control center at the top of Shaft 7.

Under this arrangement it should be possible in the future to maintain service from Norumbega Distributing Reservoir in Weston to practically all points in the District, in spite of any accident re-

EXCAVATION OF HULTMAN AQUEDUCT, EAST HEADING FROM SHAFT 5.

quiring unwatering of any portion of the tunnel. However it would obviously be unwise to actually dismantle the Chestnut Hill pumps and abandon the Chestnut Hill Reservoir, from which they draw their supply, until such a tunnel loop is actually completed, even though the chance of anything happening to the first deep rock tunnel is extremely remote. In this connection it is interesting to note that Manhattan Island was supplied by New York's deep rock Tunnel No. 1 for 20 years before Tunnel No. 2, completing the distribution loop was put into service in 1936.

CONSTRUCTION

The District was fortunate in being able to locate Shaft 6 on a large vacant lot of the Newton Cemetery Corporation. This is one of the largest cemeteries in the Metropolitan area and in addition to its principal development from the main entrance on Walnut Street, it controls a large area fronting on Commonwealth Avenue which has never been developed for cemetery purposes on account of its low swampy condition. Consequently an agreement was made beneficial to both parties under which the District is given a temporary easement to locate its principal construction shaft on the edge of this swampy area, and to use the tunnel spoil to fill in the low

spots and create an attractive topography which will provide not only a future entrance to the cemetery from Commonwealth Avenue but will make possible a considerable extension for cemetery purposes. Under this agreement the tunnel contractor is required to see that no tunnel spoil containing larger than 4″ stones is deposited within 5 feet of the final surface, which he must grade to desired contours using soil stripped from the entire area before any tunnel spoil was deposited.

At Shaft 7 the depositing of tunnel spoil was made even easier by the fact that the District's Chestnut Hill Reservoir includes, in addition to the main Bradlee Basin, the smaller Lawrence Basin which could be cut off from water supply use during the construction period and which is of ample size so that one of the bays is used for the deposit of the spoil, moving the shore line out a distance of only about 300 feet in this bay.

One outstanding feature of the construction is that it is under a densely populated area. The tunnel passes not only under many residences but even directly under many important buildings, such as Boston College, the Brae Burn Country Club, etc. As a result many complaints have been and still are being received. There is a wide variation in the public's reaction varying from those who understand and appreciate the necessity for the work and whose reaction is one of interest in its progress to those who consider that the work is a calamity of the highest order. Only one court action has been brought to stop the work. On April 7, 1947, residents located across Commonwealth Avenue from Shaft 6 obtained a temporary restraining order requiring the shaft contractor to lessen the noise of the construction. The case was heard May 15 and shortly thereafter was dismissed.

The uptake shaft for connection to the force mains from the Chestnut Hill Pumping Station was originally located at the most convenient point for making the connection. There was one residence across the street only about 100 feet from the shaft site, another only about 130 feet, and several others nearby. The work at the surface in starting this shaft required the driving of a ring of steel sheet piling and was necessarily noisy, and suit to stop it was threatened. The Commission gave a hearing to the complainants and after careful consideration of the situation ordered the abandonment of that location in favor of the one in the pumping station yard, Shaft 7B.

This change involved an increase of 360 ft. in tunnel length and necessitated a horizontal angle of 28° in the tunnel line which was made by a 517 ft. radius.

The densely populated locale also introduced difficulties in establishing the tunnel line precisely, through residential Newton. To establish the line more accurately, or at least more readily, than could be done by offsets from a survey line along the highways, eight 92-foot steel Bilby towers were erected at convenient points on the tunnel line. Each tower provided an independent support for the instrument and an outer platform for the observer. The field engineers always take great pride in making the tunnel headings meet on the line. Some of the headings in Quabbin Aqueduct were several inches off line but many of them were about 1 inch off, and the headings in Hultman Aqueduct under Sudbury Reservoir averaged 1 inch off line. There is, of course, less difficulty so far as the grade is concerned, the headings meeting generally within a fraction of an inch. The first meeting of headings under Newton Center is expected to occur next March at the corner of Commonwealth Avenue and Chestnut Terrace, and we will then know the result obtained by the use of these Bilby towers.

The character of the locale also introduced difficulties in the storage of dynamite. In compliance with the rules of the Department of Public Safety the contractor can store 3,000 lbs. of 40 per cent and 60 per cent dynamite in the extensive open area at Shaft 6, and only 1,000 lbs. at Shaft 7. Both contractors use purchased power and to date, October 1, have consumed a total of about 2,613,800 KWH.

The distance between Shaft 5 and 6 is 2.26 miles, 12 ft. diameter; between Shaft 6 and 7, 2.53 miles, 12 ft. diameter; and between Shaft 7 and 7B, 0.65 mile, 10 ft. diameter. Excellent rock conditions have prevailed to date. The rock encountered in the westerly section is generally a sedimentary conglomerate, with beds or lenses of arkose, a type of sand-stone. However in the westerly heading from Shaft 6 the tunnel bore has penetrated about one-half mile of melaphyre, an intrusive igneous rock. There are also igneous dikes of basalt and aporhyolite. In the easterly section, Shaft 7, the rock is principally a sedimentary argillite. This is also encountered with increasing frequency in the east heading from Shaft 6.

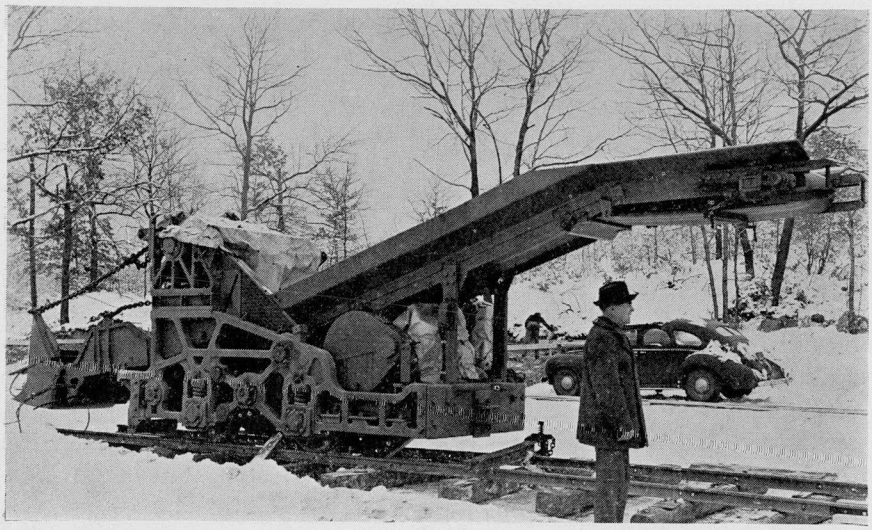
At Shaft 6 the depth of overburden that had to be excavated to

the surface of sound ledge was 24 ft. and ledge was excavated to a depth of 162 ft. by bucket and crane. At Shaft 7 the depth of overburden was only 8 ft. and ledge was excavated to a further depth of 112 ft. by bucket and crane. For excavation at the greater depths the shaft contractor was required to set up at each of the two shafts a headframe with an overhung bullwheel to guide the bucket. For removing the bottom 19 ft. of excavation at the large-diameter Shaft 7, the contractor experimented with a new type of equipment. He installed an air-operated shaft mucking machine having a 9 cu. ft. clam shell bucket with 3-way travel. The operation was satisfactory, but not of sufficient scope to effect any economy. In excavating the tunnel headings away from the shaft bottom, the shaft contractor used an electrically-operated dragline-type mucking machine.

The tunnel contractor removed the light headframes of the shaft contractor, one to Shaft 5 and one to the new Shaft 7B, and installed at each of the principal construction shafts, 6 and 7, a heavy headframe in the upper story of which the muck cars are emptied into hoppers lined for sound deadening. The tunnel contractor is using the full-face method of driving; and drilling, blasting and loading is alternated between headings. A drill jumbo with automatic drifters is used and the drilling time for an 8 ft. round is approximately two hours. The average use of dynamite varies from about 4 1/3 to 5¼ lbs per cu. yd. A shovel-type mucking machine is used to load 4 cu. yd. muck cars, for transportation to and up to the shaft. An added feature at the top of Shaft 6 is the passing of spoil through a 4 in. grizzly to separate the finer materials for use in the upper 5 feet of the spoil bank.

In both headings at Shaft 6 the contractor is loading the muck cars with equipment of his own design. Each unit contains a belt conveyor about 80′ long and 36″ wide, operating directly behind the mucker and supported 9′ above the top of the rail by a steel superstructure running on an independent track outside the regular muck rail. It is provided with a hopper at the receiving end into which the mucking machine discharges; and it permits a full train of 5 or 6 cars to pass under the belt so that a complete train may be loaded without coupling or uncoupling cars. This type of unit appears to be reducing the mucking time by at least 10 per cent.

The following table shows the progress made in sinking and lining the typical construction shaft and in excavating the tunnel headings for the pressure tunnel.



CONWAY MUCKING MACHINE PREPARED FOR LOWERING INTO THE TUNNEL EXCAVATION AT SHAFT 6, HULTMAN AQUEDUCT.

PROGRESS TO DATE (OCTOBER 1, 1948)

At Shaft No.	Average weekly progress shaft sinking in rock, ft.	Weekly progress tunnel excavation per heading, ft. Average in most recent period of 10 weeks	6 weeks	Recent Maximum
6	14	164	170	210
7 (12 ft. tunnel heading)		151	160	191
7 (10 ft. tunnel heading)		174	182	228

At Shaft 7B, the uptake shaft which is now being sunk in the pumping station yard, 47 ft. depth of overburden had first to be excavated using the ring of steel piling which was moved over from the abandoned location at 7A. At Shaft 5, the contractor is preparing to unwater the excavations on which work was suspended in 1941.

The contractor is required to provide medical and hospital services. To date, October 1, there have been on both contracts 232 accidents of all sorts, including 68 which resulted in the loss of 1,524 hours. There has been one fatal accident. The Contractor's force

now averages about 430 men working on 3 shifts in the tunnel and 40 men on 2 shifts at Shaft 7B.

Cost

The summarized cost figures are the equivalent per-foot prices which the District actually paid the contractor for sinking in rock and lining the typical construction Shaft 6 and the larger Shaft 7, and which it must pay the present contractor for excavating and lining the pressure tunnel. The figures for the tunnel, which is still under construction, are based on experience to date so far as dryness of the excavation is concerned. A total of not more than 150,000 million foot-gallons of pumpage is indicated. Steel support has been stocked in readiness for use but only 16 ft. of the total 2.3 miles of tunnel excavated to date, October 1, has required support. From the figures given, the cost for any length of support can be interpolated. Although somewhat more than 1.5 bbls. of cement per cu. yd. of concrete was used in lining the construction shafts, the figures as given for the tunnel anticipate the use of 1.5 bbls., the same as in the case of the earlier tunnels.

Total Costs per Foot Based on Bid Prices for Both the Shaft Contract and the Tunnel Contract

Shafts in Rock (Bid in 1946)				Pressure Tunnel (Bid in 1947)			
				10 ft. diameter		12 ft. diameter	
Construction Shaft		Special Shaft		8 in. "A-line"		8 in "A-line"	
14 ft. diameter		20 ft. 2 in. diameter			Support		Support
8 in. "A-line"		8 in "A-line"		No	10% of	No	10% of
Ungrouted	Grouted	Ungrouted	Grouted	Support	length	Support	length
$446	$460	$683	$705	$204	$214	$242	$253

An additional cost, chargeable to the shaft, over and above the per-foot cost of the tunnel, of excavating with shaft equipment each heading adjacent to the shaft, say for a length of 200 feet under typical normal conditions was more readily determined in this case of separate bids for shaft and tunnel, and was

$9,400 $12,200

Organization

The Commissioner of the Metropolitan District Commission is William T. Morrissey, and the Associate Commissioners, William F. Rogers, Max Ulin, Joseph McKenney and John J. Grigalus. The

work is being done by the Construction Division of the Commission, of which the writer is Chief Engineer. Stanley M. Dore, Mem. ASCE, is Deputy Chief Engineer, Lawrence M. Gentleman, Mem. is Designing Engineer, and Frederick W. Gow, Assoc. M., is Head Construction Engineer in charge of the actual construction in the field, not only of the present tunnel but also of the first pressure tunnel under the Sudbury Reservoir, and has furnished valuable assistance in the preparation of this paper. The Construction Division is a successor to the former Metropolitan District Water Supply Commission which was abolished June 30, 1947. The first tunnel construction, of Quabbin Aqueduct, was initiated 22 years ago when the late Frank E. Winsor, a Vice President of the Society, was Chief Engineer of that Commission. Many will recall the dedication of the Winsor Memorial which overlooks Winsor Dam of Quabbin Reservoir, and which was sponsored jointly by the New England Section of the American Society of Civil Engineers and by the Boston Society of Civil Engineers.

HEADACHES FROM TREATMENT OF COMBINED INDUSTRIAL WASTES AND SEWAGE

By Stuart E. Coburn, Member*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on October 6, 1948.)

In the 19th century, when communities discharged their household wastes directly to the nearest watercourse, and when industry was located directly on a stream to obtain a plentiful supply of process water, "headaches" from treatment of combined industrial wastes and sewage were unknown.

As public water supplies were installed, new industries located away from the stream and discharged their wastes into the public sewerage system.

With the inception of sewage treatment the presence of these wastes offered no serious difficulty as the mills were small, or treatment consisted of sedimentation only. However, with the enormous development of industry in this country, together with advancement in sewage treatment by biological and mechanical processes these wastes which had been unobjectionable, now became a serious problem in sewage treatment.

An examination of the literature of the past few years reveals a very large number of articles describing the objectionable effects of various industrial wastes on sewage treatment. The factual evidence from these report shows that these wastes caused various troubles from "excessive foaming" or "retarding digestion of sludge" to a complete breakdown of treatment plant facilities.

Sewage Treatment Plants:

The extent to which sewage must be treated in any locality, depends upon the relative volume of the receiving waters, and the subsequent uses of the water. The completeness of treatment required is usually specified by the State Boards of Health, Stream Authorities, or other regulatory bodies.

Sewage treatment works include grit and screenings removal

*Chief Chemist, Metcalf & Eddy, Engineers, Boston, Mass.

facilities, sedimentation tanks, sludge digestion tanks and means of sludge disposal, chemical treatment, biological treatment and chlorination. It is not necessary to describe here the functioning of each of these processes.

Industries Discharging Liquid Wastes:

The list of industries discharging liquid wastes is long and varied, and this list is still increasing with new industries and different types of wastes being added from time to time.

In New England the following industries are among the principal ones discharging wastes sufficient in strength and volume to constitute a problem, but varying in degree:

Textile, including cotton, wool, silk and man-made fibres
Metallurgical and metal working
Tannery
Plating
Brewing and distilling
Paper and pulp
Cannery
Rubber
Chemical

In other parts of the country other industries such as pharmaceutical, dairy, meat-packing, gas and coke plants, and oil refineries are perhaps more important than in New England.

All of the above industries discharge wastes containing widely different constituents of varying strength and volume. Even in the same industry such as tanning, the wastes are quite dissimilar depending on whether a vegetable or mineral tan is used. Moreover the wastes from the same plant, such as a wool scouring plant will differ widely from day to day depending on the type of wool being scoured, the shrinkage, relative grease content, the detergents used and availability of process water.

Effects on Sewage Treatment:

If these wastes are discharged into a sewerage system, their effect will depend to a major degree on the relative volumes of sewage and wastes or more specifically on the actual strength of a particular constituent in the mixture as received at the treatment plant.

It is obvious that to describe the known effects of all the constituents of all wastes on the various units used in sewage treatment, would be impossible in a single paper, and frankly there is still much to be learned about this subject.

One method of approach to the subject of effects of industrial wastes on sewage treatment, would be to describe these effects in general on the physical, chemical, or biological operations of sewage treatment. Aside from mechanical troubles, upsetting any one of these reactions in a well designed or operated sewage treatment plant is the chief cause for "headaches" by the presence of industrial wastes.

A sewage plant can be affected physically by high temperatures or excess coloration from dye wastes that cannot be removed by secondary treatment. Soluble oils used in metal working establishments cannot be removed by settling, and in primary treatment would pass through the system to cause high turbidity in a stream. Excess solids and grease may overload primary treatment and sludge digestion units. Hair and wool from tanneries or textile plants, latex from rubber plants, and certain constituents in the manufacture of synthetic resins cause clogging of sewers and screens.

Gasoline and other explosives or inflammable substances are hazardous and should not be admitted to sewerage systems. Mineral acids cause disintegration of sewer structures and corrosion of metal appurtenances and should be excluded.

Excessive quantities of dissolved organic solids that overload trickling filters or activated sludge systems, or toxic substances which seriously retard or stop biological life are probably the chief causes for headaches in an operator's life. These objectionable substances may come from a large number of industries or may be contributed in large part by a single industry. As an illustration, in the manufacture of mouton fur, the following substances may cause trouble in treatment.

1. Sulfur dyes which may not be removed by treatment.
2. Grease and excessive alkali from the scouring of skins, which clog sewers and also retard biological treatment.
3. Heavy suspended solids from washing and tanning which overload primary treatment or sludge digestion units.
4. Toxic chromium from mineral tan which prevents digestion of sludge.

5. Large quantities of organic material having excessively high oxygen demand which overload oxidation processes.

Effect of Toxic Metals:

Metals and wastes discharged from metallurgical and metal working plants have had deleterious effects on sewage treatment processes. Iron, copper, chromium, zinc and cyanides are serious offenders. Other metallic ions may be contributing factors.

Acid-iron wastes have caused trouble at Cleveland, Fostoria, Ohio, and Worcester, Mass. The troubles have been overloading of sludge facilities, clogging of trickling filters, clogging of air diffusers, and retardation of oxidation.

Copper salts may be precipitated in alkaline sewage and removed with the sludge to digestors. It is known that a small quantity of copper is quite effective in killing biological life and digestion is retarded or stopped. Copper in quantities as low as one ppm., in the sewage has been reported by Mohlman as having caused serious difficulties in Chicago. In this connection it may be important to note that the concentration of copper in the sludge may be many times that in the sewage from which it is precipitated. Wise found 200 ppm of copper in sludge obtained from sewage containing only 0.5 ppm of copper and made similar observations in regard to chromium.

Zinc is reported to be a cause of retarding sludge digestion at Fostoria, Elyria, and London, Ohio.

Chromium salts, usually reported in the form of metallic chromium have different effects on different units of a sewage treatment plant. The chromium is present in the trivalent form in spent chrome alum in the wastes discharged from a tannery using a mineral tan. Chromium in this form will precipitate almost quantitatively in an alkaline solution, such as sewage, and be removed as sludge to digestion tanks, and digestion will be retarded or practically stopped. If secondary treatment is used there will probably be very little trouble biologically. On the other hand, if the chromium compounds are discharged from a plating establishment or from anodyzing of aluminum, the chromium is chiefly in the hexavalent form. Chromium cannot be precipitated by alkaline sewage in this form. Therefore, the effect on sludge digestion is very much less. However, in the hexavalent form the chromium is much more toxic, and when present

in sufficient quantities will kill the organisms in filters or in activated sludge. The quantity objectionable is controversial. It has been reported that quantities up to 25 ppm. can be handled continuously. Edwards found that shock loads of about 40 parts per million slowed up nitrification and caused deterioration in the appearance of activated sludge effluent. Information is still lacking as to the amount of chromium which is permissible in sewage under the following variable conditions.

1. Type of chromium, trivalent or hexavalent;
2. Type of sewage treatment, such as primary or secondary;
3. Continuous flow versus shock loads over a period of a few hours.

Cyanides of 5 parts per million, discharged continuously have an inhibiting effect on activated sludge treatment. Doses of 20 ppm. are reported as having an immediate effect on activated sludge, but after removal of the cyanide, the sludge recovers. Siebert reports that as low as 1 ppm. is toxic to biological activities. On the other hand, filtration of much higher concentrations of cyanide, through sand or coke breeze, has been suggested as a method of treatment.

From another point of view cyanides should not be permitted in sewage because of the possibility of acid wastes being present at the same time with resultant possible liberation of hydrocyanic acid gas.

Wastes from garbage grinders in restaurants and hotels as well as from households are gradually increasing in quantity. It is too early to say how in the future these wastes will influence new sewage treatment plant design.

Increasing use of synthetic detergents in households and industry will change the characteristics of sewage. The soaps, grease and fats now present in sewage are precipitated in primary treatment by the calcium and magnesium salts present in the sewage and are removed as sludge to the digestion tanks. Synthetic detergents are not precipitated but pass on to the secondary treatment units increasing the organic load on these units with a resulting increase in the quantity of secondary sludge.

