White

JOURNAL of the BOSTON SOCIETY OF CIVIL ENGINEERS

1848 BOSTON SOCIETY OF CIVIL ENGINEERS

JULY - 1944

VOLUME XXXI NUMBER 3

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1944 - 1945

PRESIDENT

SAMUEL M. ELLSWORTH

VICE-PRESIDENTS

HARRY P. BURDEN
(Term expires March, 1945)

CARROLL A. FARWELL
(Term expires March, 1946)

SECRETARY

EVERETT N. HUTCHINS

TREASURER

CHESTER J. GINDER

DIRECTORS

EMIL A. GRAMSTORFF
JOHN B. WILBUR
(Term expires March, 1945)

ALBERT E. KLEINERT
HENRY A. MOHR
(Term expires March, 1946)

PAST PRESIDENTS

ALBERT HAERTLEIN
ATHOLE B. EDWARDS
HOWARD M. TURNER

SANITARY SECTION

FREDERICK S. GIBBS, Chairman
GEORGE C. HAUSER, Clerk

DESIGNERS SECTION

LAWRENCE M. GENTLEMAN, Chairman
FRANK L. LINCOLN, Clerk

HYDRAULICS SECTION

ALLEN J. BURDOIN, Chairman
HAROLD A. THOMAS, Clerk

HIGHWAY SECTION

CHARLES O. BAIRD, Chairman
GEORGE W. HANKINSON, Clerk

NORTHEASTERN UNIVERSITY SECTION

HENRY J. BISHOP, Chairman
PHILIP A. FRIZZELL, Clerk

Editor—EVERETT N. HUTCHINS
715 Tremont Temple, Boston

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume XXXI JULY, 1944 Number 3

CONTENTS

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

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 XXXI JULY, 1944 Number 3

AIRPORT PAVEMENT DESIGN

By A. H. Hadfield*

(Presented at a meeting of the Designers' Section of the Boston Society of Civil Engineers held on May 10, 1944.)

General

The problems involved in the design of airport runways are greatly simplified if the site selected is characterized by favorable soil and drainage conditions. However, there are factors such as location with respect to other industries, accessibility, area available for expansion and location with respect to established air routes which must be given greater consideration than favorable soil and drainage. Consideration and proper evaluation of all these factors will in many instances result in the selection of a location unfavorable from an engineering and construction standpoint. The designing engineer must, therefore, be prepared to deal with soils having low supporting power, poor drainage and susceptibility to frost heave.

The strength and durability of an airport pavement are to a large extent dependent on the character of the underlying foundation soils and the adverse effects of water and frost on these soils.

The pavement designs made by engineers of the Civil Aeronautics Administration's Federal Airways Service are based on a classification of the subgrade soil. Determination of the characteristics involved in this soil classification requires a careful and thorough field investigation and the selection of samples for tests.

*Assistant Chief, Airways Engineering Division, Civil Aeronautics Administration, Washington, D. C.

Field Investigations and Selection of Samples for Testing

The field investigations should be started early and no opportunity lost to inspect the site and adjoining areas frequently and particularly at periods when adverse weather conditions prevail. Although the behavior of soils under load and the adverse effects of water and frost can to some extent be determined in the laboratory, the relative position of the soil on which the pavement foundation will rest and the character of adjoining and underlying soil areas may result in conditions much worse than laboratory or even field tests indicate. It behooves all designers to get acquainted with the location early and avoid first impression conclusions.

Our engineers have used extensively the soil surveys prepared by the Department of Agriculture, information from geological surveys and to an increasing extent soil surveys made from aerial photographs. Such information is most useful as a general guide and background of information, but cannot be substituted for actual field investigations, including borings and test pits on the site.

During the field investigations, careful observations should be made of all soil horizons exposed in road cuts, ravines, and water channels as it is in such locations that soil layers in-place can be examined. Some soils can be roughly classified by texture, taste and smell.

Only after complete and thorough field investigations should soil samples be taken for laboratory tests. In general, these samples should be taken from the center lines of the taxiways and runways and from apron and parking area locations. If exceptional uniformity of conditions prevail, samples taken at intervals of 400 feet will suffice. However, as a general practice, taking of samples at 200 feet intervals is recommended. The method used for taking samples is not material so long as representative samples are secured from the top soil and underlying soil layers. An open pit large enough to enable the soil layers to be examined is an advantage and if soil sampling tubes or other similar devices are used for taking the sample, an open pit should be dug at critical points and a careful examination made of the soils in place. The manner of selecting and handling samples will, of course, be governed by the tests proposed and if unusual conditions exist such as a saturated subsurface strata

or silt or muck in which considerable settlement can be expected, undisturbed samples for consolidation and shear tests are desirable. Each site will present different problems requiring special attention in the field investigations and the important thing is to thoroughly cover the areas on which the taxiways, runways and related structures are to be located.

Sufficient samples should be taken for making a minimum of four complete soil analysis for each type of soil encountered. The laboratory and field tests conducted will be dependent on the methods to be followed in the design. Minimum tests required are mechanical analysis, liquid limit, plasticity index, volume change, capillary rise and California Bearing ratio.

Soil Classification and Evaluation of Supporting Subgrade Supporting Power

Due to the wide distribution of airports and landing field sites throughout the United States and Alaska and the wide range of soil types encountered, there has been an increasing need for a method of classifying natural soils or groups of soil in order of their value as subgrade supporting material. To meet this need the classification shown in Chart 1 has been devised. This classification is based on the mechanical analysis, plasticity characteristics, expansive qualities and California Bearing ratio of the soils. The values fixed for these properties are those that the soils would normally be expected to have. Most soils are found to fall into one group of the ten groups. Frequently the CBR values determined will place the soil in a different group than the other properties and in such a case the soil is placed in the lower of the groups in question or an interpolation made. A soil classification based on the physical characteristics, together with bearing tests, will provide a more proper rating of the soil than if the classification and evaluation were based on only one of these values.

The California Bearing Test was chosen because it measures the relative bearing power of the natural soil under conditions more nearly approximating the worst conditions than can be expected. Also, this bearing test is coming into wide use and correlation of the results of the tests and service records on a large number of airports is becoming available.

Soil	Material Passing #10 Sieve			Material Passing #40 Sieve			Capillary Rise of Minus 10 Material	Calif. Bearing Ratio (Soaked)	Subgrade & Subbase Classification			
	Sand %	Silt %	Clay %	Liquid Limit	Plasticity Index	Volume Change At FME			No Frost Good Drainage	Severe Frost Good Drainage	No Frost Poor Drainage	Severe Frost Poor Drainage
E-1	85+	0-10	0-5	25-	0-6	0-6	0-12	20+	F_a R_{1a}	F_a R_{2a}	F_a R_{1a}	F_c R_{2c}
E-2	75+	0-15	0-10	25-	0-6	0-6	0-36	20+	F_a R_{1a}	F_a R_{2a}	F_1 R_{1a}	F_2 R_{2a}
E-3	55+	10-40	0-20	35-	0-10	0-10	36+	18+	F_a R_{1a}	F_1 R_{2a}	F_2 R_{1a}	F_3 R_{2a}
E-4	55+	10-30	5-25	45-	5-15	5-15	36+	13-40	F_1 R_{1a}	F_2 R_{2b}	F_3 R_{1a}	F_4 R_{2b}
E-5	65-	20-75	0-20	45-	0-10	0-15	36+	9-20	F_2 R_{1a}	F_3 R_{2b}	F_4 R_{2c}	F_6 R_{2b}
E-6	55-	5-70	10-40	50-	10-30	10-30	36+	6-12		F_4 R_{2b}	F_6 R_{2b}	F_7 R_{2c}
E-7	55-	5-70	15-50	60-	15-40	20-40	36+	4-8		F_6 R_{2b}	F_7 R_{2c}	F_c R_{2c}
E-8	55-	5-50	30+	70-	20-50	30-50	36+	3-5		F_7 R_{2c}	F_8 R_{2c}	F_9 R_{2d}
E-9	55-	5-50	30+	80-	30-60	40-60	36+	2-4		F_8 R_{2c}	F_9 R_{2d}	F_{10} R_{2d}
E-10	55-	30-80	30-	60+	0-25	—	36+	1-3		F_9 R_{2d}	F_{10} R_{2d}	F_{10} R_{2d}

CHART 1.—SOIL CLASSIFICATION

It will be noted that like other soils classifications, there are two groups: the E-1 to E-4 "granular soils" containing 55% or more of sand and the E-5 to E-10 "non-granular soils" containing less than 55% sand (the E-5 has 65% or less sand). The granular soils are further divided into non-frost heave soils, Groups E-1 and E-2, and soils subject to frost heave, Groups E-3 and E-4. The E-1 soil is a free draining, non-plastic sand corresponding to the PRA classification A-3. The E-2 soil is a sand containing slightly more silt and clay than the E-1. The E-3 corresponds to a non-plastic and moderately plastic PRA A-2 type and the E-4 is equivalent to the PRA A-2 plastic type.

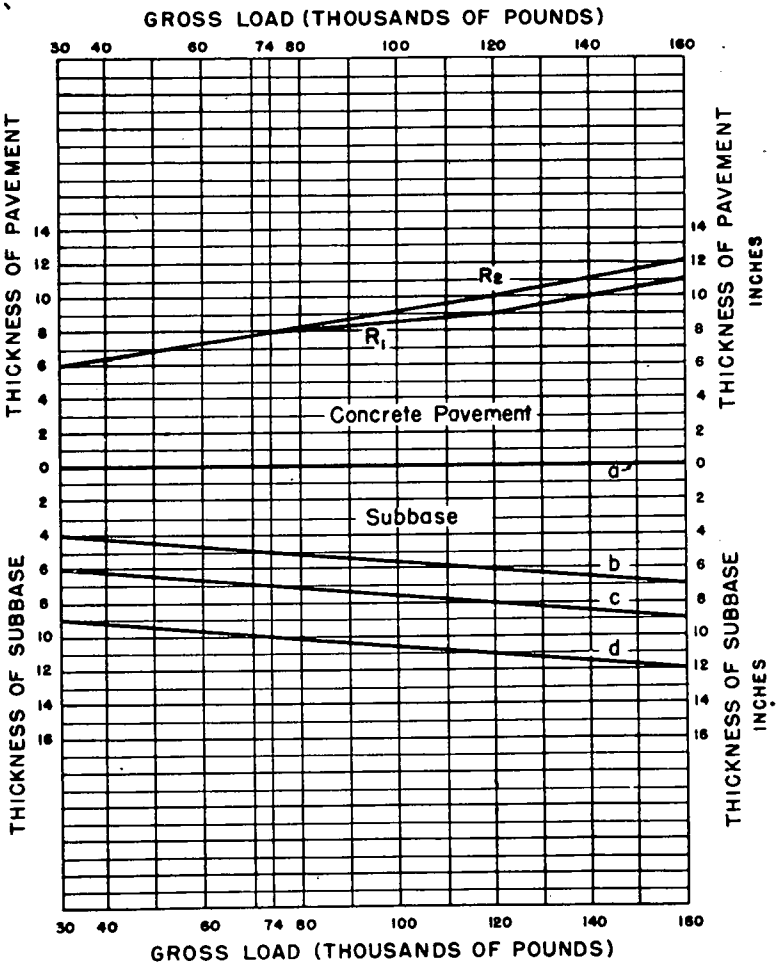
In the non-granular group the E-5 soil corresponds to the non-plastic or moderately plastic A-4 silt, and the E-6 to the more plastic A-4 silts, the A-4 and A-6 silty clays, and the A-6 or A-7 clays of low plasticity. The E-7, E-8 and E-9 groups include clay soils of average plasticity, high plasticity and very high plasticity, respectively, covered by PRA's A-6 and A-7 soils groups. The E-10 is the highly elastic soil classed as A-5 by PRA.

Since soils characteristics and supporting power can be so greatly affected by excessive moisture and frost, identifying symbols are used to reflect the soil properties under such conditions. These identifying symbols are shown in the last four columns on Chart 1. Thus, an E-1 soil is an estimate of a soil that requires no subbase under conditions of severe frost and poor drainage; and E-3 soil is an estimate of the poorest soil on which a base course could be placed under the most favorable conditions of no frost and good drainage.

This method of soil classification has been in use for a period of only 5 months and undoubtedly as additional information in assembled revisions and adjustments will be found desirable. Classification of soils on all new projects, as well as on airports and landing fields already constructed, will result in sufficient information so that eventually the soils classification values can be adjusted to meet our requirements for design.

Design Load

Having completed the field investigation and laboratory tests and determined the classification of the subgrade, it is next necessary to decide on the loading for which the pavement is to be designed.



CONCRETE PAVEMENT

CHART 2.

The size of aircraft has rapidly increased and planes are now operating which have wheel loadings of 60,000 pounds. At a tire pressure of 75 pounds per square inch and a tire rigidity factor of 1.1 the contact area would be 728 square inches for a 60,000 pound wheel load which is approximately equivalent to a circular area 30 inches in diameter. Larger planes are now in the design stage but definite information on gross weights and wheel loadings cannot be made public. Although use of dual wheels indicate that designers have reached the upper limit for single wheel loadings, it cannot be safely assumed that spacing of wheels will be sufficient to result in single wheel loads being critical. There is also a tendency toward use of higher pressure tires to reduce landing gear weight and space required for retraction.

The design load used by the CAA is 15,000 pounds for intermediate fields and limited use airports; 37,000 pounds for airports intended for average use and 60,000 pounds for major terminals. Although eventually heavy cargo and freight planes are likely to be restricted to airports with specially designed runways, it is certain that planes weighing upwards of 150,000 will be in use for transcontinental and transoceanic operations within the next five years. For major terminals, a minimum design loading of 60,000 to 75,000 pounds, based on single wheel, single tire loading is recommended.

Selection of Pavement Type

The selection of the type of pavement to be constructed may not be the responsibility of the design engineer but he will, most likely, be called upon for recommendations and for estimates of costs of alternate types. The advantage of rigid over flexible type pavements is a matter on which opinion seems to be fairly equally divided. Both types are being used from the Arctic to the Equator and are giving satisfactory service at some locations and failing at others. Most of the failures can be attributed to inadequate design, construction under adverse weather conditions, and waiving of specification requirements in order to meet completion dates or allow the use of local materials.

There are unquestionably locations on which a particular type of pavement will give better service, or equal service, at lower costs.

Local factors, such as availability of good quality materials, transportation, climate, subsurface conditions and other pertinent factors will, to a large extent, determine the cost. If there is no preference, it is usually advantageous to consider quotations on alternate types. Normally, the Federal Airways Service designs pavements of both flexible and rigid type and awards the contract on the basis of the low bids received.

The average in-place costs per square yard per inch of thickness of materials commonly used in the construction of airport pavements for the past three years are listed below:

Material	*Cost Per Sq. Yd. Per Inch Thickness*
Subbase (sand or bank run gravel)	$0.05
Caliche (used in Texas and New Mexico Areas)	0.06
Crushed stone or crushed gravel	0.14
Limerock (used in Florida no frost areas)	0.16
Sand-Asphalt	0.18
Bituminous Concrete—Road Mix.	0.22
Bituminous Concrete—Hot Plant Mix.	0.31
Portland Cement Concrete	0.32

Grading and drainage costs will not be materially affected by the type of pavement used. However, in repairing or reconstructing existing runways, where established grades must be adhered to, the type of pavement may largely determine the amount of grading required due to the difference of thickness required for rigid and flexible pavements on average soils.