A large number of synthetic detergents are ionic and can be coagulated by the chemicals used in chemical treatment of sewage and therefore do not affect chemical precipitation.

Concentration Limits of Deleterious Wastes:

In order to avoid a breakdown in sewage plant operation, it is obvious that the concentration of objectionable constituents in the sewage as received at the treatment plant must not exceed certain limits. Unfortunately, data are meager as to the quantities of toxic materials present in sewage which have caused excessive trouble. It is true there is much evidence in the literature about the causative agent, but little evidence whether the quantity reported represented a shock load or a constant load over a considerable period of time. Much more evidence is necessary.

Limits have been proposed by the Allegheny County Sanitary Authority for the amounts of certain toxic materials in daily composite samples of sewage. The limits are given in the following table:

CONCENTRATION LIMITS FOR TOXIC AND DELETERIOUS SUBSTANCES IN COMPOSITE SAMPLES OF TREATMENT PLANT INFLUENT*

Names of items	*Primary treatment* No sludge digestion	*Primary treatment* Sludge digestion	*Secondary treatment*
	Limits—parts per million, except pH		
pH index (not less than)	5.0	6.5	6.5
Total iron	5.0	5.0	5.0
Copper	3.0	1.0	1.0
Chromium	5.0	5.0	3.0
Cyanide	2.0	2.0	2.0
Oils—mineral	Should be excluded by municipal ordinances.		
Oils—soluble	Removed to maximum practicable extent before discharge.		
Combustibles, etc.	Should be excluded by municipal ordinances.		
Free mineral acids	Should be neutralized at source.		
Acetelene generation sludge	Should be excluded by municipal ordinances.		

*(From "Report—Proposed Collection and Treatment of Municipal Sewage and Industrial Wastes", Allegheny County Sanitary Authority, January 1948.)

The writer believes that if these concentration limits are not exceeded there will be no trouble in treating the sewage by the standard sewage treatment processes. It is also believed that these limits can be exceeded manifold for short periods of time without other than slight temporary upsets in sewage treatment.

Remedies for Excessive Concentration of Industrial Wastes at Existing Sewage Plants:

Sewage treatment plants located in large municipalities may not be seriously affected by industrial wastes, due to the dilution factor. In small communities the industrial wastes may be a serious load on the treatment plant. For example, in Jeannette, Pa., the wastes from a single brewery contribute to the sewage plant a greater population equivalent load as measured by both suspended solids and biochemical oxygen demand than the remainder of the community.

If certain constituents in the industrial wastes of a community prevent the satisfactory operation of a sewage treatment plant, corrective measures are similar irrespective of the size of the community. These may be to

1. Remove the offending wastes at the source and require the industry to treat them separately before discharge to the stream, or sewer.
2. Require partial treatment at the source to such a degree that the sewage plant facilities can handle them satisfactorily.
3. Accept the wastes and reconstruct or enlarge the plant to care for them.

In the case of proposed new sewage treatment plants, if it is the wish of the community or its representatives to accept the objectionable industrial wastes of the municipality, the plant should be designed accordingly and capital and operating costs assessed on the industry on an equitable basis.

Examples of Sewage Treatment Plans Where Serious "Headaches" Have Occurred Due to Industrial Wastes

LOCATION	TYPE OF WASTES
Fostoria, Ohio	Acid-iron, zinc
Worcester, Mass.	Acid-iron
Jeannette, Pa.	Brewery
Danbury, Conn.	Hatters waste
Bristol, Rhode Island	Wool scouring
Clinton, Mass.	Distillery
La Porte, Indiana	Wool scouring wastes
Akron, Ohio	Rubber
Gloversville, N. Y.	Tannery

Methods of Reducing Objectionable Constituents at Source

OBJECTIONABLE CONSTITUENTS	REMEDY
Temperature	Heat exchangers
Shock loads	Equalizing tanks (for both volume and strength of wastes)
Acids and alkalies	Neutralization
Zinc	Precipitation
Copper	Flash plating or precipitation by chemicals
Chromium	Precipitation
Cyanides	Acidification, lime-sulfur treatment, chlorination, ponding
Soluble oils	Acid cracking, addition of alum or calcium chloride
Greases	Grease traps

Much more work remains to be done in obtaining the needed facts on concentration limits of objectionable constituents of industrial wastes and the effect of these constituents on sewage treatment processes.

Discussion

By Joseph A. McCarthy*

The main thought of Mr. Coburn's fine paper might be summarized this way: sewage treatment plants don't like variety. And in almost every condition he has discussed, except for complete sterilization, there has been an upsetting of the natural or normal balance, from the standpoint of the hard working bacteria. We have been working for a long time at Lawrence on sewage made abnormal by the addition of industrial wastes, and one of the measuring sticks which we and a lot of other people have used is the carbon:nitrogen ratio. When first used, it was oxygen consumed vs. albuminoid ammonia; now I prefer B.O.D. vs. Kjeldahl nitrogen, but although there may be a little difference in numerical value, the principle remains the same. Many of the disastrous effects described in the paper came from upsetting some such ratio in a treatment plant. Sometimes there seems to be nothing much which can be done about curing nitrogen deficiency, but if this is the only headache in sight in a given problem, the addition of available nitrogen, even in a rather large volume of sewage-waste mixture, may be well worth while. Frequently we have found that nitrogen deficiency is accompanied by a

*Chief of Laboratories, Lawrence Experiment Station, Massachusetts State Dept. of Public Health.

lack of other materials necessary for vigorous bacterial growth, particularly phosphorus; and we have several times used ammonium phosphate in sewage-waste mixtures with highly satisfactory results. Incidentally if any one wishes to seed a filter quickly and successfully, try adding this phosphate to your seeding mixture or liquid. If this isn't fast enough, try adding a little peptone too. In one case where nitrogen, for a change, was too plentiful, we restored the balance by adding skimmed milk, and got a good growth.

One headache mentioned by Mr. Coburn was interference with bacterial growth. We have learned that whenever we start work on the effect of a waste on a sewage, that extensive bacterial studies are worth while. If the waste in the expected proportion completely sterilizes the sewage, the problem is going to be difficult, to say the least. We have learned, and learned the hard way, that we must study mixtures of the sewage and waste in the worst possible concentration, and we must carry on our observations over rather extended periods, because we have found that some materials may have a very slow but very sure killing effect. One waste we worked on contained a little phenol, quite a lot of formaldehyde, a selection of terpene oils, and a small amount of sodium formate which we rather disregarded. After experimenting a little, we found the limiting concentration which did not sterilize the sewage after 18 hours contact; we started a filter with this mixture as indicated, got pretty good growth, and began to wait for a good ripening. Suddenly our good top growth died, and we had to start over, after checking our bacterial tryout. When this happened a second time, we went back to the beginning, tried our ingredients individually, for longer periods, and were surprised to find that the killer was the sodium formate, which waited 24 to 48 hours before doing the deed. This salt was more fatal than the formaldehyde or the phenol, apparently working by cumulative effect. This is just an example to show peculiar actions on bacteria, the value of really studying the bacterial side of a waste problem, and also the unpredictibility of things we thought we knew all about.

Mr. Coburn has spoken about the possibility that the concentration limits of certain toxic substances can be exceeded for short periods of time without more than slight temporary upsets. I think I can agree with him if the excess is small, the period is short, and the medium of treatment is a well established trickling filter. We have found that a sand or trickling filter, or to a lesser extent, an

activated sludge plant, can be trained by slow and careful additions to tolerate fairly heavy loads of materials normally toxic. We, for instance, trained a trickling filter to operate with success with 50± ppm. of phenol or cresol, and somewhat more of cyanides; and a good sand filter was taught to do at least as well. Years ago we trained both types of filters to tolerate 1.5% of gas house wastes, which contained all kinds of bactericides. However, once the maximum concentrations were established, they could not be changed to any considerable degree; if the toxic materials were withheld entirely, our poor dope fiend bacteria sulked, and filter performance suffered. These experiences have led me to believe that filters can take as steady diet doses which as shock loads might be disastrous. If this sounds contradictory to what I just said about cumulative effects, I can't explain; it just works that way.

Our new tools, high rate trickling filters, especially those recirculated with high ratios, apparently are not affected by bactericidal substances in the same way as the old slow rate units. This may be the answer to the apparently complete kill, Bruce Dickerson reported at the New England Sewage Works Association at Newport, R. I., on the formaldehyde filter now in operation at Mansfield. We worked with him on the bacterial side of this project, and found as we expected, that 1000 ppm. of formaldehyde, and indeed a lot less, killed all growth in a one-pass filter. But when the effluent was recirculated with a very high ratio, the contact time was so short that apparently only old bacteria died, the newborn health specimens flourished in a perpetual logarithmic growth, and to them formaldehyde was just another carbonaceous substance to devour. Incidentally the new filters were nourished to maturity on a medium containing all the ingredients of standard B.O.D. dilution water, and still get nitrogen from ammonium phosphate.

But to get back to headaches, instead of bragging: one we haven't cured is,—the effect of detergents and wetting substances in general. These may, as Mr. Coburn says, be harmless in themselves, but when combined with seven other devils, they may cause most severe migraines; witness,—cutting oils coming out of the same drain as finely divided carbon; result,—a permanent ink which on arrival at the treatment plant neatly plated every particle of sludge in the digester, —and the digester died. Some of our splitting headaches have come from wastes high in peroxides or chlorine mixing into sewage, giving

in one case a temporary negative B.O.D. Result: a double effect, death of most bacteria, and food apparently made unavailable for the remaining bugs. More wet towels,—some of the newer oxidation-reduction dyeing methods, especially on rayon. No color at pH 7 pink at 5, blue green at 9; all these pH values may be found in a treatment plant. No color from these dyes in the sewage when the oxygen was absent leaving the primary treatment plant; but downstream, after reaeration, the color came back. Then there is grease, apparently satisfactorily removed or tied down by primary treatment; but when the effluent reached salt water, calcium soap balls appeared, grew larger, floated ashore, and smelled. Perhaps the most impossible,—this headache needs treatment three ways: fish scales. The least bit of drying makes them stick—right where they land at that moment—forever.

Discussion

By Clair N. Sawyer*

I should like to compliment Mr. Coburn on his ability to concentrate a discussion of the subject under consideration this evening into such a short discourse. Those of us in the teaching profession find it difficult to limit ourselves to simple bald statements. We seem to be of the feeling that two or three rewordings are necessary to enable us to impress the fellows in the back row.

However, tonight I am sure the back rows are filled with just as interested and intelligent listeners as the front rows. Therefore, it is not my purpose to reiterate the points made by Mr. Coburn in order to add emphasis here and there but, rather, to single out the lessons to be learned from the past and present "headaches" so that some potential "headaches" of the future may be avoided. The present emphasis on stream pollution abatement which is hitting some industries squarely between the eyes for the first time is bound to bring increased demands for disposal and treatment of such wastes in municipal facilities with attendant troubles, if care and wisdom are not exercised. It is, therefore, the responsibility of those in public trust to see that the interests of the little taxpayer as well as the large are handled in an equitable manner.

*Associate Professor of Sanitary Chemistry, Massachusetts Institute of Technology, Cambridge, Mass.

Some of the lessons to be learned from the past are as follows:

1. *The need for adequate ordinances governing the disposal of toxic and other deleterious materials.*

The requirements will, naturally, vary depending upon the type of treatment provided. For primary treatment plants with sludge filtration and incineration, such as Minneapolis - St. Paul, the requirements would not need to be as strict as for similar plants with separate sludge digestion as planned at Nut Island, and the requirements for the latter would be much less rigid than if the wastes were to receive complete treatment by biological means such as at Leominster or Fitchburg.

The literature is replete with reports on difficulties caused across the country by various toxic and deleterious wastes. One of the best, or possibly poorest, examples, is the case at Worcester where, in spite of frequent warnings from the plant superintendent regarding the effects of excess steel mill pickle liquor, the higher officials did not take heed. You all know the consequences and the expense the taxpayers of Worcester have undergone to replace their secondary treatment facilities. You are, probably, also familiar with the fact that the new plant was planned and well into the process of construction before responsible officials began to realize that what had happened to the old plant could also happen to the new plant if proper precautions are not taken. The city of Worcester is at the present time in the throes of drafting suitable ordinances to protect their investment at the sewage plant.

Is it not time that some suggested standards or ordinances be set forth by some of the responsible technical organizations to cover various situations as a guide for the smaller cities of the country?

2. *The need for adequate sewer rental laws.*

There are numerous advantages which accrue from the application of an equitable sewer rental law. One of the most evident is that local residents know they are not being charged with bearing certain overhead charges in the production of industrial products which should be figured in as part of the cost of production. Secondly, the industry whose wastes are gaged and sampled has a constant record of its losses and, if some one is capable of translating such losses into dollars and cents, the management will see that such losses are reduced, especially if such losses can be shown to be excessive

with respect to its competitors. Thirdly, sewer rental laws based on volume concentration factors tend to reduce water consumption and, therefore, wastes volume. This aids in two ways by conserving an ever diminishing water supply and reducing the volume of wastes to be handled.

I should like to caution against simple ordinances which require that wastes be reduced in strength to compare to domestic sewage in the community. I know of one industry which circumvented such an ordinance by installing another well and by simple dilution reduced the strength of its waste so as to comply with the local ordinance. Proper flow measurements would have caught this situation with ease.

3. *The need for controlled discharge and adequate mixing of certain industrial wastes.*

The shock loads referred to by Mr. Coburn are due to the fact that most of our industries operate on a batch basis or at least dispose of their waste in a batch manner. This factor is especially serious in small communities where distance of travel in the sewers is short and dilution is limited.

The solution to the problem, of course, is the incorporation of storage, surge or retention tanks with orifices or similar devices to allow discharge in controlled quantity over a major part of the day and night.

4. *The need for a study of nutritional factors when certain industrial wastes are involved.*

Sugar is considered by most of us as a good food. But how would you like to try and live on sugar alone? Of course, it cannot be done. Our bodies require many other elements in addition to the carbon, hydrogen and oxygen contained in sugar. Similarly the bacteria and other forms of life responsible for stabilization of organic wastes require other elements, especially nitrogen and phosphorus to survive, reproduce and do the purifying job we expect of them.

Therefore it is reasonable to conclude that we must "open the mouths" of these industrial wastes and have a look inside to see whether they are good bacterial food. Recent studies have demonstrated that brewery wastes are deficient in nitrogen and sometimes phosphorus. Would it not be logical procedure to investigate some of our brewery waste problems in the smaller communities from this angle first before condemning the present treatment facilities, closing

down the brewery, or building bigger monuments of concrete and steel to handle unbalanced wastes at half speed purification rates?

5. *The need for study of existing sewage treatment facilities to determine their shortcomings.*

Are all of the headaches related to the wastes entering the treatment plant or are some of them a result of faulty design or inadequate equipment?

It would be safe to say at least a few are due to the latter. How many more is a matter of conjecture. A personal experience will serve to illustrate the case.

On an investigation into the cause of rising activated sludge in final clarifiers at a midwestern plant, it was found that the sludge removal mechanisms in the circular final clarifiers were utterly incapable of removing the desired quantities of sludge. Settled activated sludge remained in the clarifiers from 5 to 8 hours with consequent deterioration. Needless, to say the mechanisms were replaced with others of greater ability to remove sludge, at a cost of about 50 cents per capita.

6. *The need for industrial waste surveys to determine abnormal losses.*

The writer does not feel that the Chamber of Commerce should determine policies for the Sewerage Commission. There are such things as excessive losses from industrial establishments and those who have conducted waste surveys are well aware of this fact. As far as industrial management is concerned they are usually unaware of losses until the Sanitary Engineer "sewer rat" determines such losses and translates them into dollars and cents.

Numerous investigations throughout the country have established reasonable losses for most industries. For instance in the paper industry, losses in excess of one pound of fibre per ton of paper produced are considered excessive. In the meat packing industry losses in excess of 1.0 lb. of grease, 3.5 lb. of nitrogen, and 28 lb. of B.O.D. per ton of kill are considered indicative of poor housekeeping and management.

With such standards being established for many industries, we should all attempt to bring wayward plants into line by comparison and by use of the dollar and cents sign where practical.

DESIGN FEATURES OF CLARK HILL DAM AND POWER PLANT

By Robert T. Colburn, Member*

(Presented at a meeting of the Hydraulics Section of the Boston Society of Civil Engineers, held on November 3, 1948.)

The Clark Hill Project is being constructed by the Corps of Engineers on the Savannah River twenty-two miles above Augusta, Georgia. The project lies half in Georgia and half in South Carolina, as the state boundary line is the center of the Savannah River. It is essentially a hydroelectric power project, with some flood storage, and some navigation benefits on the lower Savannah River, which classify it as a multi-purpose project. The ultimate development will be seven 40,000 kilowatt units, and it is probable that the entire powerhouse structure will be built with the initial construction, although originally a structure to house 5 units was contemplated for the initial development. The annual output is estimated to average 703,000,000 kwh for the ultimate development. The cost of the project is estimated to be about 50 million dollars.

Congress appropriated money to start work in 1945-46 under the direction of the Corps of Engineers. A contract for the dam was let late in 1947, and the first concrete was poured on October 27, 1948. Separate contracts were let several months earlier for the access railroad, first stage cofferdam and excavation, part of the embankment rolled fill, and the temporary construction bridge. The power units and powerhouse are expected to be contracted for in time to be available when the dam is completed.

Clark Hill Dam consists of a center concrete section 2282 feet long with an earth embankment at each end, making a total length of 5680 feet as shown in Figure 1. The maximum height is 200 feet from rock foundation to roadway. The spillway section is 1096 feet long and located across the channel of the river. The powerhouse is on the South Carolina side or left bank, which is the most economical location for construction, due to tailrace and draft tube excavation conditions. A 24 foot roadway crosses the dam connecting

*Civil Engineer, Chas. T. Main, Inc.

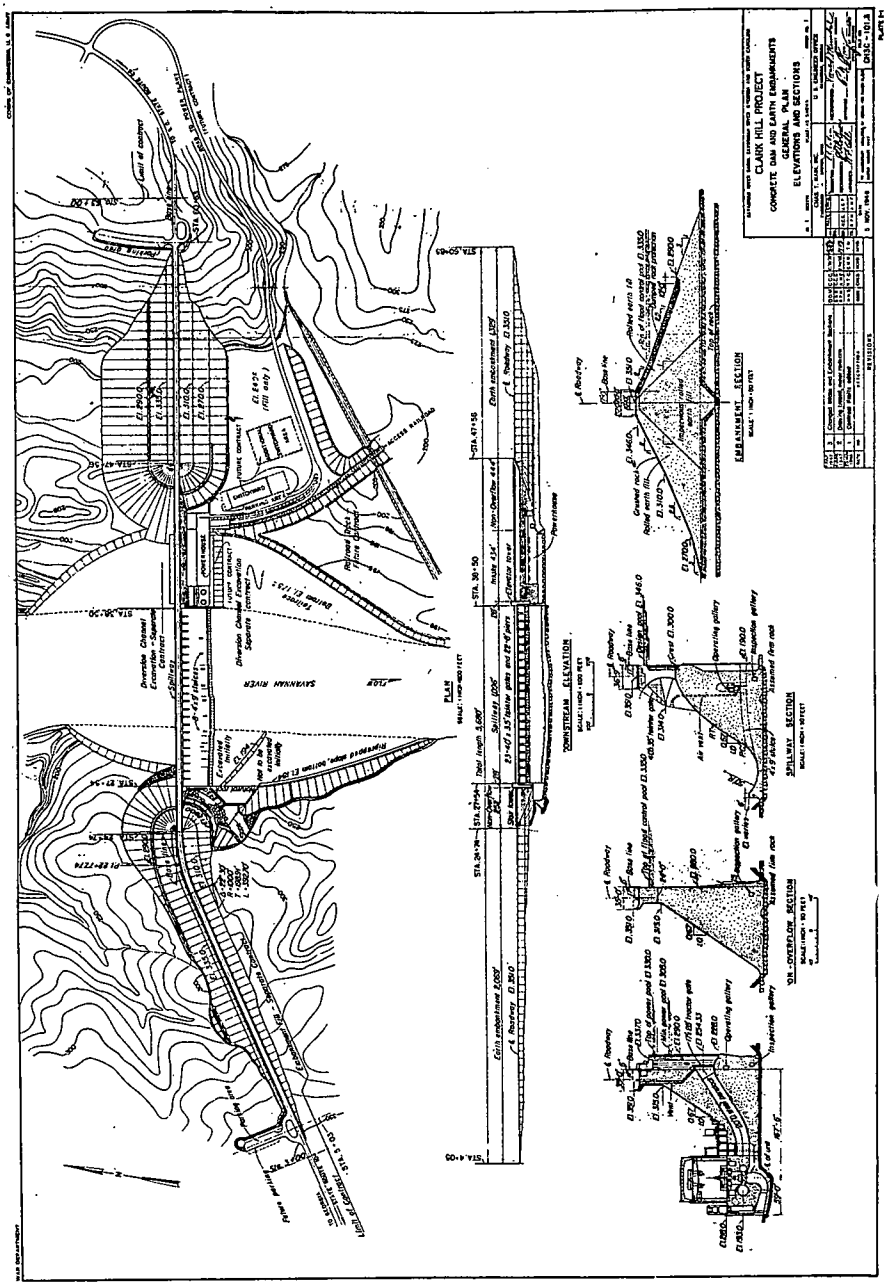
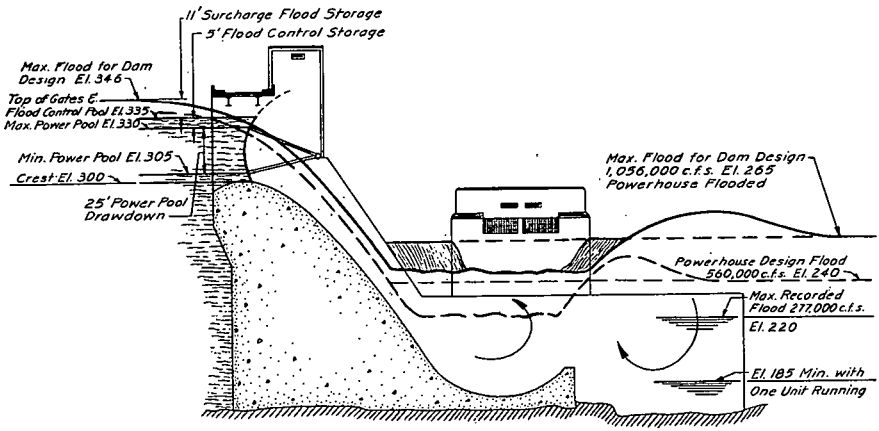


FIG. 1.—GENERAL PLAN AND SECTIONS.

Georgia Highway 104 with South Carolina Highway 28. The foundation of the dam is exceptionally good, consisting of granite and granite gneiss formation. There are very few surface cracks, and few seams in the rock. The overburden on the abutments is very satisfactory for earth dam construction.

The spillway crest is at Elevation 300 ft. above mean sea level, and the top of the spillway gates is at Elevation 335 as shown in Figure 2. The power pool has a maximum elevation of 330 and mini-



HEAD AND TAILWATER ELEVATIONS
FIGURE 2

mum of 305 providing a maximum drawdown of 25 feet. The upper five feet between the maximum power pool and the top of gates is used for flood control. The maximum surcharge which would occur during the design flood would bring the reservoir eleven feet above the top of the gates to Elevation 346. The reservoir storage capacity is as follows:

Power	1,340,000 acre feet
Flood Control Storage	390,000 acre feet
Surcharge above gates	950,000 acre feet

The full pool length of the reservoir at Elevation 335 is 37 miles, and the area at this elevation is 123 square miles.

The top of the flash boards on the Stevens Creek Dam about 12.6 miles downstream is 187. With one unit running and no flow over the spillway the tailwater elevation at Clark Hill will be 188.

If the Stevens Creek flash boards are down then the tailwater with one unit running will be 185. The tailwater elevation under conditions of the maximum design flood will be 265 and for the maximum flood of record is 220.

The drainage area at the site is 6144 square miles. The average flow in the river is 10,810 cu. ft. per second, and the greatest flood of record was 277,000 cu. ft. per second in September, 1929. The second greatest flood was in August, 1940, and equalled 203,000 cu. ft. per second. The design flood which determined the spillway capacity, as fixed by the Corps of Engineers, corresponds to a total rainfall of 21.8 inches, with a total runoff of 14.5 inches, over a period of 7 days. Under these conditions the peak rate of discharge would amount to 1,055,000 cu. ft. per second, which is almost four times the flood of record.

Although the spillway is designed for a maximum flood of 1,055,000 cfs, the powerhouse is designed for only 562,000 cfs without flooding. This corresponds to tailwater at Elevation 240 which is 20 feet higher than the flood of record. If the water should rise higher than this, the power plant will be flooded. It was impractical and would be too costly to design the powerhouse for operation under maximum flood conditions. This situation imposed problems in design of the power plant which do not ordinarily occur. The end of the ultimate power plant is adjacent to the spillway bucket. Model tests showed that under conditions of maximum flood the water opposite the end of the power plant is depressed by the action of the water passing over the spillway and down into the bucket. With the powerhouse flooded and tailwater at Elevation 265, this has a tendency to burst the end of the building outward into the spillway, and this factor had to be considered in the powerhouse design. Even on the temporary end of the originally planned initial powerhouse, which is 125 feet from the spillway, this condition exists to some extent although the difference in water levels would not be as great.

The spillway consists of 23 tainter gates 40 feet wide and 35 feet high. These radial gates are hinged on trunnions which are anchored into the piers between gates. The spillway section is a concrete gravity type with ogee spillway face. The design head on the crest is 46 feet which will occur during the maximum flood. The curve of the spillway upstream from the crest is made by the arcs of two circles, one of 23 foot radius and one of 9 foot radius. The shape of the spillway

downstream from the crest, as established by the Corps of Engineers, is an exponential curve conforming to the following equation:

$$y = \frac{x^{1.85}}{51.81}$$

From the stability analysis, it was determined that the downstream face should be extended downward on a tangent with a slope of 6.7 horizontal to 10 vertical from the point where the curve becomes tangent to that slope.

The concrete on the spillway crest and in the bucket will be surfaced by the vacuum process to obtain a smooth, durable concrete face. Absorbent form lining will be used on all other surfaces. A face mix of concrete from 4 to 8 feet thick is made to provide a denser concrete at the surface.

The 23 spillway gates are operated by individual hoists mounted on the concrete piers between the gates. Access to the hoists and controls is by a continuous walkway underneath the highway bridge at the hoist machinery level just above the gates. A five horsepower motor located on each pier is arranged to operate either one of the two adjacent gates. The gates are raised by two chains connected to the bottom of the gates, one at each end. The raising speed is one foot per minute. With the gate in closed position or at any point between closed position and a point above the upper nappe for reservoir Elevation 335, pushing the raise button will cause the gate to raise and stop automatically after raising one foot. Above the upper nappe, for reservoir El. 335, the raise button will cause the gate to raise continuously to clear the upper nappe for reservoir Elevation 346. A push button station is located at each hoist, operated from the walkway above the gates. With this arrangement an operator can walk along the walkway and open (or close) each gate a distance of one foot and repeat as many times as necessary to open or close the desired distance. With this procedure an even flow over the entire spillway is always maintained.

The power supply for the hoist motors is obtained from either of two independent feeders originating at the distribution switchboard in the stair tower at the end of the spillway nearest the powerhouse. In order to provide power during a maximum flood with the powerhouse out of service, an emergency gasoline engine driven generator is located in the stair tower. This has sufficient capacity

(approximately 125 kw) to start a tainter gate hoist motor while several others are in operation and to carry a reasonable lighting load in the dam.

Servicing the gates will be done by the use of a floating bulkhead gate. This is a steel gate which is long enough to span from center to center of piers, designed to float upright in the water. It can be floated into position in front of the spillway gate seated on the sill and sealed against the pier. The water pressure holds the gate tightly against the piers. This gate has compartments which can be filled or emptied with water by an electric pump, and the gate can be capsized to float on its side when it is being moved from one place to another. When not in use the gate is stored on brackets set in the upstream face of the dam.

At the downstream toe of the spillway a bucket is provided to deflect the water and to reduce erosive tendency of high velocity flows. Since the foundation is rock of good quality, no other means of erosion control was considered necessary. Model tests made by the Corps of Engineers formed the basis of determining the radius of the bucket and also the height of the training walls at each end of the spillway. A fifty-foot radius on the bucket was adopted for the design. The model tests indicated that the tailwater depth will be sufficient to produce satisfactory roller action at all phases of operation and no spraying should occur.

In addition to the crest gates, eight sluices, each 4 ft. wide and 9 ft. high, are provided. They are for the purpose of providing a means of drawing down below crest level if necessary and for diversion during construction. These sluices are located in the center of spillway blocks, under the piers; the sluice gates are hydraulically operated from the operating gallery extending through the spillway. These gates are single gates but an emergency bulkhead is provided to close off the intake in case repairs are required. This emergency gate is designed to be lowered and raised by a crawler crane from the roadway bridge over the spillway.

The bottom of the sluice gates are at Elevation 190 which is as low as feasible and still have the intake above river bed. The elevation of the spillway buckets opposite the sluices varies from 163 to 179. With the pool at spillway crest, elevation 300, the total discharge through the sluices is approximately 17,000 cfs which can be maintained through the sluices with the reservoir down to 263 which

is below the minimum drawdown. The intakes for the sluices have flared rectangular entrances, the sides conforming to an elipse having an equation—

$$\frac{x^2}{4^2} + \frac{y^2}{\left(\frac{4}{3}\right)^2} = 1$$

and the top and bottom conforming to an elipse having an equation—

$$\frac{x^2}{9^2} + \frac{y^2}{\left(\frac{9}{3}\right)^2} = 1$$

The floor of the conduit is horizontal to a point 50 feet from the base line where it will slope downward in the form of a parabola tangent to the floor and to the curve of the spillway bucket. The formula for this parabola is

$$y = \frac{x^2}{360}$$

which conforms to the shape of a jet having a velocity of 76 ft. per second or about 10 per cent greater than the maximum velocity with headwater at the top of the crest gates. This eliminates any tendency for a vacuum between the bottom of the sluice and the jet. In order to maintain positive pressures throughout the conduit, the roof of the conduit will be restricted at the outlet.

The width of the concrete blocks in the spillway section of the dam is 48 ft., the gates being 40 feet wide and the piers 8 ft. wide, making a total of 48 ft. Joints are made in the center of the spillway opening, which places the pier in the center of the block.

The non-overflow sections of the concrete dam are adjacent to the earth dam abutment sections and form a tie between the earth and concrete dam. They are a gravity section designed to take full load. No account is taken of the support given by the portion of the earth dam which slopes against the concrete section of the dam at the ends. The blocks in the non-overflow section are 48 feet wide to provide uniformity with the spillway section. The seven intake blocks opposite the power plant in which the penstock are embedded are 62 ft. wide, being governed by the unit spacing in the power plant.

A seal between the blocks in the dam is provided by two copper seals spaced 3′0″ and 6′0″ from the upstream face, between which, centered on the joint, is an 8″ diameter vertical well drained at the lower inspection gallery level. Concrete will be poured in 5 foot lifts except the first ten feet which will be 2½ feet.

The 24 foot roadway is supported on the top of the dam by cantilevering from the downstream face. A sidewalk is provided on each side, and a low parapet with an aluminum railing on top was designed to permit easy vision from automobiles without sacrificing safety.

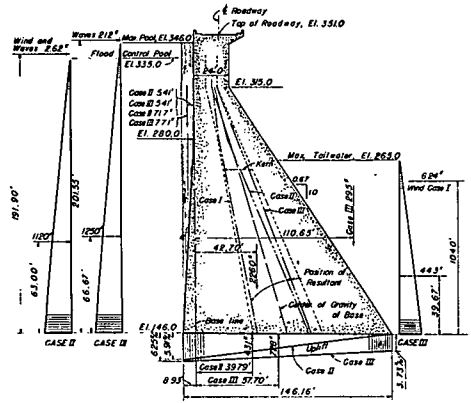
There is an inspection gallery 5′×7′, extending the full length of the concrete section of the dam, and located 10 feet from the upstream face of the dam close to the foundation rock. An operating gallery 6′×8′ extends across the spillway for access to the sluice gates, and it continues across the penstock sections of the dam for access to the 24″ by-pass valves at the intake gates.

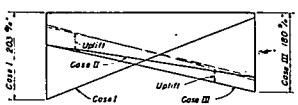
There is a stair tower at each end of the spillway connecting both galleries and extending to the roadway level. There is also an elevator in the stair tower nearest the power plant. Direct access to the operating gallery is obtained by a short gallery from the generator room of the powerhouse. There is also an adit to the outside at the switchyard level at the end of the power plant. Emergency ladder exits are provided at each end of the dam from the lower inspection gallery.

Drilling and grouting under the concrete dam will be done from the inspection gallery in two stages. The first stage consists of low pressure grouting through 1½″ holes 8′0″ on centers 40 feet deep. The second stage consists of high pressure grouting through 1½″ holes 16′0″ on centers 80 feet deep. Three inch diameter drain holes will be drilled about 60 feet into the rock from the inspection gallery downstream from the grout curtain after the grouting has been completed. These will drain into the inspection gallery and then to the main sump in the dam from which the water will be pumped.

Figure 3 shows the stability analysis for the principal sections of the concrete dam. The design assumptions were as follows:

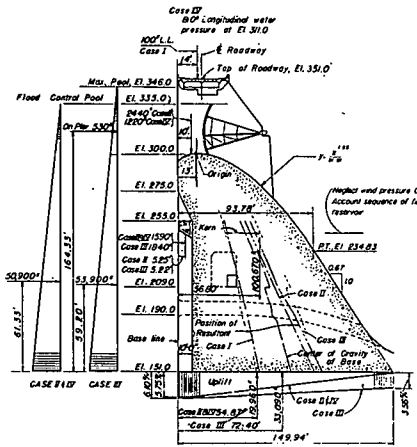
1. Earth fill and silt, both upstream and downstream of the dam, were neglected.
2. No allowance was made for ice pressure.
3. Seismic forces were neglected. (The resistance to earthquake forces was investigated for a horizontal acceleration based on 1/10 gravity. The effect of vertical acceleration was not considered.)



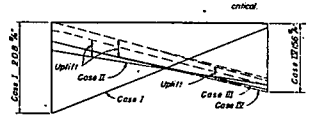


STRESS TABULATION - NON-OVERFLOW SECTION												
ELEV.	CASE	ΣV (KIPS)	ΣH (KIPS)	$\frac{\Sigma H}{\Sigma V}$	Resultant angle with vertical (°)	$\frac{M_r}{M_o}$	CONCRETE STRESSES (LBS/SQ IN.) HEEL T_v	HEEL T_i	TOE T_v	TOE T_i	S_o
	I	158	12	-	4.2	-	36	36	40	40	-
315	II	124	15	0.18	8.8	5.93	13	13	63	63	4
	III	121	32	0.26	12.8	2.70	6	6	68	68	9
	I	389	2.2	-	0.0	-	97	97	0	0	-
280	II	280	98	0.35	8.8	3.17	38	38	35	58	14
	III	285	138	0.61	1.6	1.64	91	81	60	87	20
	I	2250	62	-	2.6	-	203	204	12	17	-
146	II	1901	1123	0.59	9.4	8.00	75	75	178	214	53
	III	1904	809	0.42	8.2	1.76	74	74	180	238	39

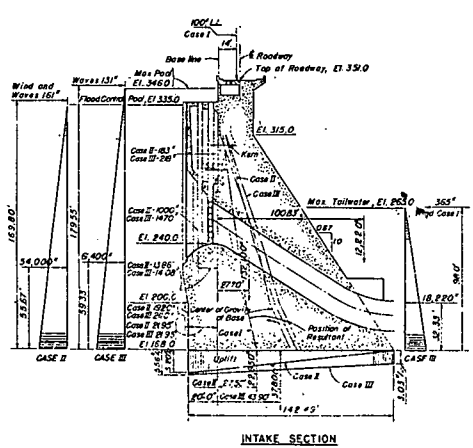
* Dist. from upstream edge of kern Case I, and downstream edge of kern Cases II and III

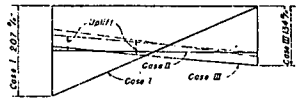


NOTE: The concrete stresses for the spillway section without sluices and upstream bulkhead varied slightly from those shown below but in all cases were less critical.



STRESS TABULATION - SPILLWAY SECTION												
ELEV.	CASE	ΣV (KIPS)	ΣH (KIPS)	$\frac{\Sigma H}{\Sigma V}$	Resultant angle with vertical (°)	$\frac{M_r}{M_o}$	CONCRETE STRESSES (LBS/SQ IN.) HEEL T_v	HEEL T_i	TOE T_v	TOE T_i	S_o	S_i
	I	2,060	-	-	1.8	-	96	96	5	8	-	-
234	II	19,050	8,040	0.31	7.2	2.03	47	47	49	75	20	-
	III	15,100	10,080	0.73	9.3	1.71	32	32	30	78	20	-
	I	30,840	-	-	0.7	-	190	190	0	0	-	-
209	II	42,840	13,800	0.36	11.3	2.07	84	84	76	110	37	-
	III	33,340	25,330	0.73	8.6	1.74	68	68	88	118	39	-
	I	64,770	-	-	4.2	-	178	179	9	13	-	-
190	II	53,600	31,500	0.59	7.9	1.96	63	63	113	164	43	-
	III	44,850	33,930	0.77	3.3	1.74	44	44	183	178	47	8.8
	I	102,710	-	-	1.9	-	208	208	8	18	-	-
151	II	81,360	50,900	0.62	5.1	1.83	63	63	140	208	52	-
	III	67,140	54,430	0.83	0.1	1.53	44	44	131	220	55	8.1
	IV	80,340	52,000	0.61	3.8	1.88	45.	45.	154.*	226.	56.	-





STRESS TABULATION - INTAKE SECTION												
ELEV.	CASE	ΣV (KIPS)	ΣH (KIPS)	$\frac{\Sigma H}{\Sigma V}$	Resultant angle with vertical (°)	$\frac{M_r}{M_o}$	CONCRETE STRESSES (LBS/SQ IN.) HEEL T_v	HEEL T_i	TOE T_v	TOE T_i	S_o
	I	12,640	-	-	4.2	-	29	29	39	39	-
315	II	18,060	2,130	0.08	4.6	9.31	23	23	41	41	3
	III	11,940	19,210	1.17	41.1	5.25	22	22	47	47	5
*	I	52,400	-	-	3.0	-	138	132	18	26	-
240	II	30,200	17,660	0.58	17.8	2.56	98	98	65	84	23
	III	48,400	20,700	0.43	18.8	2.48	78	78	96	120	27
	I	94,300	-	-	0.8	-	173	173	4	6	-
200	II	79,700	33,000	0.44	17.3	2.69	98	98	98	135	35
	III	72,500	33,000	0.46	18.7	2.52	79	79	125	169	33
	I	122,800	365	-	0.1	-	207	207	1	1	-
168	II	108,500	54,600	0.50	18.0	2.29	101	101	108	154	43
	III	105,600	43,300	0.41	13.8	1.95	98	98	134	188	34

* Dist. from upstream edge of kern Case I, and downstream edge of kern Cases II and III.

FIG. 3.—STABILITY ANALYSIS.

4. Full uplift at the heel and toe respectively, varying uniformly between these two points, was considered effective over 50 per cent of the area
5. Wave pressure for an 80 mile per hour wind over a fetch of 4 miles.
6. Weight of concrete equals 150 lb. per cu. ft.

The loading conditions were as follows:

Case I	Reservoir empty Wind on downstream side
Case II	Reservoir at El. 335.0 Spillway gates closed Penstocks empty No tailwater Uplift Wind and wave pressure
Case III	Reservoir at El. 346 Spillway gates open Penstocks empty Tailwater at El. 265 Uplift Wave pressure

Under these conditions the factor of safety against overturning is greater than the minimum allowable of 1.5, and the inclined stress in the dam or foundation is within the maximum allowable of 416 lb. per sq. inch or 30 tons per sq. ft.