Pavement Design

Before undertaking the design, it is often helpful to establish a clear mental picture of the structure under consideration and its components parts. An airport pavement is often mistakenly considered as a very simple structure. While made up of simple elements, namely, the surface, base and subbase, the principal components of these elements are soils. Soils, although the most commonly used material of construction, are very complex. Their behavior varies widely under load and the influences of water, frost, structure and other factors. Despite increased interest in soil mechanics, equipment and procedures have not been devised which can be used to determine the exact

supporting power of soil under the wide range of conditions existing. The result of this situation is that most engineers base the design of both rigid and flexible pavements on empirical or semi-empirical methods. The design of both types of pavements will be briefly discussed.

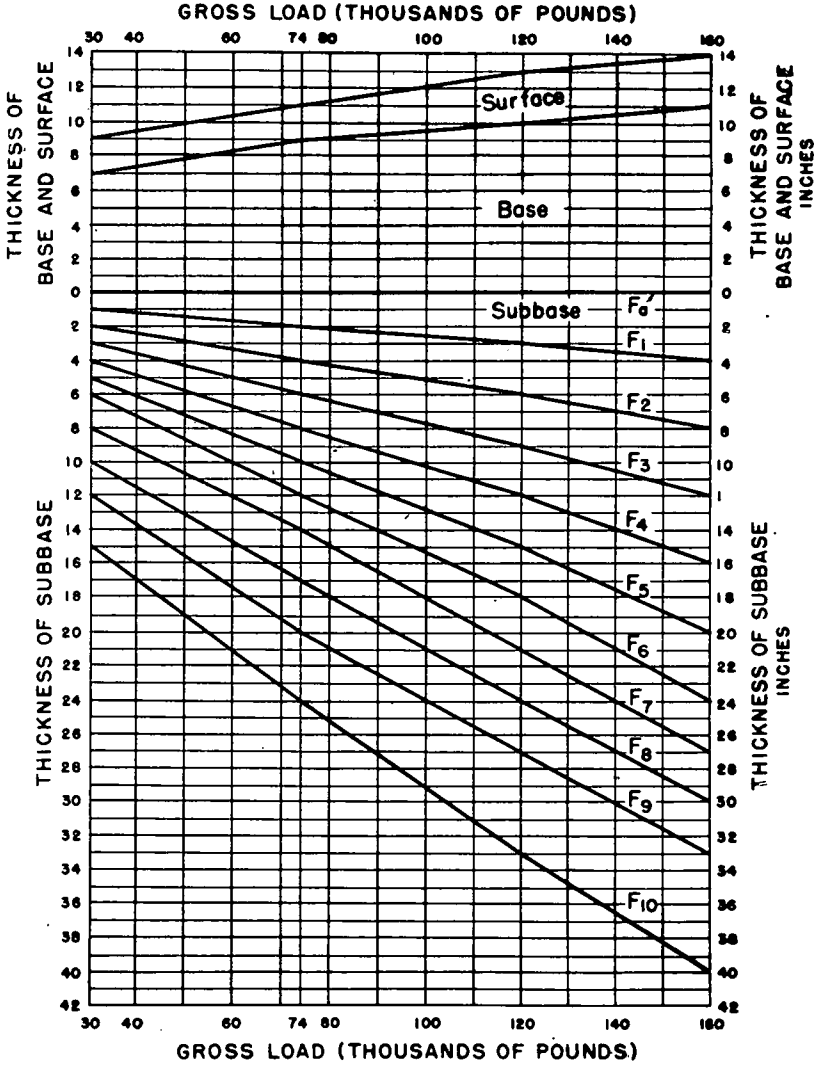
Flexible Pavement—In a flexible pavement design the elements of surface, base and subbase are logically treated separately, bearing in mind construction limitations and not losing sight of the fact that these three elements combined form the pavement structure, which acting as a unit support the imposed loads.

The surface course is the roof serving the primary function of shedding water and in addition provides a wearing surface. It should be sufficiently dense and stable to resist displacement under concentrated wheel loads and sufficiently plastic to retain life and waterproofing qualities. Asphaltic mixes are universally used for surface courses with hot mixed asphaltic concrete or a blend of natural asphalts being generally preferred. The thickness required varies with the materials used, location and design loading. The minimum thickness recommended is two inches.

There are two methods of approach in general use for designing the base and subbase for flexible pavements—(1) the semi-empirical method based on experience and service records, and (2) methods involving the use of plate bearing tests. The semi-empirical approach is used by the Federal Airways Service and consists essentially of (1) soils classification and evaluation of subgrade support, and (2) determination from "experience curves" the total thickness required for the design wheel load.

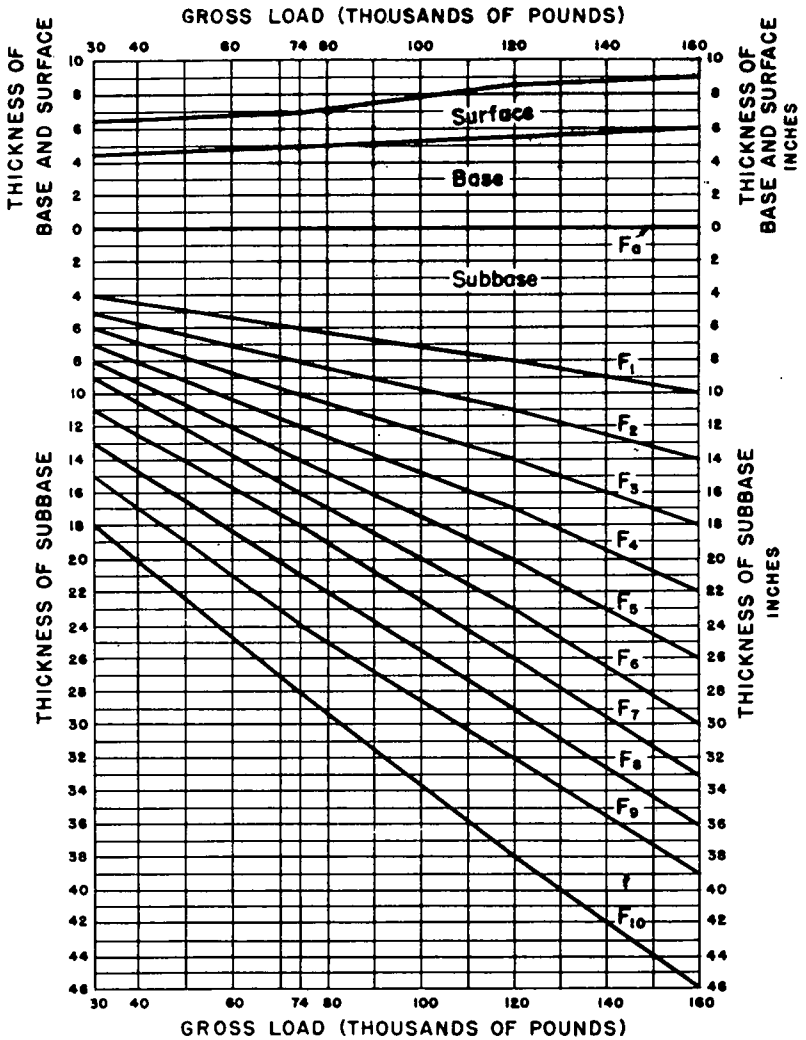
The method of classifying soil and the evaluation of subgrade support has been previously outlined. The "experience curves" used for determining the thickness of base and subbase, as well as over-all thickness are shown on Charts 3, 4 and 5. These curves are the result of our own experience in constructing pavements on some 200 airports and landing fields, studies of the Public Roads Administration's recommendations, the California, New Mexico and North Dakota Highway Department's published service data and reports of the Highway Research Board Committee on Flexible Pavement Design.

The failure of flexible pavements is generally due to excessive de-



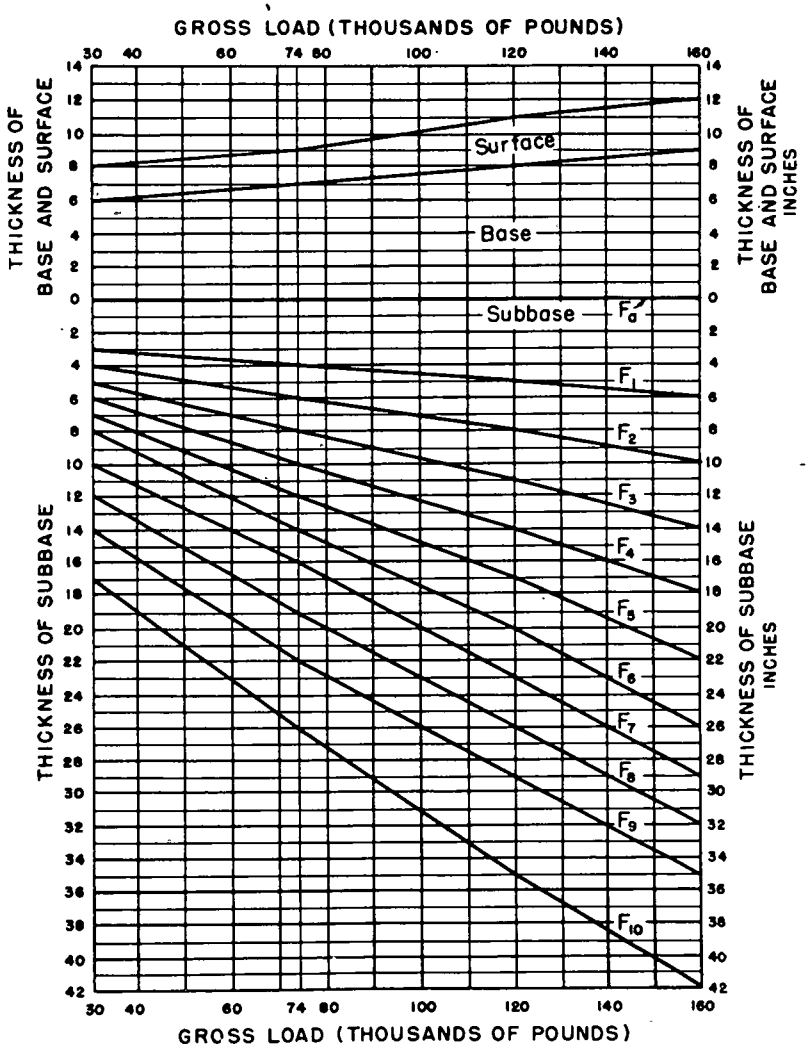
FLEXIBLE PAVEMENT
NON-BITUMINOUS BASE

CHART 3.



FLEXIBLE PAVEMENT
BITUMINOUS CONCRETE BASE

CHART 4.



FLEXIBLE PAVEMENT
EMULSIFIED ASPHALT AGGREGATE BASE

CHART 5.

formation which may be caused from excessive shearing stress in any of the pavement elements or subgrade, or from settlement of the subgrade due to compaction. The design should, therefore, take into account the type of subgrade which may consist of cohesionless sand or a cohesive soil. A theoretical analysis and experiments by the Public Roads Administration and others indicate that within limitations the settlement of a loaded circular area varies in proportion to the diameter of the contact area under the same unit load on cohesive soils. Likewise, on a non-cohesive soil (disregarding the "pile effect" occurring on small areas) settlement progressively increases but at a rate somewhat lower than the ratio of the diameter of the loaded area. Since pavements must be designed for both cohesive and non-cohesive subgrades, different extra-polation factors have been used for each case.

The curves shown on Charts 3, 4 and 5 are based on 30,000 pound gross loads, extrapolated to provide thickness of base and subbase varying in the approximate proportion of the cube root of the load for the granular non-cohesive soils approaching subbase in quality and in the approximate proportion of the square root of the load for the poorer cohesive soils. It will be noted from Charts 3, 4 and 5 that although the thickness of the base varies with the type of materials used in the base course construction the combined thickness of the base and subbase remains the same. Thus, a soil classified as E-6 under conditions of no frost and poor drainage would require a 21″ thickness of base and subbase combined for each of the three types of base course construction shown.

On all taxiways, aprons and runways used for taxiways and for a length of 500 feet on the end of all runways the thickness is increased 20%. This 20% increase in thickness is intended to result in a 40 to 50% increased supporting value on the areas of maximum traffic. There is ample justification for this increase in thickness as the repetition of loading on these areas is several times as great as on the runways proper. Such areas are subjected to loads under vibration due to warming up and checking of motors which from experience have been found to be more destructive than moving or static loads. Failure on airport pavements constructed to the same thickness over all areas usually start as localized failure at critical points on the aprons, taxiways and runway ends.

Rigid Pavements—In a rigid or concrete pavement design, the surface and base are combined and the structure proper can be considered as consisting of the rigid slab and the subbase. These two elements act together in supporting the loads and transmission of stress to the subgrade.

The design of the rigid pavement slab may be made either on an analytical or empirical basis. The methods of stress analysis, most of the experimental data and practically all of the service records of concrete pavement have been made for highway loads and highway conditions. A number of differences are found effecting the design of highway and airport pavements—(1) highways are subject to a large number of load repetitions with traffic cencentrated along the outside edge of the pavement where subgrade support is likely to be lowered due to saturation from the shoulders. On airport pavements the 100 to 300-foot paved widths result in much greater distribution of traffic and less concentration of loads, (2) impact loads which may run as high as two times the static loads on a highway are largely absent on airport pavements, (3) the range of loadings on airports is greater than on highways and the great majority of the planes are in the light weight class whereas on highways the vehicles cover a narrow weight range and a sizable proportion of the vehicles fall in the medium weight class. Due to these differences the highway design methods are used as a guide rather than being followed rigidly. The method used by the Federal Airways Service involves the design of the rigid slab on an analytical basis modified in certain respects to take into account these differences.

The three most important factors to be considered in the design of rigid pavements are—(1) the exterior loads, (2) the natural stresses in the concrete from temperature and moisture changes, and (3) the subgrade support..

The subgrade support is considered to be the most critical. To increase and maintain subgrade support, subbases are required under all concrete pavements unless the natural subgrade soil is of subbase quality required for the conditions of drainage and frost. Other advantages of subbase are that a layer of non-frost heaving and non-softening material is provided to counteract frost heave and pumping respectively. Use of subbase to increase subgrade support also

allows use of a thinner slab with lower temperature warping stresses.

Of the stresses due to natural causes, temperature warping stresses are the most serious. These were found by the Public Roads Administration to be as high as 200 to 400 lbs./sq. in. in the vicinity of Washington, D. C., depending on the length and thickness of the slabs. To keep these stresses a minimum, short slabs are used which are generally 15 feet or 25 feet in length.

For determining load stress we have used the Bradbury modification of Westergaard formula for interior thickness as recommended by the Public Roads Administration (P.R.A. July 1939).

$$\text{Stress due interior loading} = 0.31625 \frac{P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + 0.6330\right]$$

P = load in pounds
h = pavement thickness in inches
l = radius of relative stiffness
b = radius of equivalent distribution of pressure

The above formula obtains for:

Z	(Ratio of reduction of maximum deflection)	= 0.20 (average value)
E	(Modulus of elasticity for concrete)	= 5,000,000
V	(Poisson's ratio)	= 0.15
L	(Maximum value of radius of circular area within which redistribution of subgrade reaction is made)	= $5l$

A combined working stress of 500 lbs./sq. in. for load and temperature warping stresses is used for concrete of 700 lbs. per sq. in modulus of rupture at 28 days. One-third the computed temperature warping stress is used since the occurrence of maximum load and maximum temperature warping will be rather infrequent. This gives a working stress for load in the order of 350 to 425 lbs./sq. in. for 20-foot slabs, depending on the slab thickness and subgrade modulus. Our k values vary from a minimum of 150 (since subbase would be provided for the poorer subgrades) to a maximum of 300 for the best subgrades. Plate bearing tests are made on most sites to check the k value and adjustments may be made in the subbase thickness if found to be necessary.

Uniform thickness slabs are used on all rigid pavements eight

inches or greater in thickness. For slabs less than eight inches the edges are thickened to eight inches to provide space for keying. Heavy load transfers will occur at edges of dummy joints, and since slabs of uniform thickness are weaker at the edges, dowels are provided at dummy transverse joints which are not keyed. Dowels or other approved load transfer devises are installed at expansion and at construction joints. It is considered more economical to strengthen the slab at the joints by providing for load transfers than to design thickened edge slabs. There is also the advantage of having some steel to prevent possible faulting. For uniform thickness slabs the subbase can be placed and compacted to a uniform section without the reshaping and disturbance of the subbase as is required for thickened edge slabs.

The thickness curves for rigid pavements shown on Chart 2 have been developed on the above basis. The thickness shown for the heavy loadings (12,000 pounds and above) is somewhat less than actually calculated due to the probability that only a very small proportion of the planes using the runway will be in this weight class.

The thickness on runway ends, aprons, and all taxiways (which represent about 40% of the airport surface) is one inch greater than the thickness in the central areas of the runways. This greater thickness is one inch more than the values given on the design chart and is provided to allow for greater repetition of stress on these areas.

The methods discussed are believed to result in fairly conservative designs for both rigid and flexible type of pavement on a site in which the natural soils have been in place for a long period and consolidation in underlying materials is complete. For sites located on fills in which settlement has not reached equilibrium the best solution may be to construct a temporary pavement that can later be incorporated into a permanent pavement.

Our investigations of pavements have indicated, as would be expected, that the high moisture content of subgrade is the most serious cause of pavement failures. Near the plastic limit a small increase in moisture content will make the difference between failure and non-failure in an underdesigned pavement. Thorough compaction of the subgrade soils to a depth of 2 to 3 feet is highly advisable and every effort should be made to secure sufficient compaction so

that excessive settlement will not result from the loads of heavy aircraft. High requirements, 95% modified Proctor, are therefore specified for compaction of soils under the paved area. The performance of all soil subgrade materials is increased to a considerable extent by greater densities, and performance of sand, in particular, is increased a very large extent due to a slight increase in density.

The runway pavement structure should be protected to the fullest possible extent by providing a water tight surface course and an adequate drainage system. The drainage system should include provision for the immediate removal of all surface run-off from both the paved areas and adjacent shoulder strips. The shoulder strips should be graded in such a manner that run-off will be away from the paved area. We have found it advisable to install gutters and inlets for the removal of all run-off from paved areas except on locations where the subgrade soils are free draining granular type, or desert areas where little rainfall occurs. Where gutters and inlets are not provided the grades of the paved areas are raised 0.2 feet above the adjoining shoulder strips and a small fillet placed along this raised edge which results in the runway run-off being removed at increased velocity along the edge of the paved areas.

The behavior of our airports in service is being closely observed. Since little experience is yet available for heavy aircraft loadings, a number of traffic tests have been and are being conducted on completed pavements and on trial sections of pavement during construction. These tests have been proved to be very effective in determining the adequacy of our designs.

In conclusion, I would like to point out that the experience gained in constructing highway pavements and the service records of these pavements constitute the principal source of dependable information. The designing engineer should utilize this information to the fullest possible extent. Service records of many airport pavements on which medium weight planes (25,000 to 55,000 pounds gross) have been operating in numbers approaching the capacity of the airport will soon be available for guidance of design engineers. By the end of this year we shall have service records of pavements on which aircraft of 120,000 pounds gross weight have been operating.

There has been a tendency to underdesign, due to underestima-

tion with respect to both numbers and weight of aircraft. Lowering of specification requirements to allow use of available local materials has also resulted in inadequate pavements. These tendencies are being rectified to some extent but in many pavements little or no factor of safety has been provided. Since the cost of reconstruction of an existing pavement is likely to approach the original cost, greater factors of safety and use of only high grade materials would be in order.

HYDRAULICS OF THE PARK RIVER CONDUIT, HARTFORD, CONN.

By Scott Keith, Member*

(Presented at a meeting of the Hydraulics Section Boston Society of Civil Engineers held on May 3, 1944.)

General

The Park River conduit is part of the general flood protection works for the City of Hartford, Connecticut, and was designed and constructed under the direction of The District Engineer, U. S. Engineer Office, Providence, R. I.

The original project was included in the report on flood control for the Connecticut River and published in House Document 455, 75th Congress, 2nd Session. The project was authorized under the Flood Control Act, approved June 28, 1938.

During the flood of 1936 serious inundation of high value areas of the City of Hartford resulted from the backing of the Connecticut River into the low-lying valley of the Park River.

The purpose of this project is, therefore, to provide protection for that part of the City of Hartford adjacent to the Park River from backwater from the Connecticut River floods and also from floods from heavy rainfalls in its own drainage area.

The protection works for the Park River, as recommended by the Board of Engineers for Rivers and Harbors, was primarily designed for protection against backwater from Connecticut River floods, as modified by the twenty reservoirs of the Comprehensive Plan. This requirement would have been satisfied by an open channel with walled sides from the Connecticut River upstream to Bushnell Park or Hudson Street; above this point by a single wall following the north bank of the river to high ground near Asylum Street; and from Hudson Street to high ground along the southern edge of the Park by an earth dike to prevent flood waters of the Park River from reaching the protected area. Under this plan a large area of Bushnell Park

*With Metcalf & Eddy, Engineers, 1300 Statler Building, Boston, Mass.

would have been inundated during floods and the tops of walls would have been above the elevation of several streets which cross the river, requiring extensive grade changes in city streets.

For these and other reasons the City of Hartford proposed a closed pressure conduit from Bushnell Park near the State Capitol to the Connecticut River, in general located in the bed of the Park River. The general location is shown on Fig. 1.

Metcalf & Eddy were engaged by The District Engineer for the design of such a conduit. The City of Hartford agreed to bear the additional cost of a closed conduit over and above the cost of constructing the open channel as originally recommended by the Board of Engineers for Rivers and Harbors.

Proper design of the conduit required consideration of rainfall and runoff in the Park River drainage area, maximum flood discharge and maximum permissible water levels in Park River at the inlet to the conduit, and coincident flood stage levels in the Connecticut River.

Drainage Area

The drainage area of the Park River comprises:

At U. S. Geological Survey gaging station (Riverside Street) just below the junction of north and south branches	74.0 sq. mi.
At inlet of conduit	75.2 " "
At junction of Park River and Connecticut River	77.8 " "

Park River watershed lies for the most part westerly of Hartford. It is formed by two branches, the North Branch with a drainage area of 27.8 square miles and the South Branch having a drainage area of 45.9 square miles.

The western portion of the watershed is hilly, rising to a maximum elevation of about 900 ft. above sea level. The area includes six water supply reservoirs of the City of Hartford (tributary area 11.9 sq. mi. and total water surface area of 0.7 sq. mi.) which are now held in reserve, having been superseded by larger reservoirs elsewhere, and cannot be counted on to provide any appreciable flood control effect.

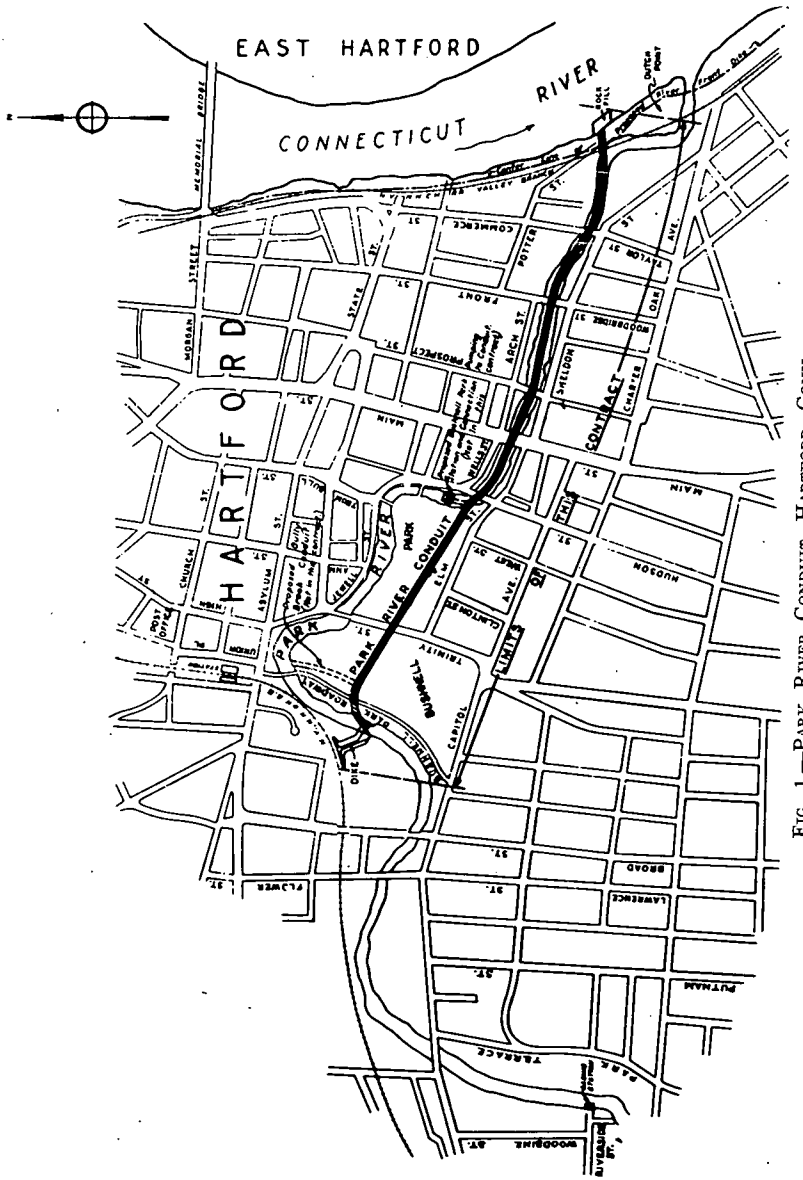


FIG. 1.—PARK RIVER CONDUIT, HARTFORD, CONN.

The remainder of the watershed is quite flat, particularly that part adjacent to Hartford. The branch streams have relatively slight slopes and winding channels.

Gully Brook, with an effective drainage area of 1,600 acres, enters Park River in Bushnell Park near Asylum Street. This is the only area of material size that will contribute storm water to the conduit below its inlet. This brook is now confined in a new conduit which has been extended to an elevation sufficient to prevent flooding by waters backing up this watercourse.[1]

Flood Flows

Studies of rainfall and run-off in the Park River drainage area were made by the U. S. Engineer Office at Providence, R. I., by the Flood Control Commission and the Department of Engineering of the City of Hartford and by the City's Consulting Engineers, Messrs. William F. Uhl and the late Charles H. Paul.

The maximum recorded rainfall in the general region is that of September 1938 (The Hurricane flood). The maximum rainfall over an area of 78 square miles (the drainage area of Park River) was 16.1 inches and occurred in a period of about 96 hours, an average of about 4 inches per day.

A study of available records of great rain storms in New England (prior to September 1938) indicates a maximum rainfall intensity for a 15-hour period of 0.72 inch per hour, average, or a total of 10.8 inches for the 15 hours.

Mr. Uhl estimated a flood flow of 17,500 cubic feet per second (equivalent to 233 c.f.s. per sq. mi.) by the B.S.C.E. Flood Committee formula[2] based on Cf (the flood coefficient, depending upon the flood characteristics of the stream) equal to 278, a drainage area of 78 sq. mi., and R (the run-off in inches on the drainage area) equal to 67% of the above-mentioned 10.8 inches of rainfall, or 7.25 inches.

Mr. Uhl, from a study of flood peaks which have occurred in New England rivers from drainage areas of 25 to 500 square miles in area (prior to September 1938), also estimated a flood peak of

[1]Engineering News-Record, Vol. 131, No. 25 (Dec. 16, 1943), p. 888.
[2]Report of Committee on Floods, Journal B.S.C.E., Sept. 1930.

18,300 c.f.s. equivalent to 244 c.f.s. per sq. mi. He concluded that the proposed conduit should be designed for 18,000 c.f.s.

Studies made by the U. S. Engineer office at Providence, using the unit hydrograph method of analysis, for three floods ranging from 5,380 c.f.s. to 850 c.f.s. showed close agreement, and when the composite unit hydrograph thus determined was combined with the maximum probable storm, assumed as a rainfall depth of 16.1 inches occurring in 48 hours (instead of 96 hours—the original record) resulted in a peak discharge of 19,700 c.f.s., equivalent to 262 c.f.s. per sq. mi.

These studies were reviewed by Metcalf & Eddy and the conclusions reached that in providing for a flow of 18,000 c.f.s adequate protection would be gained and the cost of such protection would be fully justified.

Maximum River Levels

Flood peaks in Connecticut River at Hartford, usually occur 3 to 4 days after heaviest rainfall, whereas the peak flow in Park River may occur in 24 hours or less. There appeared little likelihood of the simultaneous occurrence of high stages in the two rivers.

Data on stage and frequency of the Connecticut River at Hartford, prepared by the U. S. Engineer Office indicated that a stage of Elevation 26 may occur once in about 10 years and Elevation 27 once in 15 years (no allowance being made for the effect of the proposed reservoir plan). With the reservoir plan in effect the estimated frequencies would then become 45 and 70 respectively.

Considering all factors involved, it was concluded that the conduit should be designed to carry 18,000 c.f.s. with the water level at the inlet at Elevation 44 and with the Connecticut River at Elevation 26.

While a larger flood flow may occur at rare intervals the combination of a greater flow than that designed for together with a higher river stage than Elevation 26 is not likely to happen often enough to justify additional expense therefor.

Frequency of Flooding during Construction

From a study of the hydrographs of the Connecticut River from 1917 to 1938 it appeared that the water level in the Connecticut

might be expected to exceed Elevation 8 on the average of about 4 times during each working season. The contractor elected to carry the cofferdam at the outlet end of the conduit to Elevation 10. It was impracticable to build it much higher. Experience showed that this height was sufficient and the contractor flooded the cofferdam several times to save it.

Study of similar hydrographs of Park River for 1937 to 1939 indicated that rates of flow in excess of 400 c.f.s. might be expected to occur 7 times and in excess of 600 c.f.s. 5 times during each working season. The wooden flume built to handle the flow during construction had an estimated capacity of 900 c.f.s. The work was flooded out several times during the construction of the conduit.

Determination of Cross-Section

The conduit had to be of such interior cross-section as to permit passage of 18,000 c.f.s. with all hydraulic losses such as velocity head, entrance losses, friction loss, transition losses and losses in bends not to exceed the available head represented by the difference between the water level in Park River at the inlet and the corresponding stage of the Connecticut River or 18 feet.