Figure 1 shows a typical cross section of the maximum section of the rolled fill earth dam which is 200 feet high. The slopes vary from 1 on 3 on the lower portions to 1 on 2.5 near the top. The top was made 40 feet wide to provide for sidewalks and a berm on each side of the roadway, which is more than necessary for the stability requirements of the earth dam. This permits the steeper slope at the crest. It was found that the ratio of permeability of the embankment core material to the foundation material is about 1 to 200. A cutoff trench was mandatory, and one was excavated to sound rock and backfilled with rolled core material. The bottom width of the trench was made 12 ft. to permit construction operations. A concrete core wall or sheet pile cutoff was not used because the other type was more economical. To prevent seepage through the rock foundation, a grout curtain was made the full length of the cutoff. It is an extension of the grout curtain under the concrete dam, and it was placed in the same manner except the grouting was done from the rock surface instead of from a gallery. With the comparatively

high permeability of the foundation material the foundation itself will act as a drain blanket so that no constructed drain blanket was required. The depth of the overburden foundation material varies from about 20 ft. to 50 ft. Suitable borrow material for the rolled fill is available nearby and some of it will come from the quarry stripping.

A free board of 5 ft. above the maximum flood is provided making the crest at Elevation 351. It was considered that wave action against the embankments would not endanger the security of the dam provided the slopes were protected with heavy riprap. The upstream face will be protected with an 8 foot thick surface of quarry run stone down below the minimum drawdown. The downstream face will be covered with 12″ of crushed rock, and the lower slopes near the power plant will be heavily riprapped.

A cross section through the powerhouse and intake is shown in Figure 4. The upstream wall of the powerhouse is separated from the toe of the dam at the lowest rock surface by a distance of about 8 feet. The powerhouse structure is separated from the structure built over the toe of the dam by a contraction joint and all loads upstream from this joint are cantilevered from the toe of the dam. The separation is intended to avoid any trouble due to possible difference in foundation loading between the dam and powerhouse, which might cause differential settlement with consequent alignment difficulties on the power units. The walls of the powerhouse are designed to withstand outside water pressure up to Elevation 240′ or 22′ above the generator room floor which dictates a concrete structure.

The main turbines will be Francis type rated 55,000 hp at best efficiency at 136 foot average head and will be guaranteed for 46,200 hp at 111 foot minimum head. The speed will be 100 rpm. The center line of the distributor is at Elevation 193′, which is 8 feet above the minimum tailwater with one unit running and the same as tailwater with seven units running. The unit spacing is 62 feet. The penstock is 20 feet in diameter, and the draft tube is the elbow type with two openings to the tailrace. The scroll case is made of steel plate embedded in the concrete.

The main generators will be 40,000 kw capacity. They are rated 44,444 kva with temperature rise of 60° C. continuous, 13,800 volts, 3 phase, 60 cycles, .9 power factor.

A 250 ton crane travels on concrete crane girders over the gen-

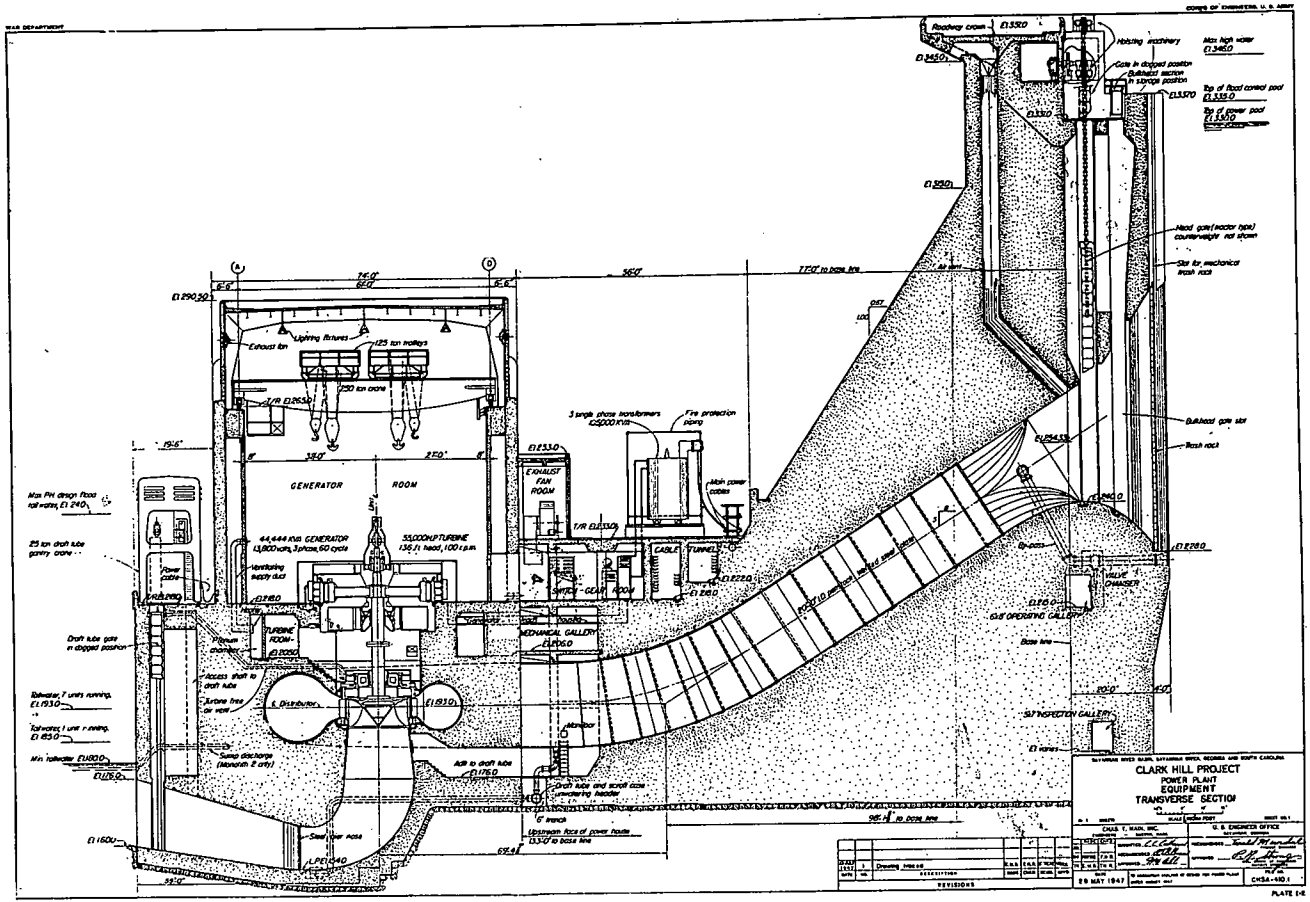


FIG. 4.—CROSS SECTION THROUGH POWERHOUSE AND INTAKE.

erator room. The crane is designed to handle the weight of the generator rotor and water wheel and will also be used to unload railroad cars which can be run into the powerhouse on a spur track at the shore end.

A 25 ton gantry crane located on the draft tube deck is designed to handle the draft tube gates.

Two station units, each 1000 kw at .8 power factor, 1250 kva, 480 volts are provided for carrying the station load requirements. The station service system and the main units will not be interconnected for power purposes. The control room is located at the shore end of the powerhouse in the service bay at an elevation above any possibility of flooding, and the offices are all on the top floor.

Facilities for the visiting public are provided at the shore end of the powerhouse. A balcony at the end of the generator room gives a view of the entire room. A large glass window provides a complete view of the control room. The reception room is located directly over the main lobby and furnishes space for exhibits, pictures, etc. The public space is so arranged that visitors cannot enter the operating parts of the powerhouse.

The headgates have a clear opening 17'0" wide and 28'0" high and are the tractor type weighing about 55 tons each. A single gate is used for each penstock, and it is operated by a hoist located in a chamber just below the roadway. Upstream from the gate slot is another slot for stop logs. These stop logs are made of steel beams and are handled by a crawler crane from the roadway when it is necessary to use them. The main gate can be raised to a position above the maximum power pool level and dogged off for inspection, painting, or repairs. A trash rack is provided on the upstream face of the intake with provision for a mechanical trash rake also operated by the crawler crane. The velocity of the water through the gross area of the trash racks for the average flow of 3840 cfs in the penstock will be 2.7 ft. per second. The velocity at maximum turbine discharge will be 3.4 ft. per second. The velocity in the penstock for average flow will be 12.2 per second.

The gates are operated by individual hoists and are designed to be closed under unbalanced pressure but to be opened only under balanced pressure. They are normally hung in the gate slots just above the intake opening. The gates are lifted by two link and pin type chains fastened to a pin in the end of each gate. The chains

pass over sprockets at the roadway level and down over drive sprockets inside the hoist chamber just below. A 40,000 lb. counterweight is hung on each of these chains, making possible a considerable reduction in the capacity of the hoisting equipment. Normal operation is from the main push button station in the hoist chamber, but it is possible in an emergency to lower the gate from its "Normal Operating" gate position just above the penstock opening, by a control switch on the unit turbine and governor board in the powerhouse. The lowering speed is 4 ft. per minute and each gate is operated by a 15 horsepower motor.

The main single line wiring diagram is shown in Figure 5. Future Generator No. 1 will be connected directly to a three phase transformer. The other generators are to be operated as paired units, each pair connected to a bank of 3 single phase transformers. Each of these transformers are 35,000 kva and step up the voltage from 13,800 volt generator voltage to 115,000 volt high tension voltage. The high tension side of the transformers will be connected to the switchyard through pipe cable of the high pressure oil or gas filled type. The switchyard has not yet been designed, but its voltage will probably correspond with the 110 kv prevailing transmission voltage of the area. There is also a possibility of a second voltage of 220 kv being used to interconnect with other stations.

One of the principal factors in determining the general plan of construction of this project was the method of diverting the river and the sequence of the construction stages as shown in Figure 6. The flow in this river indicates that a flood may be expected at any time of year even though certain months are drier than others. A flood of 160,000 cfs occurs on an average of once every 6 years and 140,000 cfs every 4 years.

The first stage cofferdam includes the major portion of the spillway and non-overflow section on the right bank. All the enclosed spillway blocks have to be left low. This cofferdam has already been built and is of the steel sheet pile cellular type of sufficient height to protect against the flood of 140,000 cfs. To provide a channel for the river wide enough to prevent scour, the left bank was dredged away to the limits of the powerhouse and tailrace excavation, the material being placed in the switchyard fill downstream from the left abutment.

The second stage cofferdam will include the remainder of the

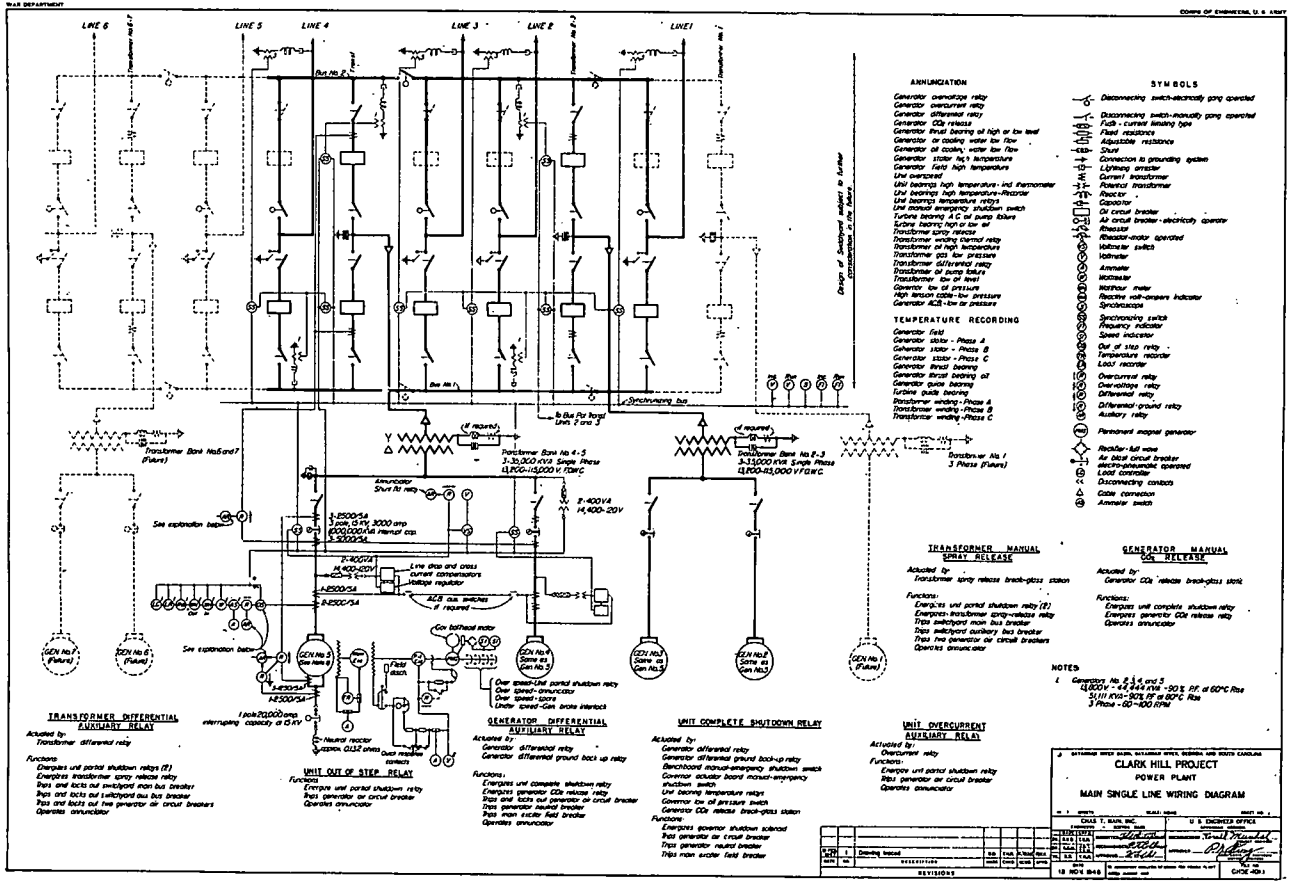


FIG. 5.—MAIN SINGLE LINE WIRING DIAGRAM.

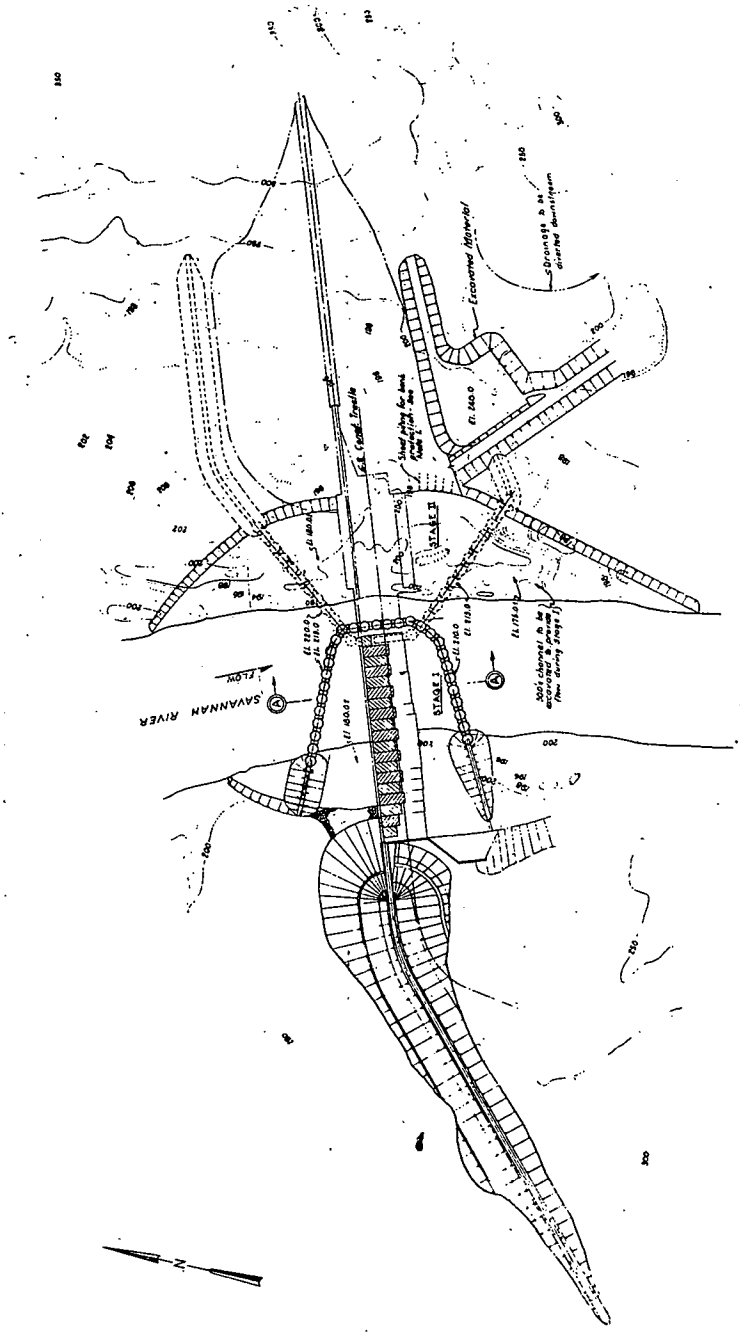


FIG. 6.—RIVER DIVERSION AND CONSTRUCTION SCHEME.

spillway and all the powerhouse and intake section and the left abutment. It will be designed to protect against a flood of 160,000 cfs and low blocks left in the first stage spillway construction are designed to pass this flow. During this stage the intake structure and spillway within the cofferdam will be carried to full height as well as the earth dam abutment. The downstream part of the cofferdam will be left in place for the powerhouse construction which may extend over a longer period than the dam.

The final or closure stage will consist of building up the low blocks left in the spillway. One or more steel gates will be required to accomplish this, and it can be done with a flow of approximately 12,000 cfs. Flow will be diverted through the permanent discharge sluices as soon after start of closure as practicable depending on the status of the sluice gate erection. The eight sluices will take the entire flow after the headwater reaches a certain elevation (Elevation 204 for 5000 cfs and Elevation 242 for 12,000 cfs). Until the elevation is reached at which the sluices will take at least 75 per cent of the flow, ponding of the reservoir will not be sufficient to prevent some flow over the low blocks.

The Clark Hill Dam is a type of structure adapted to a concrete placing method using a steel trestle on the downstream side from which are operated revolving cranes for placing concrete in the dam and on which trains are operated bringing concrete to the cranes. The cross sectional dimensions of the dam are such that a crane can easily reach all parts of the dam and can also be used to erect the spillway gates, spillway bridge, penstocks and all other equipment. By properly locating the trestle between the dam and powerhouse, the cranes can also reach the entire powerhouse. The contractor for the dam is using such a scheme of construction.

The use of cableways for placing concrete is not practical for this project due to the excessively long span and the fact that there is no high ground above the dam on either side.

The granite quarry for producing aggregate is located about a mile downstream on the right bank, and the contractor has built a crushing and screening plant between the quarry and the dam for processing coarse and fine aggregate and sand. The concrete mixing plant is located close to the end of the concrete placing trestle and cement storage is close by. Cement is brought to this location by railroad which crosses the river on a temporary combination highway

FIG. 7.—FIRST STAGE COFFERDAM.

and railroad bridge. This bridge is constructed of surplus military truss and girder bridge spans supported on pile bents.

The first stage cofferdam is shown in Figure 7 which is an air view taken the latter part of October, 1948, looking toward the right abutment. It shows the completed earth embankment extending out to the point where the concrete section of the dam will begin. The contractor's concrete mixing plant and cement storage silos are at the left of the picture, and the temporary steel construction trestle for placing concrete is in the center. Two electric revolving cranes rated at 20 tons capacity at 75 foot radius are shown operating on the trestle. Trains for transporting concrete from the mixing plant to the trestle will be hauled by five 10-ton locomotives. The cement will be stored in 3 steel silos, two of which can be seen adjacent to the mixing plant. Two of these are 7,500 bbl. capacity and will be used for Portland Cement. The other is of 4,800 bbl. capacity and will be used for Natural Cement. The mass concrete in the dam will have 75 per cent Portland and 25 per cent Natural Cement in the

FIG. 8.—AGGREGATE PROCESSING PLANT.

concrete mix. The cofferdam cells are circular type with connecting arcs and are filled with sand and gravel from the river bed. The sheet piling will be pulled and reused again in the second stage cofferdam.