It was decided to use the same interior dimensions throughout the entire length of the conduit, thus making it possible to utilize a single type of traveling form. It was believed that the economy of this feature would offset any possible advantage in changing the shape to meet special conditions. Moreover, any transition sections would introduce hydraulic losses.

In the selection of conduit dimensions the width was limited by the least available space between substantial existing structures which it was impracticable to remove; the width, nevertheless, had to be such as to fully utilize that space in order to minimize the height, thereby keeping the top of the conduit down to such elevation as would interfere as little as possible with the City's plans for a future roadway to be built on top of the structure.

Studies were made of twenty-nine different sections, including single-barrel, double-barrel and triple-barrel conduits of rectangular cross-section and with arched roofs and inverts. The single-barreled conduit resulted in a wide, shallow shape which was uneconomical

structurally. The three-barreled structure offered little or no advantage and the greater area made necessary by the hydraulic disadvantages led to embarrassment in the regions of restricted width.

The adopted cross-section, as shown on Fig. 2, consists of a

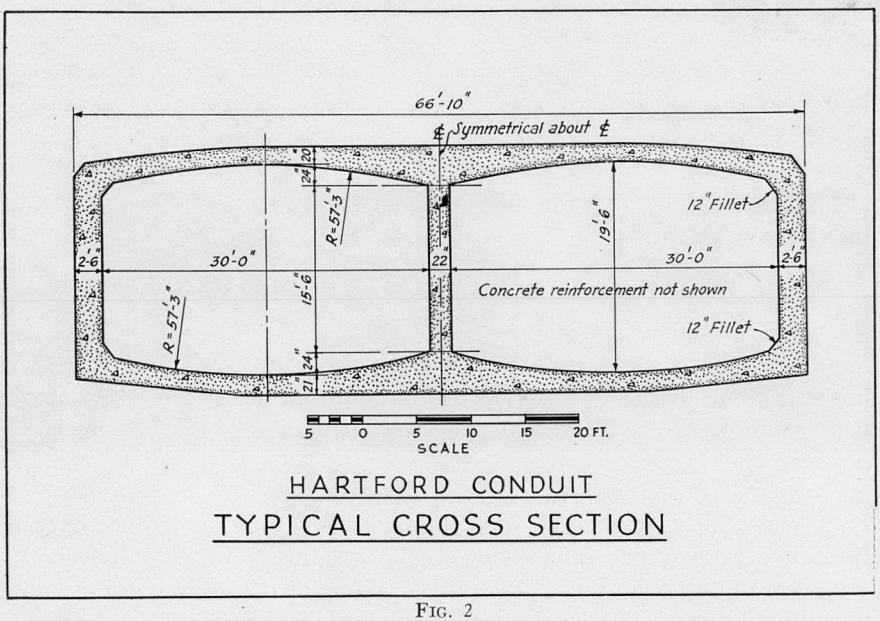


FIG. 2

double-barreled conduit of relatively wide and shallow proportions. Each barrel is 30 feet inside width, 19 feet 6 inches maximum inside height. Both invert and crown are flat circular curves with middle ordinate of 2 feet. Small fillets at the top and bottom of exterior walls are provided for structural reasons.

The cross sectional area of each barrel is 544.3 square feet, the wetted perimeter is 90.82 feet and the hydraulic radius is 5.99 feet. The velocity at the design rate of flow of 18,000 c.f.s. is 16.53 feet per second.

INLET ENTRANCE LOSSES

The inlet structure shown on Fig. 3, consists of parallel vertical walls conforming to the width of the old river channel to Elevation

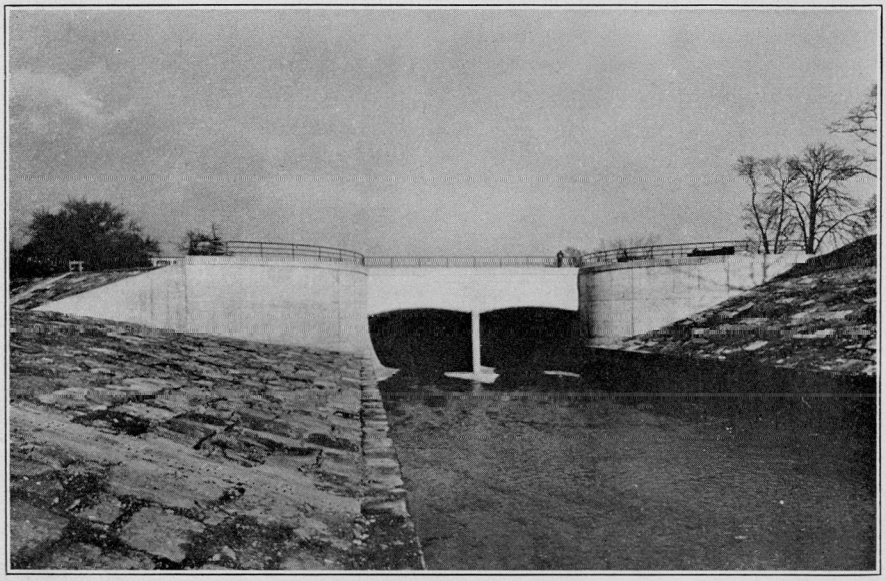


FIG. 3.—CONDUIT INLET

28.0. Cylindrical concrete wing walls, with radius of 40 feet extend from Elevation 28 to Elevation 51 and guide the water smoothly to the inlet of the twin-barrel conduit. The crown of the conduit inlet is rounded elliptically at the junction with the head-wall.

The maximum height of water in the pool upstream of the conduit inlet, under the requirements of the basis of design, is at Elevation 44.0 with a discharge of 18,000 c.f.s. The velocity in the approach channel at the entrance to the wing wall at this discharge is 7.73 feet per second, representing a velocity head of 0.93 feet. The velocity within the conduit at this same rate of discharge is 16.53 feet per second, which represents a velocity head of 4.25 feet. A study of the probable entrance losses resulted in adopting an entrance loss of 0.2 times the increment of velocity head or a total loss including the change in velocity head of 1.20 (4.25-0.93) or 3.98 feet.

FRICTION LOSSES

The specifications required metal interior forms, and the re-

moval of fins and offsets at form joints by chipping and carborundum stone, thus assuring smooth interior surfaces.

Friction losses were computed on the basis of a roughness coefficient $n = 0.012$ using the Manning formula, with separate allowance for losses in bends.

The total length of conduit is 5,594.83 feet. With the velocity of 16.53 feet per second corresponding to a flow of 9,000 c.f.s. per barrel, an hydraulic radius R equal to 5.99 feet, the required slope of the hydraulic gradient is equal to 0.00164, resulting in a friction loss of 9.18 feet.

Losses in Bends

A radius of 150 feet at the center line of the structure (that is. to the midpoint of the dividing wall) was used for all curves. Hydraulically the twin-barreled conduit is equivalent to a single pipe 33 feet in diameter. The ratio of radius of curvature to pipe diameter is, therefore, approximately 4.5, which is the ratio at which losses in bends are at a minimum, according to tests conducted at the University of Wisconsin[3] and also is not far from the ratio of 6 giving the minimum losses as determined by Hofmann.[4]

The hydraulic losses in bends were computed on the basis of a loss of 0.25 times the velocity head for a 90-degree bend and varying this loss for angles greater or less than 90 degrees by the square root of the deflection angle divided by 90, the relation used by the Metropolitan District of Southern California for large conduits.[5]

Thus, $H = K \dfrac{V^2}{2g}$, in which $K = 0.25 \sqrt{\dfrac{\Delta}{90}}$ where:

H = loss of head in feet,
V = velocity in feet per second,
Δ = deflection angle of bend in degrees.

The conduit has nine bends with various deflection angles ranging from a minimum of 4 degrees to a maimum of 99 degrees, the summation of all deflection angles being equal to 187 degrees 15 minutes.

[3]Bulletin of the University of Wisconsin, No. 578 (1913).
[4]"Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, p. 270.
[5]Trans. A.S.C.E., Vol. 100 (1935), p. 1033.

The resulting loss of head for all the bends was equivalent to 0.938 times the velocity head (4.245 feet) or 3.98 feet.

It is of interest to note that the combined friction loss and loss in bends (9.18 + 3.98 = 13.16 feet) results in an average equivalent roughness coefficient of 0.0144.

Summary of Losses

The losses previously discussed are summarized as follows:

Inlet loss and velocity head	3.98	feet
Friction losses	9.18	"
Losses in bends	3.98	"
Outlet loss (no recovery)		
Total	17.14	feet
Total head available	18.00	feet

The estimated discharge at a total loss of head of 18.0 feet is 18,440 c.f.s. If the water level at the inlet is at Elevation 48 and the Connecticut River level is at Elevation 29, thus providing a total available head of 19.0 feet, the estimate of The District Engineer that the maximum flood may be 19,700 c.f.s. is just met. At this rate there remains a freeboard of 2 feet on the inlet dike which is adequate in view of the limited wave action expected here.

General Hydraulic Behavior

Under the conditions assumed for maximum flow, the conduit will be submerged for its entire length and there are no velocity changes which require consideration except beyond the outlet, where scour is controlled by an apron of rock fill on the bed of the Connecticut River.

At moderate rates of flow in the Park River the conduit will flow part full at its upper end. Under these conditions shooting velocities will obtain in the upper reaches of the conduit and an hydraulic jump will result. The location of the jump will depend upon the stage of the Connecticut River and the rate of flow in the conduit.

The conditions of the hydraulic jump were investigated, not because of the effect on the maximum capacity of the conduit, but

because of the effect upon the structure when the jump might come in contact with the roof.

HYDRAULIC JUMP

General

Studies were made of the hydraulic jump for rates of flow in each barrel of 9,000, 7,500, 5,000 and 2,500 c.f.s. The computations were based on the following:

(a) All computations on backwater curves at the inlet end of the conduit were started at the control section located at the upstream end of the approach apron (50 feet upstream of the conduit inlet at Station 63 + 78.83) where the channel bed upstream from this section is at a relatively flat slope, and the invert of the entrance apron downstream from this section is at a steep slope of 0.05.

(b) Friction in the approach channel was computed by the Manning formula with $n = 0.0144$ (the same value of n as determined for the conduit when the loss of head in bends was included).

(c) In the portion of the approach channel where the invert is in transition between a flat invert and a twin dished invert, the area, wetted perimeter and hydraulic radius were computed for each section used in the backwater studies, this section being assumed as constant between the stretch ΔX and equal to that at the upstream section.

(d) At the inlet to the conduit the water surface was adjusted for the change in cross-section due to the obstruction of the partition wall between the twin-barrels of the conduit. Since the flow at this section is at shooting stage a *rise* in water level will occur in order to balance the energy gradient. Since the partition wall is well rounded it was assumed that the loss of energy at this obstruction would be of small moment.

It is believed proper at this point, before discussing the details of the computations involved in the hydraulic jump, to point out a warning which is best expressed by a quotation from Fred C. Scobey's paper entitled "Notes on the Hydraulic Jump":[6] "The exact location of the jump cannot be predicted. It can be computed, but the com-

[6]Civil Engineering, Vol. 9, August 1939, p. 468.

putations involve assumptions of the friction factor that can hardly be known with enough exactness to trace the water stages in either shooting or tranquil flow. Thus V_1 and V_2 are in doubt in the computations. This doubt is carried into the momentum curves used to indicate the location of the jump."

The computations in the hydaulic jump studies consisted of the following:

1. Backwater curve in the entrance channel.
2. Adjustment in water level due to the obstruction of the partition wall at the conduit entrance.
3. Determination of the normal, critical and conjugate depths for any particular rate of flow within the conduit.
4. Backwater curve from the Connecticut River.

For illustration the studies made for a rate of discharge of 9,000 c.f.s. are discussed herein. It should be noted that computations presented herein are based upon the design of the conduit as submitted by Metcalf & Eddy. Following the design, and prior to the construction of the conduit, a model of the inlet and outlet portions of the conduit was made and tests were conducted at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute by Prof. Charles M. Allen and Prof. Leslie J. Hooper. Based upon data in these tests the slope of the approach channel was slightly modified to eliminate the shooting velocities in this channel.

Backwater Curves

Backwater curves have been computed by the method devised by J. C. Stevens.[7] The computations for the backwater curve between the control section at the upstream end of the approach apron and the conduit inlet for a discharge of 18,000 c.f.s. are shown in Table 1.

In Table 1, Column (1) is the water depth (y) starting with the critical depth for a discharge of 18,000 c.f.s. in a rectangular channel 62 feet wide. Column (5) represents the specific energy (ϵ), the energy head measured above the channel bed. Column (10), S_f is the friction slope at the section commanded by (y). The remaining column headings are self-explanatory.

Attention should be called to the fact that it was found desirable to compute the velocities and velocity heads (Columns 3 and 4) to

[7]Trans. A.S.C.E., Vol. 102 (1937), p. 666; also Vol. 103 (1938) p. 990.

TABLE 1

BACKWATER CURVE IN ENTRANCE CHANNEL FOR Q = 18000 C.F.S.

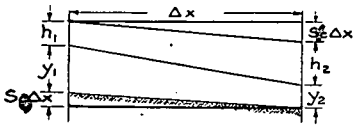
$q' = \frac{18{,}000}{62} = 290$ cfs/ft

$dcr = \sqrt[3]{\frac{(290)^2}{32.2}} = \sqrt{2610} = 13.8'$

$n = 0.0144$
ycr = 13.8±
Bed slope, So = 0.05

$S = \left\{ \frac{vn}{1.486\, R^{\frac{2}{3}}} \right\}^2$

y Ft (1)	A □′ (2)	V Ft/Sec (3)	$H=v^2/2g$ (4)	$\epsilon=y+H$ (5)	$\Delta\epsilon$ (6)	P (7)	R (8)	$R^{\frac{2}{3}}$ (9)	Sf (10)	Ave Sf (11)	Sf-So (12)	$\Delta X = \frac{\Delta\epsilon}{Sf-So}$ (13)	$\Sigma\Delta X$ (14)
13.8	855.6	21.038	6.873	20.673	...	89.6	9.55	4.501	.00205				
13.7	849.4	21.191	6.973	20.673	...	89.4	9.50	4.486	.00209	.00207	.04793	..	0
13.6	843.2	21.347	7.076	20.676	.003	89.2	9.45	4.470	.00214	.00211	.04789	.06	.06
13.4	830.8	21.666	7.289	20.689	.013	88.8	9.36	4.441	.00226	.00220	.04780	.27	.33
13.2	818.4	21.994	7.511	20.711	.022	88.4	9.26	4.410	.00234	.00230	.04770	.46	.79
13.0	806.0	22.332	7.744	20.744	.033	88.0	9.16	4.378	.00243	.00239	.04761	.69	1.48
12.8	793.6	22.681	7.988	20.788	.044	87.6	9.06	4.346	.00257	.00250	.04750	.93	2.41
12.6	781.2	23.041	8.244	20.844	.056	87.2	8.96	4.314	.00268	.00263	.04737	1.18	3.59
12.4	768.8	23.413	8.512	20.912	.068	86.8	8.86	4.282	.00282	.00275	.04725	1.44	5.03
12.2	756.4	23.797	8.793	20.993	.081	86.4	8.75	4.246	.00299	.00290	.04710	1.72	6.75
12.0	744.0	24.194	9.089	21.089	.096	86.0	8.65	4.214	.00309	.00304	.04696	2.04	8.79
11.9	737.8	24.397	9.242	21.142	.053	85.8	8.60	4.198	.00317	.00313	.04687	1.13	*9.92*
11.8	731.6	24.604	9.400	21.200	.058	85.6	8.55	4.181	.00325	.00321	.04679		10.00
NOTE: Downstream of this point the invert is dished. Area, perimeter and hyd. radius taken as of upstream section.													
11.8	731.6	24.604	9.400	21.200	.058	85.6	8.55	4.181	.00325	.00321	.04679	1.24	11.24
11.7	725.25	24.819	9.565	21.265	.065	85.28	8.50	4.165	.00334	.00330	.04670	1.39	12.63
11.6	718.88	25.039	9.735	21.335	.070	84.94	8.46	4.152	.00342	.00338	.04662	1.50	14.13
11.5	712.50	25.263	9.910	21.410	.075	84.59	8.42	4.139	.00350	.00346	.04654	1.61	15.74
11.4	705.92	25.499	10.096	21.496	.086	84.26	8.38	4.126	.00360	.00355	.04645	1.85	17.59
11.3	699.03	25.750	10.296	21.596	.100	83.94	8.33	4.109	.00369	.00365	.04635	2.16	19.75
11.2	692.01	26.011	10.506	21.706	.110	83.60	8.28	4.093	.00380	.00375	.04625	2.38	22.13
11.1	684.39	26.301	10.741	21.841	.135	83.20	8.23	4.076	.00392	.00386	.04614	2.93	25.06
11.0	676.35	26.613	10.998	21.998	.157	82.77	8.17	4.057	.00404	.00398	.04602	3.41	28.47
10.9	667.14	26.981	11.304	22.204	.206	82.28	8.11	4.037	.00420	.00412	.04588	4.49	32.96
10.8	656.33	27.425	11.679	22.479	.275	81.71	8.03	4.010	.00438	.00429	.04571	6.02	38.98
10.7	643.33	27.979	12.156	22.856	.377	81.02	7.94	3.980	.00465	.00451	.04549	8.29	47.27
10.65	625.43	28.780	12.862	23.512	.656	80.23	7.80	3.933	.00502	.00484	.04516	14.52	



$$S_o\Delta x + y_1 + h_1 = y_2 + h_2 + S_f\Delta x$$
$$\Delta x(S_f - S_o) = (y_1 + h_1) - (y_2 - h_2) = \Delta\epsilon$$
$$\Delta x = \frac{\Delta\epsilon}{S_f - S_o} \qquad \text{where} \quad \Delta\epsilon = (y_1 + h_1) - (y_2 + h_2)$$

three places beyond the decimal point in order that $\Delta\epsilon$ (Column 6) might be determined to two significant figures.

Table 1 shows that the water depth in the entrance channel will be reduced from the critical depth of 13.8 feet to 10.7 feet at the conduit inlet.

Adjustment of the water level due to the obstruction of the partition wall at the conduit entrance is shown in Table 2. The computations for Table 2 consist merely in assuming values for the water depth within the conduit, from which the area, velocity and velocity head may be computed. The specific energy ϵ at the conduit section is then the total of the depth and the velocity head, and since no loss of energy is assumed, should equal the specific energy of the approach channel just upstream of the partition wall. From Table 2 it is seen that the water depth increases from 10.7 feet to 11.35 feet within the conduit.

Hydraulic Characteristics of the Conduit Section

In order to analyze the hydraulic jump it is necessary to determine the normal depth, critical depth and conjugate depths for any given rate of discharge.

The normal depth is defined as the depth of water at normal flow, that is the idealized depth resulting from computations for uniform flow. It is best determined by assuming values of the depth (y) and from the determinations of area, wetted perimeter and hydraulic radius for the section under consideration, and the given bed or invert slope, the corresponding discharge may be computed. Such computations for the two invert slopes of 0.006 and 0.0007 are

TABLE 2—ADJUSTMENT IN WATER LEVEL AT CONDUIT ENTRANCE; Q = 18,000 C.F.S.

Approach Channel Q = 18,000 c.f.s.

d_1 = 10.7 ft. (See Table 1)
A_1 = 623.4 square feet
V_1 = 28.874 ft. per sec.
$V_1^2/2g = h_1$ = 12.946 ft.
$\epsilon_1 = d_1 + h_1$ = *23.646* ft.

Conduit Q = 9,000 c.f.s. per barrel.

d_2	A_2	V_2	h_2	ϵ_2
11.0	309.65	29.065	13.118	24.118
11.5	324.65	27.722	11.933	23.433
11.4	321.65	27.981	12.157	23.557
11.35	320.15	28.112	12.271	*23.621*

The energy gradient is in balance when y_2 = 11.35 and y_1 = 10.7.
Rise in water level is 0.65 ft.
Depth of 11.35 ft. is the normal depth with Q = 9,000 c.f.s. per barrel and n = 0.0144.

Velocity in Conduit Before Hydraulic Jump

d_1 = 11.35 ft. (See above)
A_1 = 309.65 + 0.35 × 30 = 309.65 + 10.5 = 320.15 sq. ft.

$$V_1 = \frac{9{,}000}{320.15} = 28.11 \text{ ft. per sec.}$$

Hydraulic Gradient With Conduit Flowing Full

V_2 = 9,000/544.3 = 16.535 ft. per sec.
R = 5.99; $R^{2/3}$ = 3.298
n = 0.0144

$$\text{Required slope } s = \sqrt{\frac{Vn}{1.486\,R^{\frac{2}{3}}}} = \sqrt{\frac{16.535 \times 0.0144}{1.486 \times 3.298}}$$

$$s = (0.048584)^{\frac{1}{2}} = 0.00236$$

Elevation of Hydraulic Gradient at Station 36 + 0

H_f = 2,866 × 0.00236 = 6.77
16.54 Elevated Crown at Outlet.
23.31

shown in Table 3, based upon a friction coefficient of 0.0144. A plot of the results shown in Table 3 is given in Fig. 4.

The critical depth is defined as that depth at which, for a given energy content of the water, the maximum discharge occurs, or the depth at which in a given channel a given quantity of water flows with the minimum content of energy. It is best determined by assuming values of the critical depth (y_{cr}) and from $V_{cr} = \sqrt{gA/B_w}$ computing the critical velocity and the corresponding discharge. Computations

TABLE 3

Discharge per Barrel at Normal Depth S_o = 0.0007 and 0.006

Depth of Flow Feet (y) (1)	Area Sq. Ft. (A) (2)	Wetted Perimeter Feet (P) (3)	Hydraulic Radius Feet (R) (4)	$R^{2/3}$ (5)	S_o = 0.0007, n = 0.0144 velocity Ft./Sec. (6)	Discharge c.f.s. (7)	S_o = 0.006, n = 0.0144 Velocity Ft./Sec. (8)	Discharge c.f.s. (9)
0.5	5.00	15.03	0.333	0.481	1.313	6	3.847	2
1.0	14.14	21.33	0.664	0.761	2.077	29	6.086	86
1.5	25.98	26.20	0.994	0.996	2.718	71	7.966	207
1.74	32.46	28.25	1.15	1.097	2.994	97	8.774	285
3.0	69.65	31.91	2.18	1.681	4.587	319	13.445	936
4.0	99.65	33.91	2.94	2.052	5.600	558	16.412	1,635
5.0	129.65	35.91	3.61	2.353	6.421	832	18.819	2,440
6.0	159.65	37.91	4.21	2.607	7.115	1,136	20.851	3,329
7.0	189.65	39.91	4.75	2.826	7.712	1,463	22.602	4,286
8.0	219.65	41.91	5.25	3.021	8.244	1,811	24.162	5,307
9.0	249.65	43.91	5.69	3.187	8.697	2,171	25.490	6,364
Mid 9.75	272.15	45.41	5.99	3.298	9.000	2,449	26.377	7,179
10.0	279.65	45.91	6.09	3.335	9.101	2,545	26.673	7,459
11.0	309.65	47.91	6.46	3.469	9.467	2,931	27.745	8,591
12.0	339.65	49.91	6.80	3.589	9.794	3,327	28.705	9,750
13.0	369.65	51.91	7.13	3.705	10.111	3,736	29.632	10,954
14.0	399.65	53.91	7.42	3.804	10.381	4,149	30.424	12,159
15.0	429.65	55.91	7.69	3.896	10.632	4,568	31.160	13,388
16.0	459.65	57.91	7.94	3.980	10.861	4,992	31.832	14,632
16.5	474.65	58.91	8.05	4.017	10.962	5,203	32.128	15,250
17.76	511.84	62.57	8.18	4.060	11.080	5,671	32.472	16,620
18.0	518.32	64.62	8.01	4.003	10.924	5,662	32.016	16,595
18.5	530.16	69.49	7.63	3.876	10.578	5,608	31.000	16,435
19.0	539.3	75.79	7.11	3.698	10.092	5,443	29.577	15,951
19.5 Full	544.3	90.82	5.99	3.298	9.000	4,899	26.377	14,357

$S = 0.0007 \quad n = 0.0144$

$$V = \frac{1.486}{0.0144}(0.0007)^{\frac{1}{2}} R^{2/3}$$

$$V = 2.729\ R^{2/3}$$

$S_o = 0.006 \quad n = 0.0144$

$$V = \frac{1.486}{0.0144}(0.0060)^{\frac{1}{2}} R^{2/3}$$

$$V = 7.998\ R^{2/3}$$

for the critical velocities are shown in Table 4 and results are plotted on Fig. 5.

For a given discharge in any channel there are always two depths of flow at which the sum of force due to velocity plus hydrostatic head at the respective cross-sections will be the same. These depths are called conjugate depths. To obtain the conjugate depths it is necessary to compute and plot the momentum curve for the particular discharge under consideration. The momentum curve may be computed from the expression $F_m = \frac{Q^2}{gA} + A\bar{y}$,[8] where Q is the discharge, A is the area of cross-section and $\bar{y}$ is the depth to center of gravity of the cross-section. The computations and plot of the momentum curve are shown in Table 5 and Fig. 6 respectively. It

[8] "Handbook of Hydraulics" by King, 3rd Edition, p. 377; also "Hydraulics of Open Channels" by Bakhmeteff, p. 234.

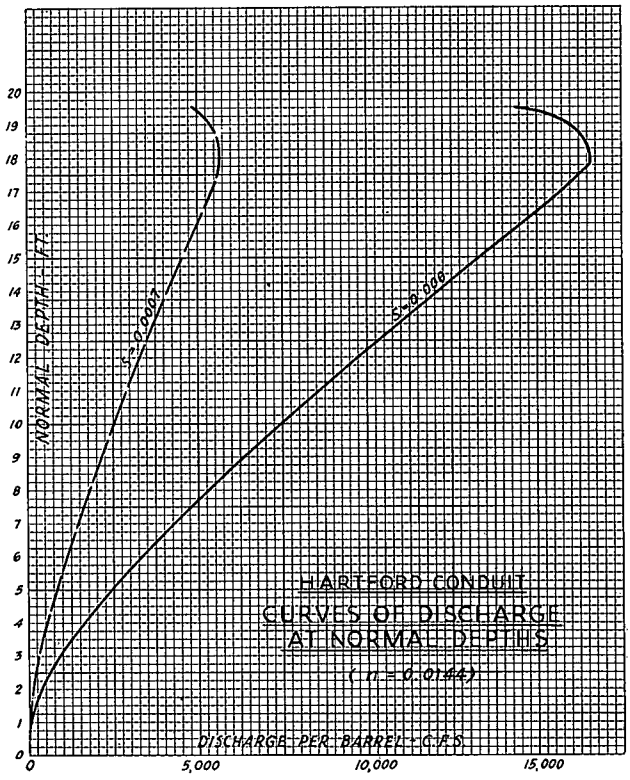


FIG. 4

is seen from Fig. 6 that with an initial depth of 11.35 fee the conjugate depth, or the depth after the hydraulic jump is 18.8 feet.

The specific energy curve is useful not only as a means of determining the critical depth for any given rate of flow but also to determine the energy dissipated in the hydraulic jump. The specific energy is the total of the potential and kinetic energy heads referred to the bottom of the section and is represented by $\epsilon = y + v^2/2g$. Table 6 shows the computations for deriving the specific energy for a discharge of 9,000 c.f.s. per barrel. Fig. 7 gives the specific energy curves for the various rates of flow used in the studies.

In Fig. 7 the depth at minimum specific energy is the critical

TABLE 4
PRESSURE CONDUIT FINAL DESIGN

Discharge at Critical Depth
(Applies to both upper and lower stretches regardless of S_o)

Assumed Critical Depth Feet (y_{cr})	Area Sq. Ft. (A)	Top Width Feet (B_w)	Critical Velocity at Critical Depth Ft./Sec. $V_c = \sqrt{gA/B_w}$	Discharge c.f.s. per Barrel $Q = A V_c$
(1)	(2)	(3)	(4)	(5)
0.5	5.00	15.0	3.27	16
1.0	14.14	21.2	4.64	66
1.5	25.98	26.0	5.67	148
3.0	69.65	30.0	8.65	604
4.0	99.65	30.0	10.35	1,030
5.0	129.65	30.0	11.79	1,530
6.0	159.65	30.0	13.08	2,080
7.0	189.65	30.0	14.25	2,700
8.0	219.65	30.0	15.38	3,370
9.0	249.65	30.0	16.38	4,080
10.0	279.65	30.0	17.32	4,840
11.0	309.65	30.0	18.20	5,630
12.0	339.65	30.0	19.05	6,460
13.0	369.65	30.0	19.90	7,350
14.0	399.65	30.0	20.7	8,270
15.0	429.65	30.0	21.45	9,200
16.0	459.65	30.0	22.2	10,200
18.0	518.32	26.0	25.4	13,150
19.0	539.3	15.0	34.0	18,300

depth; the potential energy head is represented by the horizontal distance from the Y axis to the $Q = 0$ line (the line at 45 degrees with the horizontal) and the kinetic energy head is represented by the distance from the $Q = 0$ line to the specific energy curve. The zone of flow at shooting stage is that lying below the line of critical depth and the zone of flow at streaming stage is that lying above the line of critical depth.

The particular conditions of the jump obtaining with a flow of 9,000 c.f.s. may be summarized as follows:

Item	Before jump	After jump
Depth	$d_1 = 11.35$ ft.	$d_2 = 18.8$ ft.
Velocity	$v_1 = 28.11$ ft./sec.	$v_2 = 16.81$ ft./sec.
Velocity head	$v_1^2/2g = 12.27$ ft.	$v_2^2/2g = 4.38$ ft.
Specific energy	$\epsilon_1 = 23.62$ ft.	$\epsilon_2 = 23.18$ ft.

The energy dissipated by this particular hydraulic jump is, therefore, the difference between ϵ_1 and ϵ^2, or 23.62 — 23.18 or 0.44 ft. This may also be obtained approximately from Fig. 7.