An air view of the construction plant area taken at the same time is shown in Figure 8. This gives a complete picture of the contractor's aggregate processing plant which extends from the primary crusher at the extreme left to the concrete mixing plant at the upper right of the picture. The carpenter shop with lumber storage piles is in the center of the picture, and the watertank is visible in the upper center. The long earth embankment forming the right abutment is in the background. The quarry is not shown in the picture but is at the left adjacent to the primary crusher. The primary crusher is a 42″ gyratory type with 250 hp motor. The product of this crusher is transported by an inclined conveyor to a 5′×14′ double deck screen, and part of the material is then passed through a secondary 20″ gyratory crusher with 125 hp motor which supplements the primary crusher. The material is then placed in an intermediate stock pile by an inclined conveyor. This stock pile is the cone shaped pile with crater in the top in the left center of the picture. The material

is transported from this stock pile to two screening structures where it is screened and part of it crushed again. The first screening structure consists of two 5′×12′ double deck screens and a 4¼ ft. cone crusher powered by a 150 hp motor. At this point the cobbles and coarse aggregate are removed and placed in the storage piles. The second screening structure consists of two 5′×12′ double deck screens and a 48″ gyrosphere crusher powered by a 150 hp motor.

At this structure the medium and fine aggregate are taken out and placed in the stock pile. These four stock piles are the four cone shaped piles of rock visible in the right center of the picture. The sand manufacturing plant is located to the right of the screening structures and consists of an 8′×12′ double ended rod mill with a 350 hp motor, a sand classified, and two 5′×12′ double deck screens. Sand is discharged into a fifth stock pile at the right end of the other four in the picture and somewhat smaller in size. The sized aggregate and sand is reclaimed from the stock piles by a conveyor operating in a tunnel beneath the piles. This conveyor transports it to the concrete mixing plant, which consists of three 4 cu yd concrete mixers. Close to the mixing plant is an ice plant and water cooler. This consists of five 30 ton ice machines and one water cooler. Under certain conditions ice will be used instead of water in the concrete mix to reduce the tendency to crack caused by the increase in temperature due to heat of hydration during the period of concrete setting.

Figure 9 is a perspective drawing of the project drawn to show the powerhouse in detail. The architectural design was made to por-

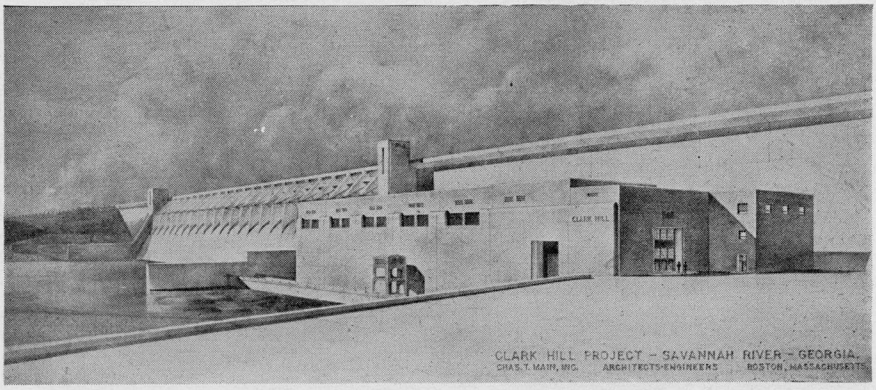


Fig. 9.—Drawing of Clark Hill Dam and Powerhouse.

tray the functions of the powerhouse building and dam in simplest form. Due to designing against flooding by high tail water, the power plant will of necessity be built of concrete. A design expressing the seven units of the main powerhouse as the principal motive with a somewhat subordinated service bay was used. Glass block windows over the crane rail of the powerhouse and in the stair towers on the dam, glass entrance doors, and aluminum hand railing and trim are featured. The only embellishments are the aluminum Corps of Engineers insignia over the main entrance, the aluminum letters "Clark Hill" on the face of the powerhouse and the flagpoles on the top of the stair towers on the dam. The stair towers at each end of the spillway were designed to harmonize with the powerhouse and to provide a pleasing transition from the spillway to the non-overflow structure of the dam. The massive structure of the dam itself was outlined only by requirements of structural design. This combined to give a harmonious although simple and functional architectural treatment.

The Clark Hill Project was recently awarded second prize for architectural design at the Annual Exhibition of Federal Architects held in the National Museum in Washington. A large color rendering based on the engineers' design also received first prize as the best presentation drawing. This rendering was made by E. A. Moulthrop of Atlanta, Georgia, for the Corps of Engineers and is shown in Figure 10.

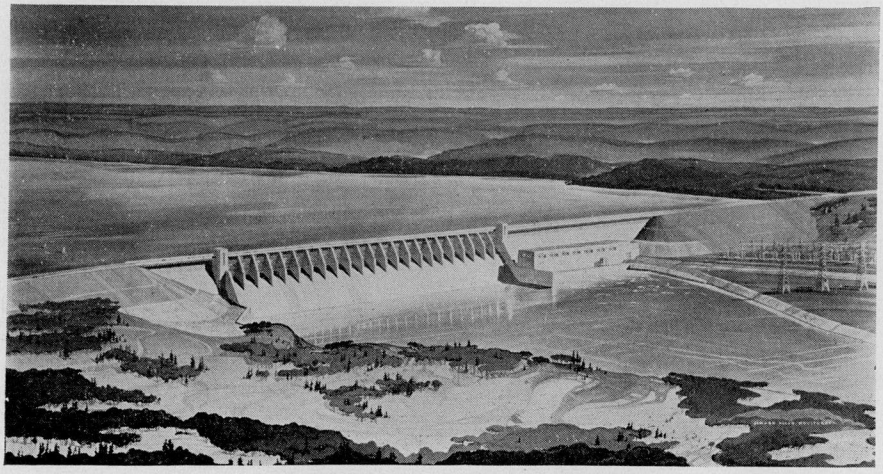


Fig. 10.—Drawing Clark Hill Project.

The project is being constructed under the direction of Colonel Paschal N. Strong, District Engineer for the Savannah District, Corps of Engineers. Torald Mundal was Project Engineer at the time the design was being made and C. M. Weston is Chief of the Clark Hill Branch for the Corps of Engineers. All States Constructors, Inc., are Contractors for the dam. Chas. T. Main, Inc., were the engineers for the design of the project for the Corps of Engineers. The writer was in charge of the design for Chas. T. Main, Inc.

OF GENERAL INTEREST

STATUS AND TRENDS OF LABORATORY INVESTIGATIONS OF SOILS AS EVIDENCED BY THE PROCEEDINGS OF THE SECOND INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING

By W. J. Turnbull*

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

Many papers on laboratory investigation, submitted to the Second International Conference on Soil Mechanics and Foundation Engineering, treat on several subjects and are difficult to classify, and a considerable number of papers outside those in Section II contain pertinent information on current practice and trends in laboratory investigations and should be included in the review. The writer does not profess to have studied all the papers in the Proceedings, but papers in several sections in addition to those in Section II have been considered in preparing this paper. The Proceedings of the Conference represent in the main the results of completed work; however, by attending the Conference and visiting several of the principal soil mechanics laboratories in Europe, the writer found much new work in progress and gained the impression that some of this work may be more significant in regard to future developments than the papers published in the Proceedings, and these impressions form a background for some of the opinions expressed in the following summary.

*Chief, Soils Division, Waterways Experiment Station, CE, Vicksburg, Miss.

Laboratory Investigations in General

Field explorations and tests, laboratory investigations, and application of the data obtained in design of earth and foundation structures form three closely interrelated and overlapping phases and should be considered as a whole. The assumptions made in or the limitations imposed by each one of the phases should be taken into consideration in planning the work in its entirety.

Because of the above-mentioned interdependence of the three main phases of the work, unified direction is highly desirable although not always possible, but the supervising personnel should, at least, be thoroughly familiar with all three phases, not only in general but also in regard to any particular project. The majority of large organizations, engaged in soil mechanics and foundation engineering, undertake by their own forces not only the laboratory investigations and basic design but also the field explorations and tests. The smaller organizations are generally represented by a qualified engineer during field investigations, who usually assumes direct supervision of the work.

As evidenced by the number of engineers attending and the papers submitted to the Second Conference, the use and stature of soil mechanics have increased enormously since the First Conference was held at Harvard in 1936. Returned questionnaires in the United States indicate that at least 144 public or private organizations and institutions are actively engaged in soil testing. A large number of these laboratories are small and primarily engaged in routine or control testing or are maintained for the purpose of instruction, but about 50 per cent of the laboratories are equipped for all types of soil tests, including consolidation, shear and triaxial tests.

Data obtained from similar questionnaires abroad are not yet available but are to be published in the forthcoming Volume VI of the Proceedings. The writer gained the impression that the majority of large soil mechanic laboratories in Europe are Government institutions, often connected directly or indirectly with the universities and engaged in both research and instruction, although primarily supported by the practical testing and consulting work they perform. The methods used in both field and laboratory investigations differ considerably in various countries and regions, and these differences are to a large extent governed by or are the result of the prevalent foundation conditions.

Review of Conference Papers

At the time of final revision of this paper for publication, only the first five volumes of the Proceedings and a list of the papers in the proposed Volume VII, but not of the papers to be published in Volume VI, have been received, and the following tables and review are therefore incomplete. A classification of the papers in the General Report Section and in Volumes I, II, III, IV, V, and VII in accordance with the countries of origin is shown in Table 1. It will be seen that England is first and United States second in the number of papers contributed to Section II. The total number of papers in this section is about one-seventh of all papers submitted; however, as shown in Table 2, laboratory investigations are discussed in over one-third of all the papers.

Table 1.—Papers Submitted by Various Countries

Country	Number of Papers Sect. II	Other Sect.	All Sect.
Argentina	1	1	2
Australia	2	11	13
Austria	4	10	14
Belgium	1	15	16
Brazil	—	5	5
Canada	3	4	7
Czechoslovakia	—	4	4
Denmark	—	9	9
Egypt	1	2	3
England	19	43	62
France	2	28	30
Hungary	—	7	7
India	—	3	3
Ireland	—	2	2
Italy	2	3	5
Japan	—	5	5
Mexico	—	3	3
Netherlands	3	37	40
Palestine	—	3	3
Poland	3	6	9
South Africa	—	2	2
Spain	1	5	6
Sweden	1	6	7
Switzerland	1	24	25
Turkey	1	—	1
United States	15	114	129
Venezuela	—	1	1
Yugoslavia	—	1	1
Total	60	354	414

The summary includes papers in General Reports, Volumes I, II, III, IV, and VII, but not in Volume VI.

In the Proceedings, the papers in Section II are listed under seven principal headings, but many of the papers

are placed in the "general" and "miscellaneous" groups. For the purpose of facilitating the review a more detailed classification of the papers has been attempted, and the results are shown in Table 2.

TABLE 2.—SUBJECTS OF PAPERS ON LABORATORY INVESTIGATIONS

Group	Type	Section II		Other Sections	
General Reviews		2	2	4	4
Identification	Composition, Origin	4		4	
	Properties	2	6	1	11
	Classification			6	
Preparation of Test Specimens		1	1	—	—
Flow of Water	Permeability	—		—	
	Capillarity	2		2	
	Waterproofing	1	7	1	4
	Electro-Osmosis	4		1	
Volume Changes	Consolidation-Swelling	4		11	
	Compaction, CBR	1	5	12	23
Strength and Deformation	Unconf. Compression	2		2	
	Direct Shear	7		—	
	Torston Shear	—	21	1	5
	Triaxial Tests	11		2	
	Pore Water Pressure	1		—	
Model Tests	Flow of Water	1		5	
	Surface Loading	4		4	
	Pile Loading	1	7	4	19
	Earth Pressure	—		6	
	Photo-Elastic	1		—	
Soil Properties	Structure of Soils	1		2	
	Special Properties	2	11	4	20
	Regional Surveys	7		11	
	Correlations	1		3	
Totals		60		86	

The total number of papers in Section II is 60, including the general report and papers to be published in Volume VII, but as shown in the table, a still greater number of papers in other sections contain data on laboratory investigations. On the other hand, many papers in Section II report mainly the results of basic research on soil properties and do not describe the laboratory equipment and technique in detail. Therefore, even the detailed classification in Table 2 is arbitrary and incomplete to some extent.

A review of the papers in Section II, primarily discussing the determination of the shearing resistance of soils, was presented at the Conference by W. Kjellman. Other very brief reviews by Golder, Escario and Salas, Toms, and others contain discussions of current laboratory practice in various

countries, and papers on the principal testing procedures used in and the organization of individual soil mechanics laboratories are to be published in Volume VI of the Proceedings.

Identification

The ultimate purpose of identification of subsurface materials by visual examination and simple identification or routine tests is to classify the materials in regard to their behavior under given conditions and to enable an estimate to be made of significant physical properties or coefficients, the determination of which generally requires performance of major laboratory or field tests.

As evidenced by many papers, in various parts of the Proceedings, for example by Winterkorn, Moos and Bjerrum, Turnbull and Fisk, and by Hunt, there is increased use or adaptation of geology in the application of soil mechanics to civil engineering problems. Not only is geology helpful in planning field explorations and interpreting the results, but the position and origin of the various strata often give some indication of their physical properties. It is evident that serious efforts are under way in utilizing existing tests and in developing new tests to supplement the mechanical analysis for better identification and classification of soils. The laboratory examination and identification tests as illustrated by current trends may be divided into composition and property tests.

Composition tests include examination of stratification and macro structure, determination of grain-size distribution, water content, degree of saturation, content of organic matter and significant minerals and chemical compounds. Among contributions to further development of such tests may be mentioned a new graphical solution of Stoke's Law (Youssef), a method of micro-sedimentation analysis of silt and clay fractions and examination of these fractions by the electron microscope (Clare), and determination of the unit weight and degree of saturation of irregular shaped samples by means of impregnation with and immersion in suitable oils such as kerosene (Russell). It may also be mentioned that the chalk content often is determined in Europe, and that investigation of the silica content as an index of soil properties is in progress in Belgium. The significance of the various minerals in the clay fraction is discussed in a paper by Skempton and in several papers in other sections of the Proceedings.

Properties determined as a part of the identification or routine tests include the specific gravity of the soil grains, void ratio, unit weight of the soil mass, Atterberg limits, degree of compaction or density, and the consistency of the soil in both its natural and remolded state by means of cone penetration, simple unconfined compression tests, or simply by feel.

Methods for determination of the possible maximum and minimum void ratios are discussed in two papers by Kolbuszewski, and it is evident that standardization of these tests is desirable in order to obtain uniform results in estimating the degree of compaction or density of soil deposits. Valuable information on both the degree of compaction and the consistency or strength of the soil in situ can be obtained during the subsurface exploration, when the penetration resistance of a sampler is determined or modern sounding methods are used. Such methods for field determination of the physical properties are discussed in several papers in Section III of the Proceedings.

Several papers, both in Section II and other sections, present or discuss correlations between the results of the identification tests and significant soil properties, especially the shearing resistance.

Classification of the soils, on the basis of the identification tests, in groups with similar consolidation, strength, and other significant proper-

ties, is discussed in several papers outside Section II. This is a perennial and highly controversial subject, and a generally acceptable solution is not yet in sight. There are so many variables that the classification would be too complicated if they all were to be taken into consideration. It is doubtful that a simple classification which is adequate for all purposes can be developed, but it may be possible and would be desirable to agree on a simple standard for general or preliminary classification, and this could then be supplemented by more detailed or special classifications for various uses of the soil, such as foundations, earth dams and levees, and bases for roads and airfields.

Preparation of Test Specimens

Handling of samples in the laboratory and techniques for preparing specimens for the major tests are very important when uniform and reliable test results are to be obtained. Improved rotary cutting equipment and technique which decrease structural disturbance of the test specimens are described in a paper by McRae. There are indications that swelling and increase in airspace of the soil before testing—caused by absorption of air, expansion of free air in the soil, or release of air and other gases dissolved in the pore water—in some cases may affect the test results and their interpretation to a considerable extent. Fully satisfactory methods for avoiding such an expansion of the test specimen or to counteract or correct for its influence are not yet available.

The preparation of test specimens is often time-consuming, but when a slight disturbance can be tolerated, specimens with constant dimensions and volume may be obtained quickly by means of a small piston sampler (Hvorslev). In preparation of test specimens of remolded soil it is often difficult to obtain uniformity of several test specimens as well as throughout a single specimen. An improved and apparently very successful technique for preparing uniform test specimens of sand is described by Chen.

Flow of Water

Tests concerning flow of water in soils are made for the purpose of obtaining data for solution of seepage, saturation, drainage, and unwatering problems.

Permeability determination is the principal test in this group, but new laboratory methods are not discussed in papers submitted to the Conference. However, the technique of laboratory permeability tests is fairly well established, and the principal difficulty consists in obtaining undisturbed samples of very permeable and coarse grained soils. The average permeability of such deposits can be determined by large-scale pumping test, but such tests are very expensive and cheaper and quicker field methods are needed. Determination of the permeability of peat in situ by observing the rate of fall of the water level in a piezometer is described by Cuperus in the section on Field Investigations, but the method of calibration of the apparatus, evaluation of the data, and the reliability of the results are not discussed.

Capillary rise of water in soil is of extreme importance in agriculture and also affects several civil engineering problems, such as frost action and the degree of saturation under foundations and pavements. The Beskow and Jurgenson capillarimeters are commonly used, but as suggested in a paper by Bernatzik, the capillarity of sand may also be determined by simple triaxial tests, provided the angle of internal friction of the sand is known. Krynine reviews the results of capillarity experiments by several organizations and indicates that the results obtained by the above-mentioned capillarimeters are not reliable, that the formula for the rate of capillary rise should include a term involving the particle size, and that the increase in

density and viscosity of water in the pores of fine-grained soils should be taken into consideration. The physical and theoretical aspects of capillarity are discussed in several papers outside Section II. Kastner discards the usual concept of surface tension and explains the capillary rise by action of molecular forces. In two papers Sitz reviews the basic problems in flow of groundwater and in Terzaghi's conception of transmission of tensile forces in water and discusses the difference between molecular and capillary water.

Waterproofing by means of resinous water repellents is used to decrease the flow and capillary absorption of water in soils and thereby to increase the stability of bases for roads and airfields. Clare has investigated the effect of various resins and oils by means of a capillary water absorption test and found that Vinsol resin and also fuel oil with a slight addition of paraffin wax gave maximum waterproofing but did not affect the cohesion and angle of internal friction of the soil. In a paper in Section IX, Eustis and Shockley report results of both laboratory and extensive field tests with two water repellents, Stabinol and 321, which indicated that these repellents are effective when the soils have some plasticity, and that the required amount of repellent for a given type of soil can be estimated by means of its plasticity index.

Electro-osmosis has attracted great interest in recent years as a means of increasing the rate of flow of water through soils of low permeability. The method was originated by L. Casagrande who has submitted a paper on the results of his basic research on this problem. He finds that the electro-osmotic flow is governed by a law similar to the Darcy law for gravity flow of water through soil: that the electro-osmotic coefficient of permeability does not vary greatly with the soil type; that electro-osmotic flow causes lamination of the soil; and that the flow seems to take place through fissures formed rather than through the actual pores of the soil. Electro-osmosis is not an economical method for obtaining increased rate of consolidation of large soil masses but it can be used to great advantage for temporary stabilization of silty and clayey soils during excavation of deep cuts and for foundation structures, as proved by several large-scale practical applications of the method.

Three other papers on electro-osmosis were submitted to the Conference. Bernatzik presents a review of the physical, electro-chemical, and mechanical aspects of electro-osmosis. Geuze-Bruyn-Joustra in one paper and Dawson-McDonald in another paper report the results of preliminary investigations of electro-osmosis and also on the occurrence of electro-chemical hardening when certain types of electrodes are used.

Volume Changes

Laboratory investigation of volume changes of soils may be divided into two groups. The purpose of one is to obtain data for estimation of settlement or rise of foundation or earth structures through the processes of natural consolidation or swelling, and the purpose of the other is to determine the increase in density and strength by mechanical compaction of soils to be used in earth structures.

Consolidation tests are generally performed by means of consolidometers of the Terzaghi-Casagrande type. A great variety of loading arrangements is in use, but the basic principle of confined compression prevails, although triaxial type apparatus also is used or suggested. A satisfactory prediction of settlements on the basis of the results of standard tests and by use of the consolidation theory is often obtained, especially when relatively thin, compressible strata are involved, but there are also many examples of considerable difference between the predicted and

actual settlement, indicating that the laboratory tests often should be supplemented by field observations. Only three or four papers in Section II concern consolidation, and to gain a reliable conception of the problems and present status of this subject it is necessary also to study papers in Sections I and VI. The principal problems encountered in the use of standard confined compression tests and in application of the results obtained may be summarized as follows.

Structural disturbance of the soil during sampling and preparation of the test specimens may have considerable influence on the test results and are discussed by Zelst, Fadum, and others. The structural disturbance can be decreased by use of recently improved methods of sampling and preparation of test specimens, but a temporary reduction of external stresses on and a swelling of the test specimen cannot be avoided. This swelling may cause complications, especially when it is due to expansion or release of air and other gases in the soil and pore water.