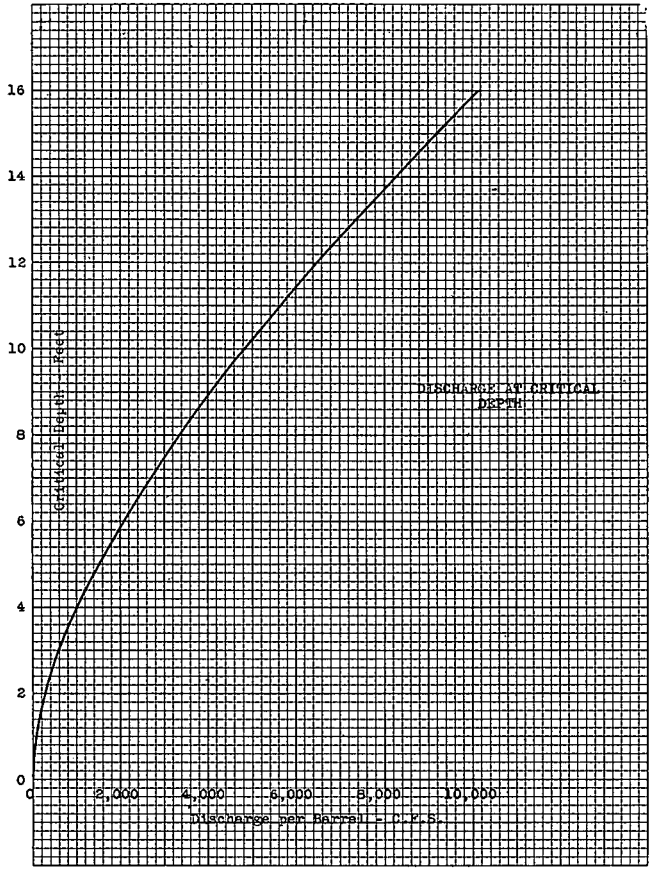


FIG. 5

LOCATION OF THE HYDRAULIC JUMP

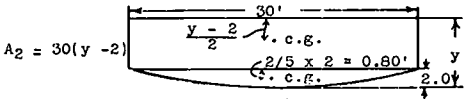
From Fig. 5 it is seen that with a flow of 9,000 c.f.s. per barrel the critical depth is 14.8 feet; from Fig. 4 the normal depth at this rate of flow is 11.4 feet. From Table 2 it is determined that the depth at the entrance to the conduit is 11.35 feet. It is, therefore, apparent that at this rate of flow the depth will remain at the normal depth until the hydraulic jump occurs, when the depth will increase from 11.35 feet to the conjugate depth (Fig. 6) 18.8 feet.

TABLE 5

Momentum Curve. Q = 9,000 c.f.s. per barrel.

Depth Feet	Area Sq. Ft.	Q^2/gA $g = 32.2$ $Q^2/g = 2{,}516{,}000$	Ay $= 15y^2 - 20y + 12$ *	$F_m = \frac{Q^2}{gA} + Ay$
(1)	(2)	(3)	(4)	(5)
10	279.65	9,000	1312	10,312
11	309.65	8,130	1607	9,737
12	339.65	7,410	1932	9,342
13	369.65	6,810	2287	9,097
14	399.65	6,300	2672	8,972
15	429.65	5,860	3087	8,947
16	459.65	5,490	3532	9,022
17	489.65	5,140	4007	9,147
18	519.65	4,850	4512	9,362
19	549.65	4,580	5047	9,627
20	579.65	4,340	5612	9,952

* Approximate (to the extent that 12" fillet is disregarded)



Assume a parabola $A_1 = 30 \times 2 \times 2/3 = 40\square'$

$$Ay = 30(y - 2)\left(\frac{y - 2}{2}\right) + 40(y - 2 + 0.8)\left[30(y - 2) + 40\right]$$

$$Ay = 15(y - 2)^2 + 40(y - 1.2) = 15(y^2 - 4y + 4) + 40(y - 1.2)$$

$$= 15y^2 - 60y + 60 + 40y - 48$$

$$= 15y^2 - 20y + 12$$

Neglecting the length of the jump itself, the hydraulic jump will occur when the backwater curve from the Connecticut River intersects the line drawn parallel to the bed slope at a distance equal to the conjugate depth (18.8 feet) above the bed. From Fig. 4 it is seen that in the lower portion of the conduit where the bed slope is 0.0007 any flow in excess of 5,670 c.f.s. will fill the conduit. In this particular case, then, the hydraulic gradient represents the backwater curve. The friction and bend losses require a slope in the hydraulic gradient of 0.00236 at a flow of 9,000 c.f.s. per barrel. The intersection of the hydraulic gradient with the curve of conjugate depth occurs at Station 51 + 50 as indicated on Fig. 8. The jump will, therefore, occur at approximately this location when the Connecticut River is at a stage of Elevation 16.54 (the crown of the conduit at the outlet).

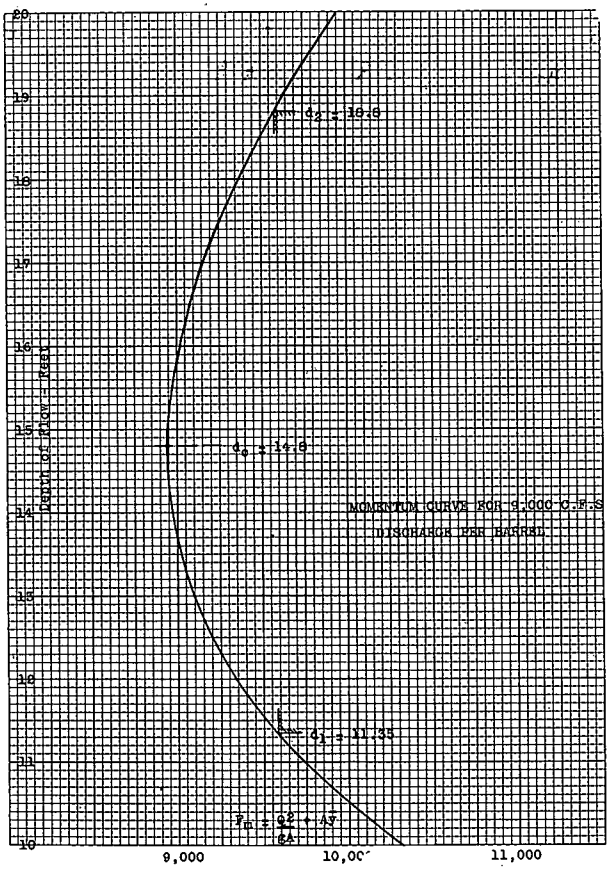


FIG. 6

If the level of the Connecticut River ~~is at a higher stage the line~~ representing the slope of the hydraulic gradient would start at the stage of the river and be drawn upstream at the slope of 0.00236. The higher the stage of the Connecticut River the further upstream from Station 51 + 50 will the jump occur.

Similar computations were made for discharges of 7,500, 5,000 and 2,500 c.f.s. per barrel. There are, of course, an infinite number of combinations of stage and rate of flow which might conceivably

TABLE 6

Specific Energy Curve - Q = 9,000 c.f.s. per Barrel

Depth Feet (y)	Area Sq. Ft. (A)	Velocity Ft./Sec. (V)	Velocity Head Feet ($V^2/2g$)	Specific Energy Feet $E = y + V^2/2g$
(1)	(2)	(3)	(4)	(5)
1.0	14.14	636.49		
3.0	69.65	129.22		
4.0	99.65	90.32		
5.0	129.65	69.42		
6.0	159.65	56.37	49.2	55.2
7.0	189.65	47.46	35.0	42.0
8.0	219.65	40.97	26.08	34.1
9.0	249.65	36.05	20.20	29.2
10.0	279.65	32.18	16.10	26.1
11.0	309.65	29.07	13.13	24.1
12.0	339.65	26.50	10.92	22.9
13.0	369.65	24.35	9.22	22.22
14.0	399.65	22.52	7.88	21.88
15.0	429.65	20.95	6.82	21.82
16.0	459.65	19.58	5.96	21.96
17.76	511.84	17.58	4.80	22.56
18.0	518.32	17.36	4.67	22.67
19.0	539.3	16.69	4.33	23.33
19.5	544.3	16.53	4.25	23.75

coincide, and at certain of these the hydraulic jump will come in contact with the roof.

Forms of Hydraulic Jump

According to Bakhmeteff[9] there are two distinct forms in which the hydraulic jump may occur, the direct or "shock" jump and the undular jump. The direct or "shock" form is typical of jumps of comparatively large height and is usually present in jumps occurring in flow through hydraulic structures. The undular jump is characteristic of jumps of comparatively low height and is observed in natural watercourses with moderately steep bottom slopes. Theoretical analysis and experiments by Bakhmeteff (9, page 249 and 10) on rectangular channels show that the so-called "kinetic flow factor" (represented by $\lambda = \frac{V_1^2}{gd_1}$, or twice the ratio of the kinetic energy head to the potential energy head) of 3 seems to have a direct bearing

[9]"Hydraulics of Open Channels" by Bahkmeteff, page 228.
[10]Trans. A.S.C.E., Vol. 101 (1936) "The Hydraulic Jump in Terms of Dynamic Similarity" by Bakhmeteff and Matzke, page 634.

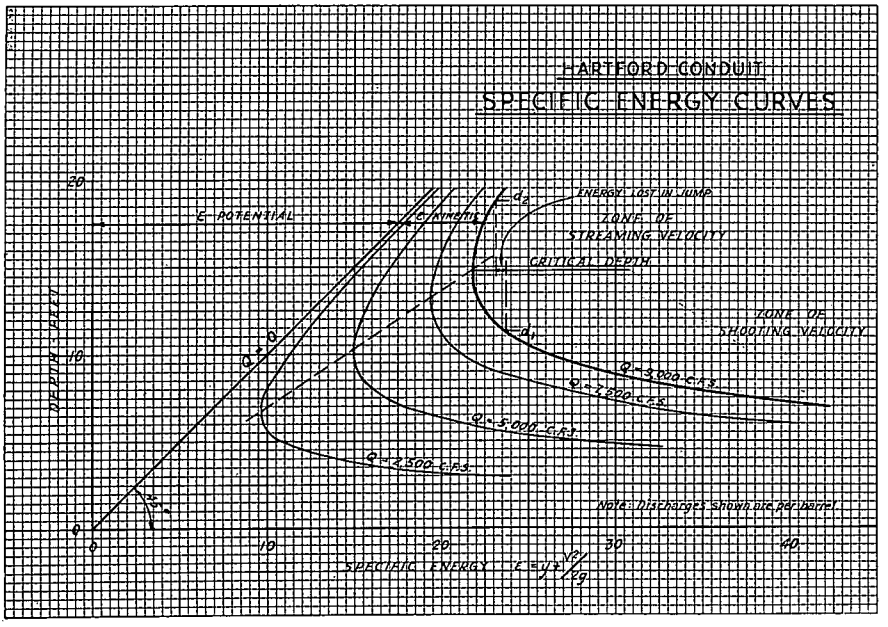


FIG. 7

on the form in which the jump occurs. In the region of high kineticities, where λ is equal to or greater than 3, the jump occurs in the direct form; in the region of low kineticities, where λ is less than 3, the jump acquires undulated features and the phenomenon of the jump seems to lose stability. This unstable feature is characteristic of flow at or near critical depth, at which depth $\lambda = 1$.

It should be noted that Bakhmeteff's kinetic flow factor is equal to the square of the Froude number F as generally expressed by $F = \frac{V}{\sqrt{gy}}$ This similarity is noted by Rouse.[11]

For the hydraulic jumps computed at the various rates of flow studied the kinetic flow factor was found to vary between 2.16 and 2.23, indicating that an unstable jump of the undular form might be

[11] "Fluid Mechanics for Hydraulic Engineers" by Rouse, page 388.

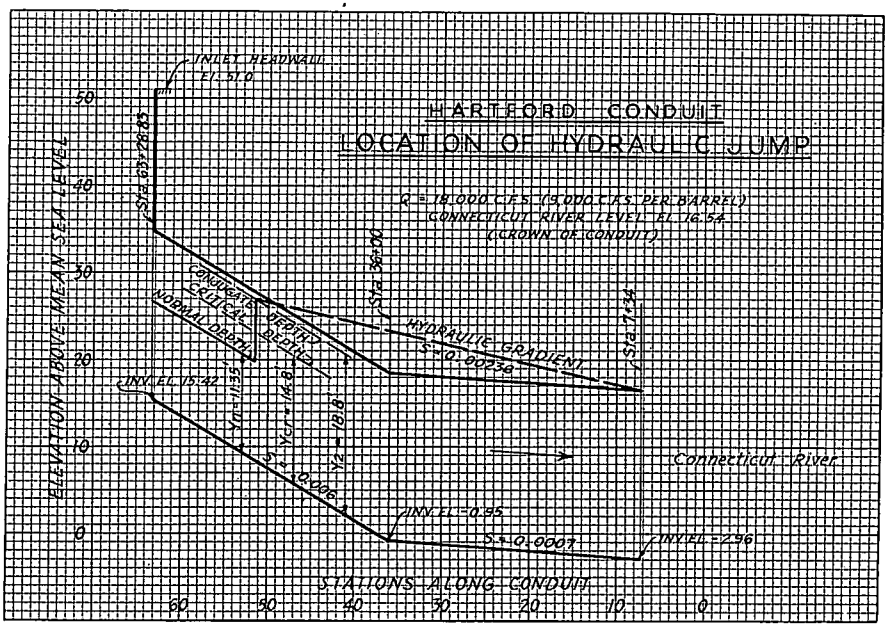


FIG. 8

expected, with resultant pulsating upward pressures on the underside of the roof of the conduit. However, the probability of coincidence of a critical river stage with a rate of flow in the conduit large enough to create a jump of troublesome proportions is extremely remote. It is believed that the structure is sufficiently massive to be undamaged by the infrequent eventuality of vibration due to undulations of the jump when in contact with the roof.

MISCELLANEOUS

The following items may be of general interest. The force exerted at the side walls at bends was computed as a matter of possible structural significance. As previously stated, a radius of 150 ft. at the center line of the structure was adopted for all curves. The radii to the deflecting walls of the near and far barrels of the conduit would be, therefore, about 149 ft. and 181.5 ft. respectively.

By Newton's second law $P = 2M\ V \sin \frac{d\theta}{2} = \frac{2 \times 62.5 \times 9{,}000 \times 16.6}{32.2}$

$\sin \frac{d\theta}{2} = 580{,}000 \sin \frac{d\theta}{2}$. For angles of this magnitude $\sin \frac{d\theta}{2} = \frac{d\theta}{2}$, therefore $\frac{d\theta}{2} = 1/298$ for the center wall and 1/363 for the outside wall. The resulting force is 1,950 lb. per. lin. ft. on the centre wall and 1,600 lb. per lin. ft. on the outside wall.

The Reynold's number for the maximum flow in the conduit is as follows:

$R = \frac{VL}{\upsilon}$, where V is the velocity in ft. per sec.; L is the equivalent diameter of the conduit, in ft.; and υ is the kinematic viscosity of the liquid, or $\frac{\mu}{\rho}$ or $\frac{\text{coefficient of viscosity}}{\text{density}}$. For water at 68 degrees F. in the English (foot-pound-second) system $\upsilon = \frac{2.1088 \times 10^{-5}}{1.93691} = 1.0887 \times 10^{-5}$ [12]. From these data $R = \frac{20.7 \times 33}{1.09} \times 10^{+5} = 63{,}400{,}000$.

[12] "Notes Prepared for John R. Freeman Lectures on Hydraulics," by K. C. Reynolds, page 33.

DISCUSSION

By Leslie J. Hooper, Member*

Mr. Keith has described in detail the hydraulic design of the Park River conduit. It is the purpose of this discussion to report briefly on the model study of the intake structure of this conduit as conducted at the Alden Hydraulic Laboratory, Worcester Polytechnic Institute.

The entrance model was constructed to a scale of 1/30th full size. The full scale structure was simulated from about Station 66 + 75 of the approach channel to Station 58 + 13 in the conduit. The approach channel was constructed of sawdust concrete between Stations 66 + 75 and 63 + 79. The transition between Stations 63 + 79 and 63 + 29 was constructed of wood. The curved wing walls and the remainder of the conduit to Station 58 + 13 were constructed entirely of lucite. Fig. 1, shows the plan of the model.

The lucite conduit was supported in wooden cradles mounted in turn on a series of wooden bents. Each cradle was fitted to the bottom section of the conduit and adjusted vertically with wedges. The grade of each barrel of the conduit was set as close as possible to the design figure, the worst discrepancy being .005 feet in the model corresponding to .15 feet in the field. This deviation of .005 feet was a local distortion at a joint and probably had very little influence upon the slope of the water surface except at the very low flows.

The two barrels of the conduit were anchored near the upper end and left free to slide over the remainder of its length in the cradles. This was necessary because of the relatively high coefficient of expansion of the lucite. The change in total length of the model was about 5/16″ for a temperature change of 30° F. This was the difference in temperature between the air in the room and the water used in the tests during the winter months.

Water was supplied to the model through two calibrated Venturi

*Professor of Hydraulic Engineering, Worcester Polytechnic Institute, Worcester, Mass.

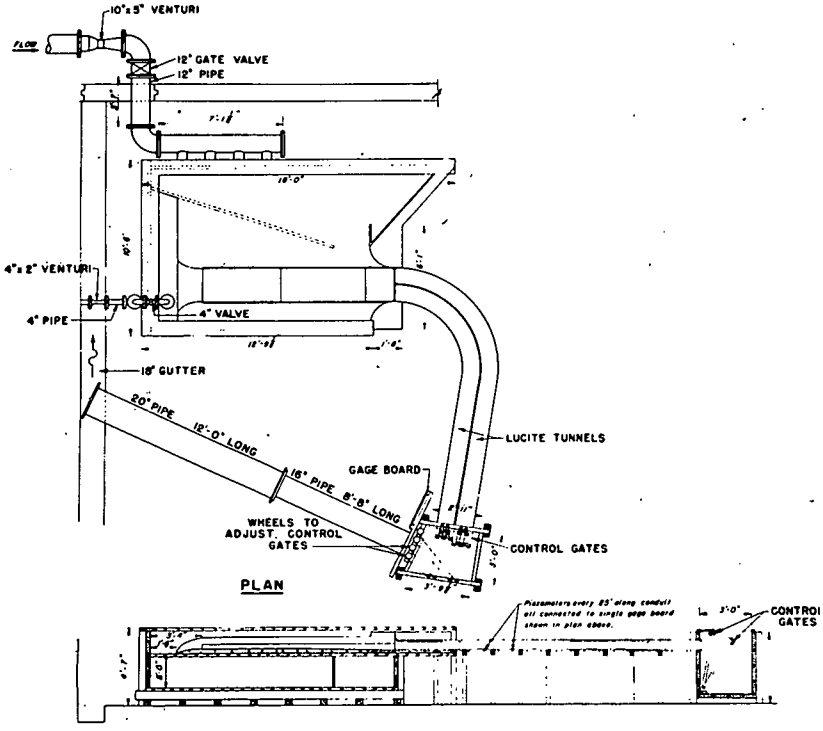


FIG. 1.—MODEL OF PARK RIVER CONDUIT INTAKE. MODEL RATIO—1:30

meters, a 10" x 5" meter to measure the high discharges and a 4" x 2" meter for the low flows. The differential pressures of the Venturi meters were measured with mercury manometers for the high discharges and water and air manometers for the low.

The Venturi meters discharged into a head box approximately 10 feet by 14 feet in plan and 4.5 feet high. The water came into the box near the bottom where a long expanding channel was provided to slow the water down gradually. The discharge then flowed up through a slot at the upstream end of the head box and thence onto the model. The water velocities in this region were so low that no racks or rafts were necessary to maintain quiet flow conditions at all discharges.

The backwater effect of the Connecticut River was provided by tilting gates at the discharge end of the model at Station 58 + 13. There were two interleaving gates, hinged across the bottom, for each barrel of the conduit. This arrangement allowed fine adjustment of the outflow area with a uniform distribution of the discharge. After passing the tailwater gates the water fell into an afterbay whence it flowed to waste.

The hydraulic gradient in the model was determined by means of a number of water manometers which were connected to piezometers installed along the bottom centerline of the inside barrel at intervals of 25 ft. full scale. Three piezometers were also installed in the roof of the inlet transition. At Station 59 + 00 an extra piezometer was installed in the outer barrel. The two piezometers at this station were used as reference piezometers and losses from there to the downstream end of the conduit were computed for various discharges.

The water profile in the approach channel was measured with a surveyor's level and a special rod equipped with a scratch point, reading elevations full scale directly.

Only a few of the test results and the general conclusions of the test work will be presented.

In considering these results, it must be borne in mind that the prototype flow was computed to Station 59, and the model was then adjusted to the corresponding flow and gradient (at Station 59). The difficulties of setting the model, and the inaccuracies of piezometers as indicators of the true water surface gradient explain, to a large degree, the discrepancies between computed and model results. All of the flow conditions observed were satisfactory and no modification of the design was necessary. A quiet pool formed upstream from the entrance of the conduit for high discharges and high Connecticut River elevations. There were no waves formed since the velocity of approach was low.

The discharge capacity of the conduit was found to be somewhat in excess of the design calculation. For instance it was expected that with a headwater elevation of 48 mean sea level the conduit would discharge 19,700 c.f.s. with the Connecticut River at Elevation 27 mean sea level. It was found that the conduit would discharge this flow with the Connecticut River at Elevation 29 mean sea level.

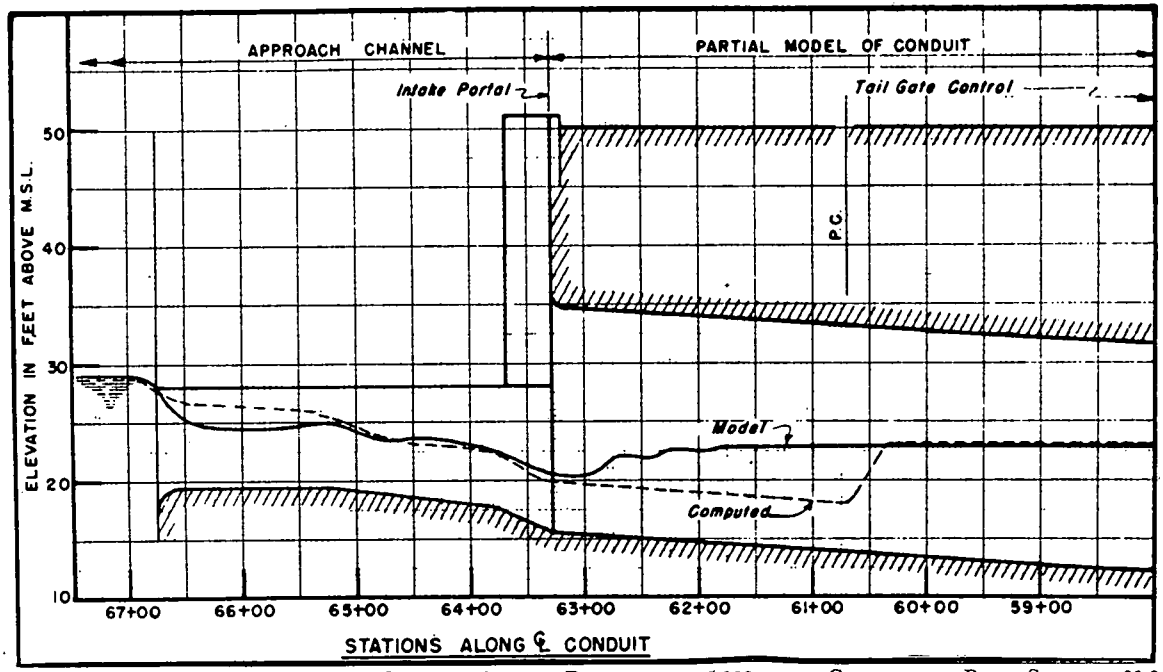


FIG. 2.—PARK RIVER CONDUIT. HYDRAULIC GRADIENT STUDY. DISCHARGE = 5,000 C.F.S. CONNECTICUT RIVER STAGE = 23.0 M.S.L.

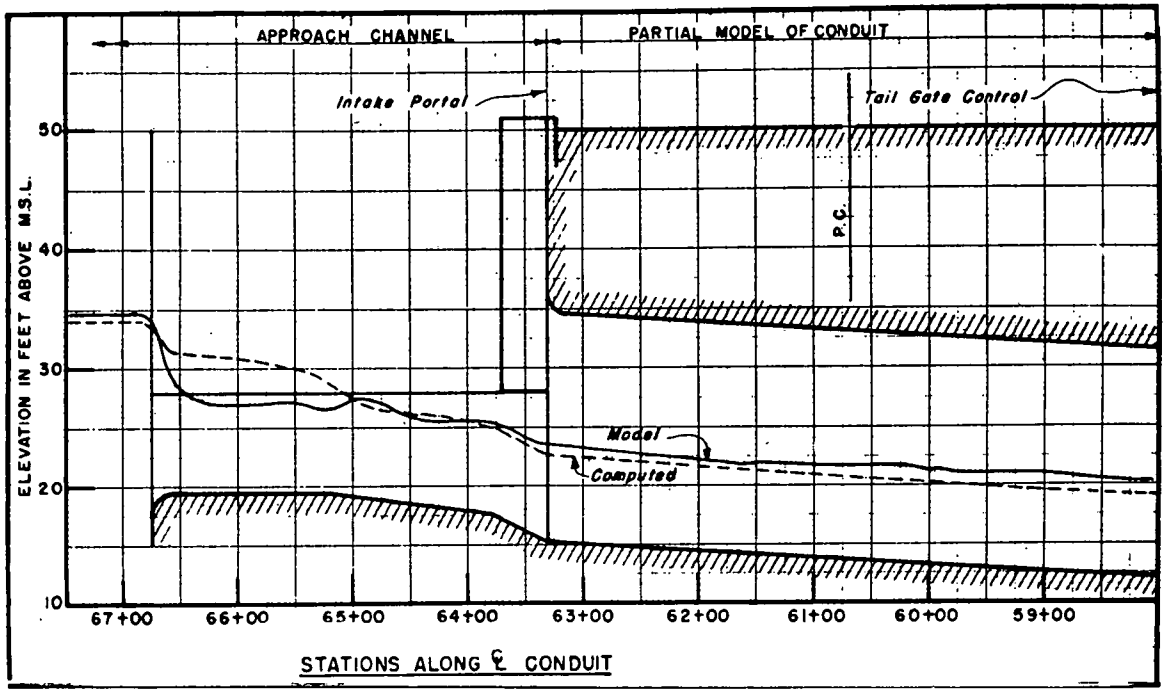


FIG. 3.—PARK RIVER CONDUIT. HYDRAULIC GRADIENT MODEL STUDY. DISCHARGE = 10,000 C.F.S.

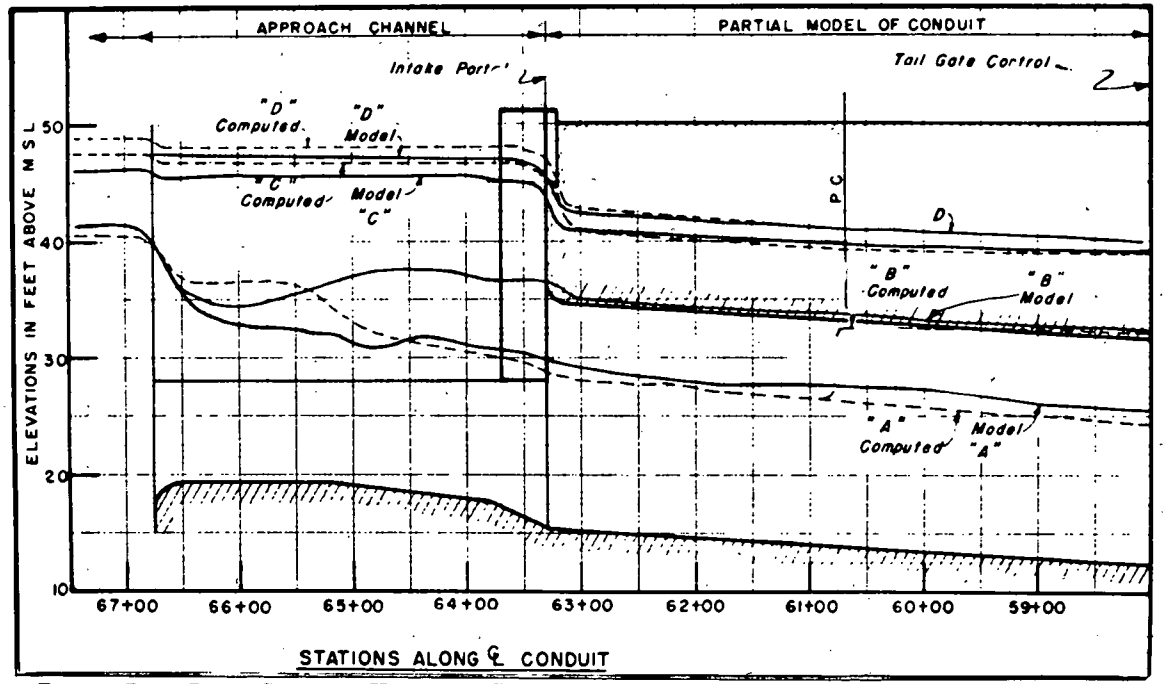


FIG. 4.—PARK RIVER CONDUIT. HYDRAULIC GRADIENT STUDY. DISCHARGE = 19,700 C.F.S. CONNECTICUT RIVER ELEVATION: A = 20 OR BELOW; B = 20.3; C = 27.0; D = 28.5

When the elevation of the water in the pool upstream from the entrance was sufficient to submerge the entrance to the conduit section it was found that there were negative pressures in the roof transition. With a discharge of 22.400 c.f.s. the least pressure measured was 5 ft. of water below the concrete. Since this pressure was not low enough to cause cavitation and since the flow conditions that cause the negative pressures occur so rarely no modifications of the designs were considered necessary.

The accompanying figures show a comparison of some of the test results with the values computed by Mr. Keith's analysis.

In Fig. 2 the comparison is shown for a flow of 5,000 c.f.s. It is seen that the jump occurred nearly 200 feet further upstream in the model than computed. As Mr. Keith emphasized in his paper, the actual location of a jump is most difficult to predict. Furthermore, the relative friction is somewhat greater in the model. The jump would be expected to move upstream for that condition.