Consolidation tests on mixed materials are often made on test specimens from which pebbles and stones have been removed, but Murdock finds that the results of such tests are not reliable, even when the volume of the removed coarse materials is taken into consideration. When the coarse material is not removed, test specimens of large diameter and thickness must be used with a consequent increase in the time required for the test and greater influence of the side friction.

The side friction in the consolidometer decreases the average vertical stress in the soil and should be taken into consideration when thick specimens are used, but it is often difficult to evaluate properly. A consolidometer in which the side friction is eliminated has been developed in Sweden but is not described in the Proceedings. Murdock suggests use of a triaxial type of apparatus in which the diameter of the test specimen is kept constant by varying the lateral pressure automatically.

The lateral stresses in the soil during confined compression do not always correspond to the stress conditions in the soil in situ, and this may account for some of the differences between predicted and actual settlements. Buisson suggests use of a triaxial type apparatus for consolidation tests on cubic test specimens and determination of the Poisson ratio, and presents formulas for application of the results. In general, when the lateral pressures are measured and are to be taken into consideration, the currently used one-dimensional consolidation theory must be revised.

The rate of load application may have considerable influence on the results of consolidation tests on some soils. This problem is given some consideration in a new theory by Geuze and Bruyn, but it has not yet been subjected to extensive experimental investigation.

The secondary—in some cases called secular—consolidation is discussed in several papers from both a theoretical, experimental, and practical standpoint. A new formula in which the effects of both the primary and the secondary consolidations are combined is proposed by Koppejan, but much research in both field and laboratory will probably be required before a generally acceptable solution of this problem is obtained.

Seasonal swelling of soil, and also swelling during excavation for foundation structures, is an important problem in several regions, but laboratory investigations to determine the amount and rate of swelling, and the pressures exerted are not discussed in papers submitted to the Conference.

Compaction tests in the laboratory serve as pilot tests for field operations. Concurrently with these tests, California Bearing Ratio tests are generally made on both the compacted test speci-

men and the soil in situ, and the CBR tests is therefore not listed separately under strength and deformation tests. Section II contains only a single paper on standard compaction tests, in which Little compares the results of the Proctor and Dietert compaction tests and suggests minor modifications. As already mentioned, a discussion of methods for determination of both the maximum and minimum attainable voids ratios of a soil is contained in two papers by Kolbuszewski.

Laboratory compaction tests are discussed in detail in several papers in Section IXb. Gawith suggests a new rammer or tamping tool for standard compaction tests; McLean and Williams describe automatic compaction machines, and Lane dscusses the Providence method of compaction by vibration. In several papers Proctor describes laboratory compaction tests, discusses the relationship between the compactive effort and the density and shear strength of the soil, and compares the results obtained by field and laboratory compaction. A summary of the papers in Section IX is contained in a review prepared by Mr. Frank A. Marston, and here it shall only be mentioned that one of the principal problems in laboratory compaction tests is the developments of compaction methods which will produce test specimens having the same moisture-density and stress-strain characteristics as the soil compacted by means of the various types of field compaction equipment. Currently there is in some cases considerable difference between the results obtained in the laboratory and in the field.

Strength and Deformation

Determinaiton of the strength and stress-strain relationships of soils under various conditions is probably the most important and controversial of all laboratory investigations, as evidenced by the large number of papers submitted. Some of these papers describe improved equipment and techniques, but the majority report on the results of basic research and practical applications. Determination of the strength of soils is discussed in many papers outside of Section II, and those in Section I concerning the conditions of failure are particularly important.

Unconfined compression tests, the simplest of all strength tests on cohesive soils are widely used for both classification and design purposes. A variety of stationary laboratory apparatus for performance of these tests are in use, but recent developments consist mainly in refinements of the load measuring devices and none are described in papers submitted to the Conference. A portable apparatus with a simple arrangement for recording the stress-strain curve was developed in England some years ago and is mentioned by Glossop, and a pocket-size apparatus for both field and laboratory use is described in a paper by Hvorslev.

On account of their simplicity and in spite of some controversy regarding their suitability, unconfined compression tests on cohesive soils are being used to an increasing extent for design purposes. Tschebotarioff and Bayliss describe the performance of unconfined compression tests on varved clays, find that the results are suitable for design purposes, and suggest that the sensitivity of clays to remolding be determined on the basis of strengths at equal strains rather than on ultimate strengths. The loss in strength by disturbance or remolding has been investigated by Moretto, who finds that some clays regain a considerable part of the lost strength in a relatively short time after cessation of disturbance, and who concludes that the changes in strength are caused in part by disturbance of the soil skeleton but to a greater extent by physico-chemical or thixotropic processes. In a paper in Section I, Burmister discusses the use of the results of unconfined compression tests in conjunction with those of triaxial

tests and surface loading tests for determination of both the bearing capacity and probable settlements of footings.

Direct shear tests are usually performed on square tests specimens and in single shear, often called box shear tests, but round test specimens and double shear are also used, especially on samples preserved in liners which are divided into very short sections or rings. The latter type of test is in some cases, but misleadingly, called ring shear tests. Direct shear tests, especially on consolidated test specimens, are easier and quicker to perform than corresponding triaxial tests and are used to a considerable extent, although not as much as formerly on account of the uncertain stress conditions during the test and because of the rapid development of triaxial tests in recent years.

Several papers describe new direct shear apparatus, but none of these represents any significant progress in elimination of the principal sources of error in this type of test. Geuze presents an analysis of the stress conditions during direct shear test in an attempt to determine the stress-strain relationships in pure shear and also a "secular" effect similar to that of secondary consolidation. Corrections for several sources of error in direct shear tests can be made, but the problem of transmitting the shearing force to an undisturbed sample without causing initial disturbance or excessive progressive failure of the soil has not yet been solved, and triaxial tests appear generally to be preferred for determination of the maximum shearing resistance of undisturbed soils which are brittle or sensitive to remolding.

Details of an apparatus for double shear tests on round test specimens are given in a paper by Proctor, and Bishop describes a direct shear apparatus for 1-ft square test specimens, primarily intended for testing of gravelly soils. In comparative tests on fine-grained soils, the results obtained with the 1-ft square and the usual 6-cm square apparatus were in good agreement.

A portable, direct shear apparatus for long rectangular samples, 2-cm wide and 25-cm long, has been developed by Steinbrenner. The soil is subjected to single shear, but two identical test specimens are used in order to avoid eccentric forces and simplify the apparatus. Results of tests on mixed sandy soil show for each type of soil a straight line relationship between the water content and the shearing strength when the former is plotted to an arithmetic and the latter to a logarithmic scale.

Mac-Neil Turnbull presents the results of direct shear tests on various soils and suggests a new method for evaluation of the cohesion. The details of the testing procedure and the method of evaluation are described in a handbook recently published by the same author. By means of a new direct shear apparatus and also the Hvorslev torsion shear apparatus, Ariano has investigated the relationship between the shear strength and the water content of various, partially saturated, remolded soils. With the water content increasing from zero to full saturation, it was found that both the cohesion and the apparent angle of internal friction first decrease to a minimum and then increase to an intermittent maximum at a water content approximately equal to the shrinkage limit.

Torsion shear tests are mentioned and some test results are reported in several papers, for example by Ariano and by Haefeli, but the equipment and testing procedures are not described in papers submitted to the Conference, and apparently new developments of this type of test have not been made in recent years. Torsion shear tests are generally performed on relatively thin, annular test specimens, although full circular test specimens occasionally are used. Some of the inherent sources of

error in direct, translatory shear tests are also to be found in torsion shear tests, and triaxial tests are generally to be preferred for determination of the maximum shearing resistance of undisturbed soils which are sensitive to remolding. However, the cross-section of the test specimen in the torsion shear apparatus remains constant throughout the test, and this type of apparatus presents the best available means for investigating plastic flow of soil and the decrease and subsequent partial recovery of the shearing resistance after the initial failure.

Triaxial tests are probably the most reliable and versatile of all strength tests. The external forces can be transmitted to the test specimen without causing initial disturbance of the soil; both the vertical and lateral stresses can be controlled or measured; and volume changes of and pore water pressures in the test specimen can be determined with greater accuracy than in other types of strength tests. Quick triaxial tests can be performed nearly as easily as direct shear tests, but triaxial tests in which full consolidation is to be attained before and/or during the test require considerable time, since the long test specimens must be drained through their end surfaces. Satisfactory methods for drainage along the cylindrical surface, at the same time permitting accurate determination of volume changes, are not yet available.

A great variety of triaxial testing apparatus has been developed, but the majority is of the open type, and the main differences are in methods of load application and measurement of forces and deformations. As a rule, the test specimen is brought to failure only once. In contrast thereto, the cell test, used extensively in Holland and Belgium, is made in a closed type apparatus, and a whole series of tests to apparent failure is performed on a single test specimen.

The papers submitted will be discussed in the following order: (1) Tests with open type apparatus and static load application; (2) tests with open type apparatus and dynamic load application; and (3) cell tests.

The following four papers treat mainly on testing procedures and methods of interpretation. Taylor conducted a series of undrained tests on three soils during which the pore water pressures were measured by a recently developed method, described in a separate paper by Lambe, and the corresponding effective or intergranular stresses were computed. After defining the critical and maximum shearing stresses, Taylor plots the ratio of the principal effective stresses and the ratio of the maximum shearing stress to the major effective stress as functions of the deformation, and finds that the resulting curves vary but little with the consolidation pressure. These curves can then be used for determining the shearing resistance of the soil in situ, provided the principal stress ratio in the ground can be estimated with reasonable accuracy. This can be accomplished when the soil is normally consolidated but only with approximation when considerable overconsolidation or precompression exists.

Skempton describes the evaluation of the results of a series of quick triaxial tests on fine-grained soils. The angle of internal friction is determined by the inclination of the planes of failure, and it was found that this angle decreases with increasing porosity of silts and clays. The pore water pressures are expressed as a function of the change in vertical and lateral stresses and of the ratio between the expansibility and compressibility of the soil. By estimating this ratio, the pore water pressures and thereby also the intergranular stresses and the cohesion can be computed.

Chen reports on tests on sand with a vacuum type of triaxial apparatus; that is variations in the lateral pressures are obtained by producing a partial vacuum in the test specimen instead

of subjecting the rubber membrane to excess air or liquid pressures, and considerable simplification of both apparatus and testing procedure is thereby obtained. Of major interest is the improved method of preparing the test specimens, using a special placement spoon and special tamper, by means of which unusual uniformity is obtained. It was found that the angle of internal friction increases with increasing angularity of the grains and increasing values of the Hazen uniformity coefficient, whereas the compressibility depends only on the angularity. The lateral strain increases at a greater rate than the axial strain, and the Poisson ratio is not constant.

Geuze determines the critical void ratio of dry sand by means of a vacuum type triaxial apparatus with special arrangement for measuring volume changes of the test specimen. The vertical load is determined by means of a hydraulic pressure cell, and it is suggested that the normal stress on the plane of failure, or for simplicity on the 45° plane, but not the vertical or lateral stress, should be kept constant while the shearing stress on this plane gradually is increased. The volume changes are plotted as a function of the shearing or deviator stress, and the critical void ratios are determined by special plots but conform to the criteria proposed by Casagrande.

The following three papers treat mainly on basic or regional research on soil properties. Golder and Skempton have investigated the shearing resistance, at constant water content, of many fully and partially saturated soils by means of undrained, quick triaxial tests. The angle of shearing resistance was found to be zero for fully saturated clays but greater than zero when the clays are only partially saturated. Angles greater than zero were also found for clay shales and siltstones and for some fully saturated silts. Wilson and Sutton present results of preliminary experiments on variations of the elasticity of sand. They show that the shearing stresses exert considerable influence on the elasticity and thereby on the Froehlich concentration factor and on the stress distribution in cohesionless soils. Florentin-L'Heriteau-Branda present a summary of tests on plastic clays from the Paris region. Both the vertical and lateral deformations of triaxial test specimens were measured by means of comparators. Good agreement of results of triaxial and torsion shear tests was obtained.

Two papers treat on the application of results of triaxial tests to practical design problems. Glynn investigates the properties of materials for an earth dam by means of a U. S. Bureau of Reclamation type of triaxial apparatus, discusses testing procedures, results obtained, and their application in design. McLeod uses triaxial tests for investigating the stability of granular and cohesive materials and discusses application of the data in design of bases for roads and airfields. It is pointed out that in case of cohesionless soils not only the angle of internal friction but also the lateral stress determine the stability, and the results of the tests are evaluated in terms of the deviator stress. Design formulas and graphs for cohesionless and mixed materials and for asphaltic concrete are presented.

Two papers describe triaxial test apparatus adapted for dynamic application of loads and discuss the results obtained. Casagrande and Shannon present a summary of the results of a large research project on dynamic properties of soils, undertaken for the Special Engineering Division of the Panama Canal. Transient loads were applied to the test specimen by means of a pendulum, falling beam, or hydraulic methods and in such a manner that either the impulse or the rate of strain could be controlled. The load and deformations were measured by means of electrical strain gages connected to a recording oscillograph. Time

of loading was varied from 0.01 second to 1000 seconds, and tests were made on one type of sand and three clays. The results show that the dynamic strength of clays for the shortest time of loading used is from 1.5 to 2.0 times the static strength, and that the dynamic modulus of deformation of clays is nearly twice as great as that obtained in static tests. On the other hand, the dynamic strength of sand is only 10 per cent greater than the static strength.

Bendel presents a photograph and summary description of a triaxial test apparatus by means of which both static and vibratory loads, the latter with frequencies up to 50 Hertz, can be applied. The loads are measured by electrical strain gages and an oscillograph. Tension can be applied to the test specimen by freezing the ends of the specimens to the pistons, and the vertical load can then be reduced simultaneously with an increase in the lateral load, and a stress condition resembling pure shear can thereby be produced. A photograph of a portable triaxial apparatus for use in the field is also shown. Formulas for application of the test results in estimating settlements of footings and piles under dynamic loads are presented. The movement of sand grains under footings subjected to static and dynamic loads was investigated by taking moving pictures of models placed between glass plates.

The cell apparatus, mentioned previously, is simple in construction and the cell test much quicker to perform than triaxial tests, since a complete series of tests to apparent failure is made on a single test specimen while it remains in the apparatus. For each increase in vertical load, liquid is slowly bled from the cylinder surrounding the test specimen until the lateral pressure reaches a stable minimum value. This pressure and the vertical pressure are treated as representing a condition of failure, and the corresponding Mohr circle is drawn. The test is then repeated for a new increase in vertical load. An extensive series of tests has been performed by De Beer for the purpose of comparing the results obtained by cell tests and unconfined compression tests. It was found that the strength obtained by cell tests is slightly lower than the unconfined compressive strength.

Pore water pressures in triaxial test specimens are being measured to an increasing extent, since knowledge of these pressures is required to determine the effective or intergranular stresses and since both the deformation and strength of soils are governed by the effective stresses rather than by the total stresses in the soil mass. In several triaxial test apparatus, for example that developed by the U. S. Bureau of Reclamation, the pore water pressures are measured by connecting porous disks at the ends of the test specimen to special piezometers or pressure cells. However, the pore water pressure in the center of the test specimen may differ from that at the ends and it should preferably be determined. This can be accomplished by inserting a hollow needle with a porous point diagonally into the sample and connecting it to a piezometer in which the water level is held constant by air pressure, thus preventing actual flow of water to or from the sample. This device is mentioned in the paper by Taylor and described in detail by Lambe. A considerable amount of data on pore water pressures in test specimens has been accumulated, but methods of computing or estimating the pore water pressures in the soil in situ must be developed before a fully rational application of the test results is possible. Many papers in other sections of the Proceeding contain records of measurement of pore water pressures in the field, but more research in both field and laboratory will be required before the data can be condensed and expressed in relatively simple and reliable rules.

Model Tests

Small scale models of prototype equipment and structures are often built in the laboratory for study of problems in basic research and also to obtain direct solutions of specific but difficult practical problems.

Flow of water in soils may be investigated by means of actual soil models, electrical analogy models, and by flow of a viscous fluid between closely spaced plates or through tubes of small diameter. Barron describes a model of the latter type, intended for solution of problems in consolidation of soils, and which also may be used for study of heat transfer and frost action in soils. The physical relationships between model and prototype are expressed mathematically, and the results of model tests on consolidation by strip loading are compared with those obtained by formal mathematics.

In other sections of the Proceedings Mansur and Perret describe investigations of the efficacy of sheet pile cutoffs by means of both soil and electrical analogy models. The results obtained by the two types of models are in good agreement and also verify the solutions obtained by means of flow nets and mathematically derived formulas. In individual papers Bendel and Jaky report on soil models used to investigate the settlement of rise of structures on account of changes in ground water levels and pressures.

Surface loading or plate bearing tests are used in both laboratory and field for study of the bearing capacity and settlements and foundation structures. Meyerhof investigates the bearing capacity of shallow footings on sand by means of model tests on very small plates and compares the results with the bearing capacity obtained by use of current design methods and formulas. Fairly good agreement was found to exist provided the maximum skin friction and the shearing resistance of the soil above footing level are taken into consideration.

Pollitt-Lewis-Lewis describe tests on small plates, simulating track-laying equipment and dynamically loaded by the falling beam method. It was found that the settlement increases and the unit bearing capacity decreases with increasing size of the plates, whereas the shape of the plates has but little influence. Fairly good agreement of the results obtained by model tests and design formulas is indicated provided the latter take perimeter shear into consideration.

Two papers describe photographic methods for study of soil movements below footings in model tests. Bekker verifies the formation of a permanent soil wedge below the footing and finds that, in general, the assumptions made in the design formulas by Terzaghi are correct. Peynircioglu investigates footings on flat and triangular fills of sand and finds that the sliding surface consists of a curved and a straight part. The former is practically a logarithmic spiral and can be replaced with a circle, but it is not justified to replace the entire surface of sliding by a spiral or circle. The sliding wedge is usually formed on only one side of the footing, and this should be taken into consideration in design. In general, the bearing capacity of and pressures created by the footings can be determined with fair approximation by means of available theoretical methods, provided a concentration factor greater than usual is used.

The following papers in other parts of the Proceedings may be mentioned. De Beer reports on investigation of soil reactions below a long beam or footing. Among other results, it was found that the commonly used assumption of parbolic distribution of the reaction pressures generally gives satisfactory results. The Belgian Communal Electric Society presents a brief account of both model and field tests on the bearing capacity of footings for transmission towers, and experiments on the stress distribution below wheel loads are described by Turnbull-Boyd-Fergus.

Pile loading tests by means of small models have been performed by several investigators for the purposes of studying the deformations and stress distribution in the soil around and below piles and thereby the validity of various formulas for computing the bearing capacity of piles. Only one paper on such tests is contained in Section II, in which the use of microphotography is described by Pogany. Impressions on soils, soil cement mixtures, and paraffin were made with a Vicat needle and a Brinnel sphere, whereupon sections of the material were cut out and microphotographs obtained by the methods used in petrography and metallurgy.

In Section VII, Tschebotarioff and Palmer describe extensive model tests for determination of the bearing capacity of piles and especially the relatively large loads carried by the point resistance and the skin friction. This load distribution was determined by means of electrical strain gages attached to the piles, and for piles in sand it was found that the total skin friction acting during load is four times as great as the skin friction determined by pull-out tests. Florentin-L'Heriteau-Fahri also investigate the amount and distribution of skin friction for piles in sand, find that it is influenced by the point resistance, and that the distribution is represented by a curve with a point of inflection at half the depth of penetration and does not follow a parabola as assumed in deriving some formulas for the bearing capacity of piles. Shilts-Graves-Driscoll describe field and laboratory tests for determination of the resistance of posts to lateral forces. They find the point of rotation to be at a depth corresponding to about two-thirds of the vertical area of the embedded section, and present formulas for computation of the permissible lateral load.

Earth pressure investigations by means of laboratory models are not described in Section II but in Section V, which contains several very important papers on this subject. A discussion of these papers is considered to be outside the scope of this review, but attention is called to the excellent papers by Tschebotarioff-Brown-Welch and by Jansson-Wickert-Rinkert, describing the results of tests with 6 to 8 ft high model walls for determination of earth pressures against flexible bulkheads and against retaining walls. In contrast to these large-scale models, Browzin and also Grador use models only 10 to 20 in. high and obtain valuable qualitative and even quantitative data in spite of the small model scale employed.

Photo-elastic model tests are used to determine the stress distributions in the soil below and around foundation structures and in and below earth structures. Such model tests are very useful when the foundation conditions are such that a rigorous mathematical solution of the problem is difficult or impossible to obtain. Philippe and Mellinger present a very valuable paper on the use of gelatin in photo-elastic model tests. The preparation, calibration, and loading of gelatin models are described in detail, and the results of photo-elastic determination of the stress conditions below the footing of a retaining wall are shown. In comparison with other materials, used for photo-elastic models, gelatin has the advantages that it easily can be cast in any desired shape, and that it has greater optical sensitivity to stress changes than other materials and is sensitive enough to allow evaluation of stresses caused by body forces. The results obtained by the model test apply only to fully elastic materials, and the influence of uplift and seepage forces cannot be determined directly by means of the model.

Soil Properties

Papers in Section II which deal with either very special or very general laboratory investigations of soils are

reviewed in the following paragraphs.

Structure of soils is the main subject of only one paper in Section II, in which Barber in a general manner discusses the influence of molding, loads, initial water content, and rate of drying on the structure acquired by remolded soils. Attention is called to an excellent paper by Grim in Section I, which contains a theory of soil structure based on the orientation of water molecules at the surface of clay minerals. The influence of various clay minerals and base changes is discussed, and it is emphasized that very small amounts of these minerals, of certain chemical compounds, and of organic matter absorbed by the clay minerals may exert a tremendous influence on the physical properties of the soil. Additional data on the same subject are contained in a paper by Winterkorn.

Special properties such as specific heat and thermal conductivity of soils is the subject of three papers by Kersten, one of which is found in Section I of the Proceedings. Details of equipment and procedures used in the experiments are described. It was found that the average specific heat of soil is about 0.19, and that the thermal conductivity of soils at optimem water content increases about 20 per cent by freezing. However, this difference in conductivity of unfrozen and frozen soils decreases with decreasing water content and is zero for dry soils.

Several other papers in Section I contain data on special properties of soils. Ruckli discusses the problem of heat flow to and the required insulation below floors of cold storage warehouses and presents results of model tests for verification of the design formulas. Winterkorn reviews the laws governing the transmission of daily and yearly temperature waves in soils and discusses the influence of thermo-osmosis and needed laboratory experiments and field observations. Haefeli and Amberg present the results of both theoretical and experimental research on the mechanics of shrinkage, demonstrate the influence of the initial water content of prepared soil mixtures, and emphasize the importance of these phenomena when soils are used for industrial purposes.

Regional surveys or reports on general laboratory investigations of soils from various localities are contained in several papers. Legget-Peckover describe Canadian silts, primarily from Steep Rock Lake, which often are mistaken for clay and are very sensitive ot remolding. Pogany reports on Silesian and Russian "Kurzawka', which is a fine sandy silt, often micaceous, and subject to disastrous slides. Strongman describes the great difficulties in sampling and testing of a boulder or chalk clay from Bedford, England, which is a hard and fissured clay containing numerous large chalk particles. Wilson presents the results of tests on a lumpy chalk with interstices filled with plastic chalk paste, found near Norwich, England. Skempton describes tests on samples from a deep stratum of soft, post-glacial clay near Gosport, England, and it was found that both the preconsolidation pressure and the shearing resistance throughout this stratum correspond closely to the depth and overburden pressure. Henry and Grace report on CBR and other tests on decomposed granite, resembling a sand clay, and found near Hong Kong. Detailed accounts of the properties of particular soils are given in several papers in other sections of the Proceedings, especially in Section III-f.

Correlations of the results of unconfined compression tests and cell tests are presented in a paper by De Beer, already mentioned in the section on triaxial tests. Correlations of the results of identification or classification tests with those of major laboratory tests are discussed in several papers outside Section II and are the main subject of a paper by McDowell. In one paper by Fadum and another by Parsons, the influence of disturbance

of the soil during sampling and preparation of test specimens on its consolidation and strength characteristics are discussed on basis of comparisons of the results of various laboratory tests and field observations.

Conclusion and General Trends

In an attempt to summarize some general trends in laboratory investigations and the practical application of soil mechanics, the writer believes that the following observations are warranted.

Much basic research, both experimental and theoretical, is in progress and is necessary for continued advance of soil mechanics, even though the results cannot always be applied directly to practical problems. Relatively speaking, theoretical and laboratory research predominated in Europe, whereas more large field tests and investigations are being undertaken in the United States.

With some exaggeration, it may be said that two general methods of approach exist in the practical application of soil mechanics. One method is to a large extent based on regional correlations and personal experience; only relatively simple laboratory tests, often supplemented or in part replaced with simple field tests and observations, are made and used in conjunction with semi-empirical methods of design. The other method consists in making detailed laboratory tests, taking more variables into consideration, and using advanced theories of design.

In the writer's opinion, these methods are complementary rather than competitive, and a meeting of minds of the proponents of the two methods of approach is developing. Experience and simple laboratory tests alone do not suffice when difficult and/or unusual conditions are encountered. On the other hand, the limitations of both testing and design are being realized. The soil conditions are seldom uniform; the laboratory tests and their results do not fully reflect field conditions, and even the most advanced theories of design are based on simplifying assumptions.

Advance field explorations and investigations are assuming increased importance, and greater care is being exerted in taking undisturbed samples, when such samples are needed. The use of allied sciences of geology, mineralogy, and colloid and physical chemistry to supplement laboratory investigations, and particularly to assist in identification and classification of soils, has increased materially in the last few years. In regard to strength test of soils there appears to be a fairly strong tendency to favor the triaxial test.

The need for improvement in the preparation of both undisturbed and remolded soils for testing is being realized. Recognition that for many soils the stress-strain characteristics of laboratory compacted samples may vary from those of field compacted soil is evidenced by considerable research in laboratory methods of soil compaction. In general, efforts are being made to devise laboratory equipment and methods of testing which will eliminate systematic errors; there is less tendency to use abstract and unproven theories of evaluation and design; and finally, the results obtained by laboratory testing and design theories are being verified or adjusted to an increasing extent by means of field observations during and after construction.

IMPROVEMENT OF THE MECHANICAL PROPERTIES OF SOIL BY MECHANICAL METHODS

A brief report on a part of Section IX of the Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering held at Rotterdam, The Netherlands, June 21-30, 1948.

By Frank A. Marston, Member*

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

The importance of the Second International Conference on Soil Mechanics and Foundation Engineering is evidenced by the registered attendance at Rotterdam of nearly 600, and a total of 420 papers submitted, all from 31 national groups.

The Proceedings of the Conference, in printed form, with pages 8½"×11" in size, are expected to occupy seven volumes, of which five have now been published.

The papers have been divided into 12 sections. This report is concerned with 14 of the papers in Section IX and one other paper of related interest dealing principally with the compaction of soil for rolled-fill earth dams and for the subgrades of pavements or other structures. Briefly, these papers deal largely with the development of better methods of laboratory and field control of soil compaction and the selection and operation of equipment to obtain the desired degree of compaction needed for stability and other characteristics.

Papers by R. R. Proctor (1) of Los Angeles give the results of tests on soils, varying from very sandy to very clayey, to show the relationships between compactive efforts of known amounts and certain soil characteristics such as sheer strength, density, penetration resistances, anticipated settlement, and others.

Comparisons between laboratory tests of compaction and field results obtained during the construction of six major dams in Southern California are offered to show that it is possible to apply the results of laboratory data to the selection and operation of sheepsfoot rollers to obtain a desired shear strength of soil to conform to the designed embankment slopes or pavement subgrade densities. The construction work involved the compacting of about 12,000,00 cu. yd. of earth fill. The compactive effort was measured by multiplying the draw-bar pull on the roller by the number of roller passes over each foot of compacted soil depth and dividing the product by the roller width.

After some years of experience with tests for control of soil compaction and moisture content, Proctor again recommends the use of the penetration needle for guidance in controlling the moisture content of soils before compaction, and the making of numerous compaction tests in the field by using a manually-operated rammer with the 1/20th cu. ft. cylinder. Just how satisfactory such a procedure would be with glacial soils containing stones up to four or five inches in size may be questioned.

Proctor points out that compacting a subgrade by means of a sheepsfoot roller leaves a loose surface of some 10 inches in depth which cannot be compacted satisfactorily by flat rollers,

*Partner, Metcalf & Eddy, Engineers, Boston, Mass.

but is subject to consolidation and settlement later on under the loads of traffic. The use of the method of overfill and trimming might obviate some of the trouble; or possibly, as suggested by G. Wilson (2) of England, compaction of the upper layers by rubber-tired wheels might solve the problem.

Dubose (3) reported the results of the comparison of the physical properties of an inorganic alluvial silt when compacted by laboratory and by field methods. But little difference was found in tests based on tri-axial shear, consolidation and permeability. Ultimate strength values varied only a little as determined by unconfined compression tests.

Zalazar (4) of Argentina, found that it was important to reduce the moisture content of the upper course of a pavement foundation to between 75 and 80 per cent of the plastic limit of the soil before the base course of the pavement was laid. Sufficient protection was provided by the pavement so that there was very little difference found in moisture content after several years of highway use. Zalazar suggested the use of "toughness index" instead of "plasticity index" for cohesive soils because the latter index "does not measure true cohesion."

Study has been given by a number of investigators to work out better methods of laboratory control of soil compaction.

Turnbull and McFadden (5) suggested that "serious consideration should be given to the development of a laboratory compaction procedure that will more accurately predict the behavior of all soils under field conditions."

Lane (6) described a laboratory test developed at Providence, Rhode Island, for compaction of cohesionless soils by vibration while confined by a surface load, giving the so-called "feasible maximum density," which is considered to reasonably represent the maximum density attainable in the field as well as the very high density produced immediately beneath a flexible pavement by traffic compaction. With this test it is unnecessary to screen out of the sample so much of the coarse aggregate, and there is less breakage of soft particles.

Turnbull (7) of Australia, developed a method of determinating the optimum moisture content and density for a soil mixture formed by addition of coarser or finer material to a soil, the moisture-density curve of which has been determined by test. This method will be of interest in a problem requiring the making of an artificial mixture of soil particles.

Studies for the San Valentino Dam in Italy, according to Wetter (8) have resulted in a plan to use a local sandy soil for the impervious core by mixing it with relatively small proportions of powdered gel-bentonite thus getting suitable water-tightness in an economical way. The rolled-fill earth dam as designed will be about 104 ft. high.

Maclean and Williams (9) of England reported on experimental work in the British Road Research Laboratory on five soils varying from heavy clay to a gravel sand-clay mixture.

A pneumatic-tired roller gave satisfactory compaction on all five soils whereas the smooth-wheel roller gave better performance on sandy soils. A sheepsfoot roller when operated on a sandy clay and on a clayey gravel gave better results with club feet than with tapered feet.

With a fairly soft chalk the best compaction was obtained with a 2¾ ton smooth-wheel roller followed by a vibrating screed.

This paper as well as others, emphasized the desirability, on important work, of making a test fill to determine the best type of compaction equipment to use, the number of passes, thickness of layers, desirable moisture content, and other factors. The information derived from such a test fill might even result in a change in design in order

to make use of local materials more economically or with greater stability.

The importance of such a test fill was further discussed by Philippe (10). He suggested that the selection of the type and loading of compaction equipment must be made to conform to the requirements of the local conditions of each particular problem.

Experience with the Crooked Creek Dam in Ohio, showed that under the local conditions a standardized compaction such as "six-inch layers, moisture control, and six passes of a sheepsfoot roller with 250 psi. rated pressure failed completely to produce the required soil densities for embankments higher than 80 feet."

Tests of compaction on subgrades of cohesive soils, described by Johnson and Maxwell (11) as clayey sand, clayey silt and silty clay, were carried out by U. S. Waterways Experiment Station using a 40,000-lb. rubber-tired wheel and a 1100-psi. sheepsfoot roller.

The wheel load was obtained by loading a 32 cu. yd. Turnapull to a total weight of 160,000 lb. or 40,000 lb. for each of the four wheels. The resulting load was equivalent to 69 lb. per square inch on the gross contact area. For these tests the 40,000-lb. wheel load was effective in increasing the density of the soil to a depth of about 2 ft.

The sheepsfoot roller had a drum 72 in. in diameter and 78 in. wide. There were 136 teeth each 18 in. long. The drum was loaded with Baroid and water. Maximum ground pressure was 1087 psi. The sheepsfoot roller caused an increased density to depths of over 50 in. or about 32 in. below the ends of the tamping feet. A loose surface layer about 18 in. in depth remained after the rolling, which required other means such as the rubber-tired wheel load, for compaction.

The results were materially affected by the moisture content of the soil. Within the limits of the tests and the numbers of passes of the equipment, a progressive increase in density was obtained with an increasing number of passes of both types of equipment.

The data given in a paper by Heiselman (12) were obtained during the building of the Merriman Dam, by the New York Board of Water Supply, involving several million cubic yards of rolled earth embankment.

The plastic, weaving behavior observed on the earth fill surface resulted in a temporary reduction in the relative shearing resistance of the soil caused by the expansive pressure of entrapped air in the voids of the soil. Intensive compactive effort expended while the soil was in this state did not produce additional densification.

Field compaction tests on a "clayey sand" (plasticity index 2), showed but small increases in maximum density with increase of pressure under the sheepsfoot roller and under increased rubber-tired wheel loads under constant contact pressure, according to a paper by Turnbull and McFadden (5).

With a "lean silty clay" (plasticity index 14), no such increase in maximum density was found using the sheepsfoot roller, but with the rubber-tired wheels there were small increases.

The sheepsfoot roller was a double-drum oscillating type. The contact area of each foot was 7 sq. in. Pressures of 250, 500, and 750 psi. were used in the tests. The drums were loaded with water and Baroid. The roller was towed by a tractor.

Rubber-tired wheel loads of 10,000, 20,000, and 40,000 lb. were obtained with heavy earth-moving equipment, but the contact pressure remained at about 65 psi.

The amount of rolling used was such as to give somewhat less than the Standard AASHO density under the lightest loadings, 250 psi. for the sheepsfoot, and 10,000 lb. for the rubber-tired wheels. The soil was compacted in 6-in. lifts.

The effects on soil density of increased pressures on roller feet and wheel loads were more noticeable in

the case of the clayey sand than with the silty clay. Increasing the load of the sheepsfoot roller did not create a proportionally greater effect on the soil density, possibly because the actual intensity of pressure on the soil was not as much greater as would be indicated by the increase in load, because the greater penetration of the feet caused more feet to come in contact with the soil.

At optimum moisture content of the clayey sand soil, the compaction obtained was about the same in the field with the sheepsfoot roller or the rubber-tired wheels as with the Standard AASHO test in the laboratory. The modified AASHO test was not as good an indicator as the Standard test.

With the silty clay the curves from field tests were closer to the zero air voids curve than those resulting from the laboratory test.

No significant change in density with either the sheepsfoot roller or the rubber-tired wheels was found in a given layer of soil as other layers were added. Each layer was compacted to such a degree that the effect of overlying layers and roller pressures did not cause a further compaction.

In the case of the silty clay, however, the field densities obtained were generally better than those indicated by the Standard laboratory tests. This indicated that with a given soil a correlation must be established between the laboratory tests and actual field results before predictions as to behavior can be safely based on the laboratory tests.

Of necessity these references have been brief. Undoubtedly some important matters have been omitted. The fact that some papers have been omitted should not be taken as any indication of lack of merit. It is hoped that attention may have been drawn to the Conference Proceedings as a whole, because they contain many papers of great interest and importance in the field of soil mechanics.

LIST OF PAPERS

Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, June 21-30, 1948

1. (Paper No. IXb 11, Vol. V) R. R. Proctor (U.S.A.) "The relationship between the foot pounds per cubic foot of compactive effort and the shear strength of the compacted soils"
   (Paper No. IXb 12, Vol. V) R. R. Proctor (U.S.A.) "Relationship between the foot pounds per cubic foot of compactive effort to soil density and subsequent consolidation under various loadings"
   (Paper No. IXb 14, Vol. V) R. R. Proctor (U.S.A.) "The relationship between the foot pounds per cubic foot of compactive effort expended in the laboratory compaction of soils and the required compactive efforts to secure similar results with sheepsfoot rollers"
   (Paper No. IXb 17, Vol. V) R. R. Proctor (U.S.A.) "Laboratory soil compaction methods, penetration resistance measurements and the indicated saturated penetration resistance"
   (Paper No. VIIIf 7, Vol. V) R. R. Proctor (U.S.A.) "The preparation of subgrades of compacted soils for paving of structures"
2. General Report on Section IX at Conference, by Guthlar Wilson (England)
3. (Paper No. IXb 13, Vol. V.) L. A. Dubose (U.S.A.) "A comparison of the physical properties of an alluvial silt compacted by field and laboratory methods"
4. (Paper No. IXb 2, Vol. II) L. M. Zalazar (Argentina) "Soil stabilization in Argentina highways"
5. (Paper No. IXb 15, Vol. V) W. J. Turnbull and G. McFadden (U.S.A.) "Field compaction tests"
6. (Paper No. IXb 4, Vol. IV) K. S. Lane (U. S. A.) "Providence vibrated density tests"
7. (Paper No. IXb 6, Vol. IV) J. MacNeil Turnbull (Australia) "Computation of the optimum moisture content in the moisture-density relationship of soils"

8. (Paper No. IXb 9, Vol. IV) G. Wetter (Italy) "Addition of Bentonite to the impervious core of San Valentino earth dam"
9. (Paper No. IXb 5, Vol. IV) D. J. Maclean and F. H. P. Williams (England) "Research on soil compaction at the Road Research Laboratory (Great Britain)"
10. (Paper No. IXb 8, Vol. IV) R. R. Philippe (U.S.A.) "Adaptation of locally available materials for use in construction of earth dams"
11. (Paper No. IXb 10, Vol. V) S. J. Johnson and A. A. Maxwell (U.S.A.) "Subgrade compaction tests with heavy rollers"
12. (Paper No. IXb 16, Vol. V) C. F. Heiselman (U.S.A.) "Air entrainment in compacted earth embankment"

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

NOVEMBER 17, 1948.—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, BS CE, was held this date at Northeastern University, Boston, Mass. Members of student chapters and civil engineering students of the New England colleges were especially urged to attend.

A catered dinner was held in University Commons Hall, Northeastern University from 6:00 to 7:00 P.M. Dinner reservations from the various colleges were as follows:

Northeastern University	40
Harvard University	25
Mass. Institute of Technology	17
Tufts College	32
Rhode Island State University	30
University of Maine	3
Yale University	1
University of Massachusetts	36

President Weaver called the meeting to order at 7:10 P.M.

President Weaver 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 Weaver introduced Miles N. Clair, President of the Northeastern Section, ASCE, and asked him to conduct any matters of business required by that Society.

President Weaver requested Mr. Oliver G. Julian, Chairman of the Structural Section, BSCE, to conduct any business required by that section.

President Weaver called upon Mr. Oliver G. Julian to introduce the speaker of the evening Mr. Ole Singstad, Consulting Engineer, New York, N. Y., who gave a most interesting illustrated talk on "Historical Development of Subaqueous Tunneling".

Three hundred and twelve members and guests attended the dinner and 347 persons attended the meeting.

The meeting adjourned at 9:30 P.M.

ROBERT W. MOIR, *Secretary*

DECEMBER 15, 1948.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts and Sciences, 28 Newbury Street, Boston, Mass.

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

Edwin F. Allbright who was elected a member November 18, 1908 and who died November 12, 1948.

Edwin G. Beugler who was elected a member May 17, 1905 and who died September 12, 1948.

John F. Shea who was elected a member June 21, 1944 and who died August 4, 1948.

Joseph A. Tosi who was elected a member January 27, 1915 and who died October 8, 1948.

The Secretary announced that the following had been elected to membership on November 15, 1948:

Grade of Student—Samuel Heyman.

President Weaver announced that

Honorary Membership in the Society had been conferred on one of the Society's distinguished members by vote of the Board of Government on March 24, 1948, to Edwin S. Webster, who became a member of the Society on February 19, 1902.

President Weaver stated that Mr. Webster has been unable to attend a meeting of the Society to receive the certificate of Honorary Membership and that on November 10, 1948, presentation of the certificate was made to Mr. Webster at his office by Frederic N. Weaver, President, Robert W. Moir, Secretary and Herman G. Dresser, Treasurer.

The certificate read as follows:

BOSTON SOCIETY OF CIVIL ENGINEERS

In recognition of a long and eminent engineering career in electrical engineering, especially in the establishment and management of public utilities

EDWIN SIBLEY WEBSTER

has been duly elected an

HONORARY MEMBER

By direction of the Board of Government

March 24, 1948

(Seal)

Edwin B. Cobb
Secretary

Harvey B. Kinnison
President

President Weaver requested the Treasurer Herman G. Dresser, to present a recommendation of the Board of Government to the Society for action. The President stated that this matter is before the Society in accordance with provisions of the By-Laws and notice of such action was published in the ESNE Journal dated December 6, 1948.

The Treasurer, Herman G. Dresser, presented the following recommendation of the Board of Government to the Society for initial action to be taken at this meeting.

MOTION—"That the Board of Government be authorized to transfer an amount not to exceed $5000 from the Principal of the Permanent Fund to cover unusual expenditures of the Centennial Year and to balance the Secretary's Budget at the end of the year."

President Weaver called for a discussion of this motion and a motion to cut the amount to $4000 was made and seconded. After further discussion a vote was taken and this motion was lost, the negative vote being many more than the required two-thirds majority.

The original motion was then put to vote and on motion duly made and seconded it was VOTED "that the Board of Government be authorized to transfer an amount not to exceed $5000 from the Principal of the Permanent Fund to cover unusual expenditures of the Centennial Year and to balance the Secretary's Budget at the end of the year". The motion was carried by more than the required two-thirds majority.

President Weaver stated that final action on this matter would be taken at the January 1949 meeting of the Society.

President Weaver introduced the speakers of the evening, who gave talks on the general subject, "The Jordan Marsh Building", as follows:

Speaker—William G. Perry of Perry, Shaw and Hepburn, Architects, Boston, Mass. Subject—"The Architecture of the Building".

Speaker—Maurice A. Reidy, Consulting Engineer, Boston, Mass. Subject—"Structural Features of the Building".

Speaker — Charles B. Spencer of Spencer, White and Prentis, Inc., New York. Subject—"Foundation Problems".

The talks were most interesting and were illustrated with lantern slides.

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

One hundred ninety-six members and guests attended the meeting.

The meeting was adjourned at 9:18 P.M.

ROBERT W. MOIR, *Secretary*

STRUCTURAL SECTION

NOVEMBER 17, 1948.—This was a joint meeting of the Boston Society of Civil Engineers; American Society of Civil Engineers, Northeastern Section; and the Structural Section of the Boston Society of Civil Engineers. After a dinner held in the Commons Hall at Northeastern University at which 312 attended, the meeting was called to order at 7:30 P.M., by President Frederic N. Weaver. As this was Student Night representatives of each of the New England College Student Chapters were called upon to rise as their schools were called.

Mr. Miles N. Clair, President of the Northeastern Section A.S.C.E. transacted the business of his section.

Mr. Oliver G. Julian, Chairman of the Structural Section, BSCE, gave the introduction of the speaker of the evening, Mr. Ole Singstad, Consulting Engineer and outstanding authority on Tunneling, of New York City. His subject was "Historical Development of Subaqueous Tunneling".

Mr. Singstad in his talk covered only vehicular tunnels constructed by the compressed air and shield method, as they are larger in diameter than railroad tunnels and have more construction problems. He first discussed the Holland Tunnel which was constructed in Hudson River silt of two tubes about 65 feet center to center and 8,400 feet between portals. One enlightening fact brought out was that a completed tunnel weighs less than the material it displaces and therefore there is no foundation problem except for ventilation buildings. Also the tunnel elevation must be such that it will have a protective blanket over the top and in the Hudson River the bed must be at least 50 ft. below mean low water. The ventilation was by means of air blown in under pressure in the space below the roadway and passing through openings to roadways. It was exhausted through openings in the ceiling to a duct above which was under negative pressure. This was to overcome the tremendous draft that might exist otherwise lengthwise of the tunnel.

It was brought out that the Holland Tunnel required 3,750,000 cu. ft. of fresh air per minute. The tunnel was opened November 12, 1927, had a capacity of 2500 vehicles per hour in each tube and 52,000 vehicles passed through in the first 24 hours. The cost of $48,400,000 divided equally between the states of New York and New Jersey was amortized from tolls in 13 years.

The Lincoln Tunnel was next constructed uptown at 38th Street under the Hudson River but has only carried a fraction of the vehicles of the Holland Tunnel probably because the New Jersey approach has considerable rise to get over a hill.

The Queens-Midtown Tunnel was constructed under the East River. This tunnel varied from the other two in that it had a brick floor instead of granite block and the ceiling was tiled. It was a much more costly project because the river bottom had gravel and boulders and other glasia deposits instead of river silt as in the Hudson River. It was opened in November 1940 and last year nine million vehicles passed through.

The Brooklyn-Battery Tunnel which is now under construction was next discussed. Some interesting features were the Ventilation Buildings. Also the dewatering shaft used at the Brooklyn end to lower the ground water level under the streets at the beginning of the tunnel section. This was done by sinking a steel shaft eleven feet in diameter and jacking out twelve radial perforated pipes. Only a few days pumping was necessary to lower the water the desired depth.

The talk which held the attention of

all present until the end was illustrated with many fine slides.

There were 347 members and guests present.

ARTHUR E. HARDING, *Clerk*

TRANSPORTATION SECTION

NOVEMBER 23, 1948.—Following dinner at the Parker House the Transportation Section met at the Society's rooms and heard a talk by Professor Dean A. Fales on "Style versus Safety" in which Professor Fales compared the current models of motor vehicles with those of earlier years, particularly with respect to style and safety. Professor Fales said that the motor vehicles produced as of about 1930 provided the maximum in safety features. He also discussed curves and grades of highways with respect to their bearing upon safe travel. There was much interest in Professor Fales' talk as evidenced by a long question period covering not only style and safety but also mechanical features of automobiles. The attendance was thirty.

WILLIAM L. HYLAND, *Clerk*

HYDRAULICS SECTION

NOVEMBER 3, 1948.—A meeting of the Hydraulics Section was held at the Society Rooms on this date, following a dinner at the Smorgasbord.

The meeting was called to order at 7:10 P.M. by Chairman John G. W. Thomas. During a brief business meeting the Chairman was empowered to appoint the last three chairmen of the Section as a Nominating Committee to present a slate of officers at the annual meeting in February.

The speaker of the evening was Robert T. Colburn, and the subject. "Design Features, Clark Hill Dam and Power Plant." Clark Hill Dam and Power Development is a 45 million dollar project now being constructed by the Corps of Engineers on the Savannah River about twenty miles above Augusta, Ga. Mr. Colburn who was in charge of design for Chas. T. Main, Inc., engineers for the project, gave a very interesting talk, illustrated with slides. After a discussion period, the meeting adjourned at 8:20 P.M.

Forty-two members and guests attended the meeting.

GARDNER K. WOOD, *Clerk*

SURVEYING AND MAPPING SECTION

OCTOBER 27, 1948.—The sixth meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:30 P.M.

Approximately 70 members and guests were present.

The meeting was called to order by Chairman Charles M. Anderson. The reading of the minutes was waived by consent of the group.

Chairman Anderson called upon Louis A. Chase, Chairman of the Committee appointed on January 21, 1948, to investigate and report on the desirability of requiring that surveys and plans prepared for the Land Court be made by a Registered Land Surveyor. The report of the Committee was read by Prof. Charles O. Baird, Jr., as follows:

> "The Surveying and Mapping Section on January 21, 1948, authorized the appointment of a committee to consider the advisability of requiring that surveys and plans prepared for the Land Court be prepared by Registered Land Surveyors.
>
> "A subcommittee interviewed the Board of Registration of Professional Engineers and Land Surveyors. The Board is opposed to compulsory registration.
>
> "A subcommittee interviewed one of the Judges and Engineers for the Land Court. The Court itself will not make a rule that surveys must be made by Registered Land Surveyors.
>
> "The majority of the Committee

finds that it is advisable that surveys and plans prepared for the Land Court be prepared by Registered Land Surveyors, but to obtain legislation when such action would probably be opposed by the Board and by the Court is impossible at this time.

"The Committee suggests that the Section mark time, as probably several bills will be filed by other organizations, and a future committee could be guided by the results of their efforts."

Louis A. Chase Charles O. Baird, Jr.
Russell H. Whiting Hyman B. Ullian
Thomas A. Appleton

Motion was made and seconded and it was VOTED to proceed to the election of a Nominating Committee to bring in a slate of officers to be voted on at the January meeting. The following members were elected:

Prof. Charles O. Baird, Jr.
Leonard S. Hubbard
Charles M. Anderson

Motion made by John Lowe, duly seconded, and it was VOTED that the Chairman appoint a committee of three members to keep the Section informed as to any legislative measures or recommendations of other Societies pertaining to Registered Professional Engineers or Land Surveyors.

There followed a show of a motion picture entitled "Topographic Mapping by Photogrammetric Methods", a sound-color film prepared by the Geological Survey. This film gives a detailed account of the use of the multiplex aero projectors making topographic maps from aerial photos. It is a 16 mm film with a running time of one and one-half hours, and some three-dimensional scenes.

Prof. Charles O. Baird, Jr., Prof. Herman J. Shea and L. T. Schofield led a spirited discussion which followed the showing of the film, testifying to the great interest in this subject.

Hugh P. Duffill announced that the speakers for the next meeting to be held on January 19, 1949, were to be Major William W. Drummey, prominent Boston architect and R. Newton Mayall, well-known landscape architect, their subjects to be, respectively, "What the Architect Expects of a Surveyor", and "What the Landscape Architect Expects of a Surveyor".

The meeting adjourned at 9:00 P.M.

HUGH P. DUFFILL, *Clerk*.

APPLICATIONS FOR MEMBERSHIP

[July 1, 1948]

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

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

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

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

For Admission

ARTHUR R. BARNES, JR., Reading, Mass. (b. July 28, 1908, Melrose, Mass.) B.S. in Civil Engineering, Worcester Technology in 1930. Experience, 1930-1932, Junior Engineer with Fred T. Ley Company; 1932-1936, miscellaneous jobs. Took post graduate

courses in design of structures; 1936-1938, draftsman with Consolidated Chemical Company; 1938-1946, draftsman and designer with J. R. Worcester Company; 1946-1948, Engineer with Thomas Worcester, Inc. During the last six years have assisted in the design of Warehouses, Mill Buildings, Apartment Houses, Garages, Hospitals, Piers, Churches, Bridges, Low Cost Housing and have been in charge of design of first class Hospital Building, steel and masonry woodworking factory, reinforced concrete and brick factory, several small bridges and two schools. At present Structural Engineer and Project Manager with Thomas Worcester, Inc. Refers to *W. T. Fairclough, F. K. Perkins, H. G. Protze, T. Worcester.*

JOHN T. BARNHILL, Wollaston, Mass. (b. May 11, 1914, Macquon, Illinois.) Graduate of Bradley University, Peoria, Illinois; majors in Chemistry and Mathematics, B.S. conferred 1936. Experience, while undergraduate employed full-time laboratory assistant 1931-1936; was Chemist for the Division of Sanitary Engineering, Illinois Department of Public Health, from 1936 to 1947, engaged in the laboratory analysis of water, milk, sewage and in industrial wastes, river surveys, teaching, report writing and research. Leave of absence from March 1943 to April 1946, to accept commission in the Sanitary Corps, A.U.S., with Sanitary Engineering M.O.S. number. June 1947 to date with Metcalf & Eddy as chemist, with duties of laboratory operation in the analysis of water, sewage, industrial wastes (chemical and biological), surveys, pilot plant operation and other functions related to consulting Sanitary Engineering. Refers to *A. J. Burdoin, S. E. Coburn, E. S. Chase, H. B. Allen, E. B. Cobb.*

LOUIS I. DEXTER, Pawtucket, R. I. (b. November 13, 1884, Valley Falls, R. I.) Graduate of Brown University class of 1906. Experience, July, 1906 to May, 1908, as Structural Draftsman for the American Bridge Company at East Berlin, Conn., Pencoyd, Pa., and Trenton, N. J.; May, 1908 to April, 1910, draftsman and checker for Post and McCord, Structural Steel contractors, New York City; April, 1910 to July, 1910, with the Bridge Designing Department of the New York Central and Hartford Railroad, New York City; July, 1910 to April, 1913, Chief Draftsman and Designer for Mulcahy and Gibson, Inc., engineers and contractors for structural steel, New York City; April, 1913 to December, 1913, Chief Draftsman for the Radley Steel Construction Company, New York City; December, 1913 to April, 1914, Structural Steel Designer for Milliken Bros., Inc., New York City; April, 1914 to March, 1915, draftsman for the Boston Bridge Works, Cambridge, Mass.; March, 1915 to January, 1920, Chief Draftsman for the James H. Tower Iron Works, Inc., Providence, R. I.; January, 1920 to December, 1945, shop superintendent and supt. of erection. December, 1945, was made construction engineer for the Tower Iron Works which is my present position. Refers to *H. R. Bliss, O. H. Horovitz, J. C. Moses.*

OTTO O. PASANEN, Ayer, Mass. (b. January 11, 1910, Kongin Kangas, Finland). Graduated from University of Maine in 1934, B.S. degree in Civil Engineering. Experience, November, 1928, to June 3, 1930, Hydro-Electric Project, Connecticut River Development Company, East Barnet, Vermont; September, 1935, to May, 1936, Sanitary Survey, Nashua River Valley Survey, Fitchburg, Mass.; June, 1936, to July, 1946, Water Supply and Sewage Treatment Works, Metropolitan District Water Supply Commission; Military Service, U. S. Army, October, 1943, to October, 1945; July, 1946, to date, Water Supply and Sewage Treatment Works, Assistant Civil Engineer,

Metropolitan District Commission, Construction Division. Refers to *K. R. Kennison, L. M. Gentleman, C. J. Ginder, R. W. Moir.*

KENTARO TSUTSUMI, Auburndale, Mass. (b. June 15, 1915, Honolulu, Hawaii). Graduate of the University of Hawaii in 1936, receiving B.S. degree in Civil Engineering. Experience, 1936-1937, Civil Engineer, Kahuko Plantation Company, Kahuko, Hawaii —surveys for irrigation and prevention of soil erosion, design and construction of earth dam and tunnels, design and construction of steel mill buildings; 1937-1938, Graduate Student, Massachusetts Institute of Technology, S.M. in Civil Engineering in 1938; 1938-1939, employed by LeRoy M. Hersum, Consulting Engineer, Boston, Mass., Bridge and Buildings Design; 1939-1940, Senior Civil Engineering Draftsman, Metropolitan District Water Supply Commission, Boston, Mass., design of structures appurtenant to Quabbin Aqueduct; 1940-1948, Project Engineer, Jackson & Moreland Consulting Engineers, planning and supervision of engineering groups in structural and machine design. Preparation of engineering reports. 1948, Special Lecturer, Northeastern University Graduate School, Boston, Mass., "Theory of Statically Indeterminate Structures". Refers to *E. H. Cameron, E. A. Gramstorff, O. G. Julian, R. W. Moir, E. L. Spencer.*

Transfer from Grade of Junior

JOHN G. JARNIS, Waltham, Mass. (b. September 9, 1920, Waltham, Mass.) Graduated from Northeastern University in 1943 with B.S. degree in Civil Engineering. Experience, 1943 to 1946. USNR, Engineering Officer aboard ship; 8 months' course with Certificate in Naval Architecture from the University of Michigan; Assistant Hull Ship Superintendent at Philadelphia Naval Yard; 1943 to date, employed by Thomas Worcester, Inc., as Civil Engineer including survey, layout, site planning, airport development and structural designer. Refers to *C. O. Baird, E. A. Gramstorff, F. Perkins, T. Worcester.*

Transfer from Grade of Student

FREDERIC M. CHILDS, Marston Mills, Mass. (b. April 7, 1923, Malden, Mass.) Graduated from Barnstable High School in June, 1941. Entered Northeastern University November, 1941. Called to Active Duty in the Navy July 1, 1943. Honorably discharged January 18, 1946. Reentered Northeastern University in March, 1946. Graduated with B.S. degree in Civil Engineering June, 1947. Experience, worked several months for Thomas O'Connor under supervision of Jackson & Moreland. This job was at the Edgar Station Edison Plant in North Weymouth. The nature of this work was field engineering and consisted primarily of Lines and Grades, and Quantity Estimates; September, 1947 to March, 1948, employed by Maurice A. Reidy. Did both drafting and design of commercial buildings; April, 1948, to date, with Engineering Sales Corporation as Sales Engineer. Much of my time recently, while with Engineering Sales Corp., has been development work in the field of industrial waste. Refers to *C. O. Baird, F. S. Gibbs, E. A. Gramstorff, R. P. Reidy.*

PROFESSIONAL SERVICES AND ADVERTISEMENTS

The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

Volume 36 JANUARY, 1949 Number 1

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROFESSIONAL SERVICES

INDEX TO ADVERTISERS

COMMITTEES

1948-1949

NOMINATING COMMITTEE

Past Presidents (Members of the Committee)

Athole B. Edwards
Dean Peabody, Jr.
William P. Hyland
Walter E. Merrill
(Term expires March, 1949)

Howard M. Turner
George F. Brousseau
Frank L. Cheney
William E. Stanley
(Term expires March, 1950)

Harry P. Burden

SPECIAL COMMITTEES

Program

Frederic N. Weaver, *Chairman, ex-officio*

Thomas R. Camp
William E. Stanley
Stanley M. Dore

Robert W. Moir
Charles M. Anderson
Allen J. Burdoin

Oliver G. Julian
R. Newton Mayall
John G. W. Thomas

Publication

Charles E. Knox, *Chairman*

John B. Babcock

Edwin B. Cobb

Leonard J. A. Fenocketti

Library

Robert W. Moir, *Chairman*

James F. Brittain
Elliot F. Childs

Edwin B. Cobb
George W. Hankinson

Ralph M. Soule
Henry I. Wyner

Thomas C. Coleman, *ex-officio*

Hospitality

Edward S. Averill, *Chairman*

Joseph C. Lawler

Charles T. Main, 2nd

James C. M. Tillinghast

Relations of Sections to Main Society

Oliver G. Julian, *Chairman*

Charles M. Anderson
Allen J. Burdoin

William J. Downey, Jr.

R. Newton Mayall
John G. W. Thomas

John R. Freeman Fund

Howard M. Turner, *Chairman*

Charles M. Allen

William F. Uhl

Desmond FitzGerald Award

Byron O. McCoy, *Chariman*

John B. Wilbur

Jack E. McKee

Sanitary Section Award

Jack E. McKee, *Chairman*

Harry P. Burden

George F. Brousseau

Structural Section Award

John B. Wilbur, *Chairman*

Henry Brask

Howard A. Gray

Transportation Section Award

Alexander J. Bone, *Chairman*

Francis T. McAvoy

Ernest L. Spencer

Hydraulics Section Award

Byron O. McCoy, *Chairman*

Elliot F. Childs

Harold K. Barrows

Surveying and Mapping Section Award

Charles O. Baird, *Chairman*

Alexander J. Bone

Chester A. Moore

Subsoils of Boston

Donald W. Taylor, *Chairman*

Miles N. Clair
Irving B. Crosby

Stanley M. Dore
Lawrence G. Ropes

Investment Committee

Herman G. Dresser, *Chairman*

Frederic N. Weaver

Everett N. Hutchins

Auditing Committee

George W. Coffin

Chester J. Ginder

Membership Central Committee

Frank L. Lincoln, *Chairman*

George F. Brousseau
Chester A. Moore

Francis H. Kingsbury
James F. Brittain

Robert W. Moir, *ex-officio*

Quarters Committee

Howard M. Turner, *Chairman*

Herman G. Dresser
Albert Haertlein

E. Sherman Chase
John B. Wilbur

Robert W. Moir, *ex-officio*

Building Laws and Building Construction

John R. Nichols, *Chairman*

Emil A. Gramstorff
Burtis S. Brown

Arthur L. Shaw
Thomas Worcester

Publicity Committee

Stephen Haseltine, Jr., *Chairman*

Jack E. McKee
Ernest L. Spencer

Llewellyn T. Schofield
James C. M. Tillinghast

Gardner K. Wood

Safety Committee

Herman G. Protze, *Chairman*

John R. Nichols

Henry I. Wyner

Insurance Committee

Frank L. Flood, *Chairman*

Donald W. Taylor

George A. Sampson

Competitive Bidding for Engineering Services

Ralph W. Horne, *Chairman*

John P. Wentworth
George W. Coffin

George G. Bogren
Herman G. Dresser

Changes in By-Laws

Frederic N. Weaver, *Chairman*

Thomas R. Camp

Carroll A. Farwell