Fig. 3 shows a comparison of flow conditions for a discharge of 10,000 c.f.s and a low water elevation of the Connecticut River. The two curves are in reasonably good agreement. It would appear that the model friction at the relatively low depth was higher than that computed for smooth concrete. However, experimentally, the agreement is satisfactory.

Fig. 4 shows the comparison of the computed and model test flow conditions with a discharge of 19,700 c.f.s. and various Connecticut River elevations. At small depths the effect of friction may still be seen. The jump for curve B is located at the entrance in the model test. Thereafter the friction readings are in excellent agreement in the conduit and the pool water surface elevations in the model test are below the computed values.

In conclusion these model tests showed that:

1. The design of the conduit entrance was adequate for all flow conditions tested.

2. The discharge capacity of the conduit was somewhat in excess of the design requirements: 19,700 c.f.s. being discharged with a headwater at Elevation 48 mean sea level and the Connecticut River at Elevation 29 mean sea level or a total head two feet less than originally computed.

3. Small negative pressures were found at the roof of the inlet transition but were not sufficiently important to require modification of the designs.

USE OF ALIGNMENT CHARTS

FOURTH ARTICLE OF A SERIES*

By William F. Covil**

The advantages resulting from the use of alignment charts have been summarized in a previous article. I would like to emphasize one of those advantages, that "checking of computations may be done rapidly and accurately." This is of particular value to busy executives who haven't time to inquire intimately into design but whose trained eye often spots a seeming error or inconsistency.

It is a simple matter in this case to make a spot check with an alignment chart without going into any refinements of design. An excellent example of this is with Chart No. 5 presented herewith for finding the steel area needed in concrete beams.

This chart is designed to accompany Chart No. 3 for Shear, and Chart No. 4 for Bond. In the interest of simplicity, these charts do not solve for the effective depth d and the width b of a beam for "balanced design" of reinforcing and concrete and it is assumed that this d has been determined beforehand for use with the Charts.

It will be particularly noted that Charts 3, 4 and 5 may be conveniently used for concrete slabs as well as for beams. This is readily accomplished by using $b = 12''$ on Chart No. 3; V = Total external shear in pounds *per ft. of width* on Charts Nos. 3 and 4; ΣO = Summation of perimeters of bars *per ft. of width* on Chart No. 4; and A_S = Necessary steel area *per ft. of width* on Chart No. 5.

Chart No. 5

Alignment Chart for the Solution of the Equation for Steel Area Necessary in a Concrete Beam

This chart is a companion chart to Chart No. 3 for Shear, and

*Series of Charts to be continued in subsequent Journals B.S.C.E. Extra copies of alignment charts may be obtained at 10 cents each at 715 Tremont Temple, Tremont St., Boston, Mass.

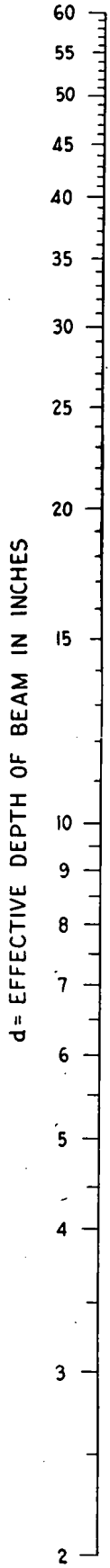
**Senior Civil Engineer, Metropolitan District Water Supply Commission, 20 Somerset St., Boston, Mass.

STEEL AREA
FOR
CONCRETE BEAMS

$$A_s = \frac{8}{7} \cdot \frac{M}{f_s d}$$

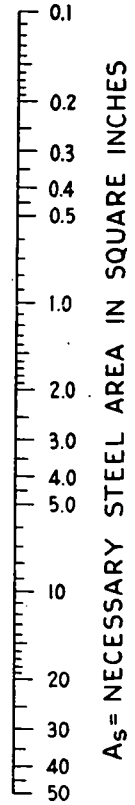
for

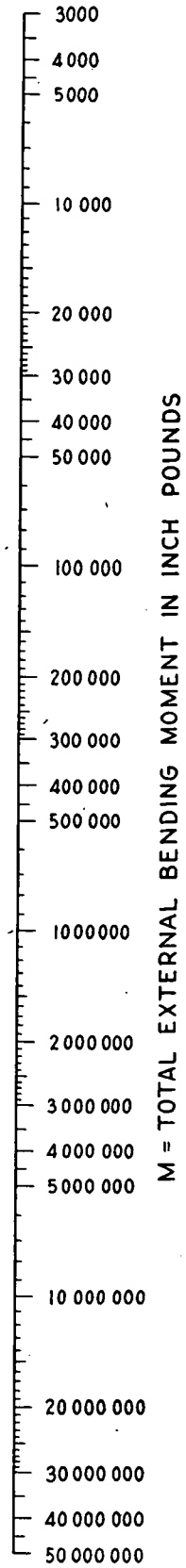
$$f_s = 20{,}000\,\#/\square''$$

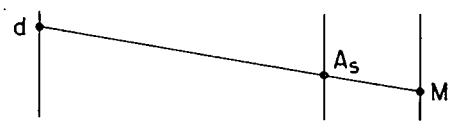


SUMMATION OF AREAS OF PLAIN BARS*									
NO. OF BARS	SIZE OF PLAIN BARS IN INCHES								
	1/2" ROUND	1/2" SQUARE	5/8" ROUND	3/4" ROUND	7/8" ROUND	1" ROUND	1" SQUARE	1 1/8" SQUARE	1 1/4" SQUARE
1	.1963	.250	.3068	.4418	.6013	.7854	1.0	1.266	1.562
2	.39	.50	.61	.88	1.20	1.57	2.0	2.53	3.12
3	.59	.75	.92	1.33	1.80	2.36	3.0	3.80	4.69
4	.79	1.0	1.23	1.77	2.41	3.14	4.0	5.06	6.25
5	.98	1.25	1.53	2.21	3.01	3.93	5.0	6.33	7.81
6	1.18	1.50	1.84	2.65	3.61	4.71	6.0	7.60	9.37
7	1.37	1.75	2.15	3.09	4.21	5.50	7.0	8.86	10.94
8	1.57	2.0	2.45	3.53	4.81	6.28	8.0	10.13	12.50

*Table may also be used for deformed bars







KEY DIAGRAM

CHART 5

Chart No. 4 for Bond, and represents the final chart which is contemplated for Concrete Design, unless there are requests for other charts along these lines.

The equation for steel area is usually expressed:

$$A_s = \frac{M}{f_s jd}$$

where A_s is the necessary steel area in square inches; M is the total external bending Moment in inch pounds; f_s is the allowable fiber stress in the steel in pounds per square inch; and jd is the moment arm of the internal force couple, where d is the depth of beam from the compression surface to the centroid of the longitudinal tensile reinforcement,—the effective depth.

For simplification j is assumed to be 7/8 and f_s is taken as 20,000 pounds per square inch, since this is now general practice, and the formula becomes

$$A_s = \frac{\frac{8}{7} \cdot \frac{M}{d}}{20{,}000}$$

The operation of the chart itself is very simple as illustrated by the key diagram.

A table showing the areas of plain bars is given for convenience of ready reference, but has no function in the operation of the chart itself. It is general practice to use the values in this table for deformed bars also.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

APRIL 20, 1944.—A Joint Meeting of the Boston Society of Civil Engineers with the Northeastern Section, American Society of Civil Engineers was held this evening at the Boston City Club. Two hundred ninety five members and guests attended the meeting and dinner.

President Francis H. Kingsbury of the Northeastern Section, American Society of Civil Engineers presiding at the Joint Meeting called upon President Samuel M. Ellsworth of the Boston Society of Civil Engineers to conduct any BSCE matters of business.

President Ellsworth outlined briefly some of the Boston Society of Civil Engineers plans for the year and announced that the subject of the next meeting May 17, at 20th Century Association, will be "War Time Planning for Needed Highway and Rapid Transit Improvements in Boston Metropolitan Area," to be covered by several speakers.

President Kingsbury then resumed the chair and called upon Prof. Gramstorff, Secretary, for announcements.

President Kingsbury introduced the head table guests including representatives of City and State Planning Boards and of the Engineering Societies and Architectural Societies and others.

President Kingsbury then introduced the speaker of the evening, Robert Moses, Commissioner of Parks, New York City, who gave a most interesting address on "Municipal and Metropolitan Improvements."

The address was broadcast over WMEX. Following this, Mr. Moses showed many lantern slides of the New York City Parkways and connections for serving the city's immense traffic.

Adjourned at 9:00 P.M.

EVERETT N. HUTCHINS, *Secretary*

MAY 17, 1944.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the 20th Century Association, 3 Joy Street, Boston, and was called to order by Vice-President, Harry P. Burden, in the absence of the President, Samuel M. Ellsworth. Sixty-six members and guests attended the meeting, fifty-eight attending the supper.

Vice-President Burden asked members to rise and announced the death of the following members:—

Arthur J. Maynard, who was elected a member May 10, 1915, and who died January 3, 1944.

Franklin H. Robbins, who was elected a member December 18, 1895, and who died March 12, 1944.

Theodore B. Parker, who was elected a member January 27, 1926, and who died April 27, 1944.

C. Leonard Brown, who was elected

a member April 20, 1910, and who died April 30, 1944.

Robert J. Greer who was elected a member November 15, 1939, and who died May 12, 1944.

James R. Gibson, who was elected an Associate Member April 26, 1922, and who died May 13, 1944.

The Secretary read a postcard notice sent to all members of the Society as provided by Vote of the Board of Government on April 13, 1944, submitting a form of motion to be acted upon at this meeting as follows:—

"That the Board of Government be authorized to cooperate with other Professional Engineering Societies to bring about an amendment to the Labor Laws, as it now affects professional engineers, i.e., to exempt engineers from the application of provisions of said law relating to employment conditions."

Mr. C. A. Farwell offered a substitute motion which he reported had the approval of the Board of Government, as follows:—

"That the Board of Government be authorized to cooperate with other professional societies in an effort to create a more favorable status under the National Labor Relations Act, or the provisions for its enforcement, for their employee members of both professional and sub-professional grade."

and this motion was adopted by the Society.

Vice-President Burden announced Excursion or Inspection Trip to be held June 10, 1944, to the General Edward Lawrence Logan Airport, East Boston, a large extension of which is being carried on under the direction of the Massachusetts Department of Public Works.

Vice-President Burden announced the speakers of the evening:—

Hall Nichols, Secretary and Chief Engineer, Mass. Emergency Public Works Commission, subject, "Public Works Construction in the Bridge from War to Peace."

Hon. Edward W. Staves, State Representative, and Vice-Chairman of the Massachusetts Post-War Highway Commission, and Phillip H. Kitfield, Ass't Project Engr. Mass. Dept. Public Works, subject, "The Post-War Highway Construction."

Commissioner Carroll L. Meins, Chairman of the Mass. Dept. of Public Utilities, subject. "Aims and Activities of the Metropolitan Transit Commission."

The Speakers presented a very comprehensive summary of the plans being made for the transportation needs of the State and Metropolitan Communities, including public works of all kinds and both highway and rapid transit facilities.

Following Vice-President Burden's comment on the fact that none of the Speakers were members of the Society. Mr. Channing Howard appropriately moved that the Society extend to these speakers a rising vote of thanks.

Adjourned at 9:15 P.M.

EVERETT N. HUTCHINS, *Secretary*

AIRPORT INSPECTION TRIP AND MEETING OF B.S.C.E.

JUNE 10, 1944.—An inspection trip was held today at the General Edward Lawrence Logan Airport, at East Boston. The purpose of the trip was to inspect the present airport facilities and to view the general site and particularly the current phase of the expansion of the airport which is being undertaken by the Massachusetts Department of Public Works. The contract now under way for the placing of fill-

ing by the use of hydraulic dredges is the initial contract work.

About 145 members and guests attended this trip, some arriving in their own cars and some by means of the "Airport" bus service, from Maverick Square, East Boston. One hundred thirty-five attended the lunch.

The excursion included opportunities to visit the present administration building, where an excellent model map of the airport project was on display, prepared by the contractor. Also the activity of arrival and departure of airplanes on regular schedules was a feature of interest.

A feature of particular interest was the boat trip from the small wharf at the southerly bulkhead to the dredges of the contractor, the Gahagan Construction Corporation of New York. Three launches were available to take the group to the dredges. One of the dredges, the "Nebraska," is electrically operated, with service cables laid across the harbor by the Edison Company; the other dredge, the "No. 5" is operated by steam, with oil burning equipment.

Following the boat trip, the group then travelled to Governors Island, which is joined to the main land or the airport by a dike or causeway constructed in recent months by material excavated from the westerly portion of Governors Island. The top of this island affords an excellent view of the whole airport project.

The dredges are located off the southerly shore of the island in the area designated as the borrow area and the material excavated (blue and yellow clay) is pumped to the outlet of the discharge pipe along the locations of the embankments on the center lines of the future runways, lying northerly of Governors Island. Up to the present time there has been pumped and placed on the lines of the first embankment about 1,000,000 cubic yards of material.

Of unique interest on the island are the Old Fort Winthrop, a granite structure of fine workmanship, although now in dilapidated condition, and the various tunnels and powder magazines and revetments of antiquated design.

Box luncheons were served to all at 12:15, under the trees on the northerly slope of Governor's Island where settees had been located.

The meeting of the Society was held at 12:50 at the same location, overlooking the great area of 2000 acres where the airport expansion is to take place.

Vice-President Harry P. Burden presided in place of President Samuel M. Ellsworth, who wrote to the Secretary that he had hoped to be able to resume his activities and attend this meeting but felt that it was wiser at the moment to forego that pleasure.

Vice-President Burden stated that this inspection trip had been arranged through the courtesy of the Massachusetts Department of Public Works, and further,—

> "He appreciates the effort that has been made by the Department in locating our meeting place under the trees below the Old Fort which was erected nearly 100 years ago. From this site we can see the expanse of water which will some day be converted into one of the finest airports.
>
> "I wish to convey to Commissioner Herman A. MacDonald our true appreciation of the work which has been done by his Department employees in fitting up this unique site for our meeting. I have learned that the Contractor,—the Gahagan Construction Corporation, has also carried out considerable work in improving the approaches to this site.
>
> "Both the State and the Contractor are due our thanks also for the water transportation for inspecting the dredges."
>
> "Our program—in the nature of a

symposium—will portray the vision of those who have endeavored to make plans and undertake the development of this Airport. The first speaker—(in public life for many years)—has the responsibility for decisions and taking the lead in the Airport Development—Commissioner of Public Works—Mr. Herman A. MacDonald."

The Commissioner emphasized the events leading to the legislation authorizing the expansion of the airport and the many factors in favor of development of an airport so near to the heart of the city.

The next speaker has been associated with the State Department of Public Works for a score of years or more; General Richard K. Hale, Director of Waterways, who outlined the development of the Airport to date and the plans for the future.

In the important matters of foundations for the Airport and the desirability of using the clays of Boston Harbor for hydraulic fill, the Department selected as its advisor Professor Arthur Casagrande, Professor of Soil Mechanics, Graduate School of Engineering, Harvard University, who discussed the foundation conditions and the characteristics of the hydraulic fill that is being used for the basic fill for the runway embankments.

Professor John R. Markham of the Department of Aeronautics, Massachusetts Institute of Technology, was scheduled to address the meeting, but was prevented from attending.

The actual work of constructing the runway embankments by the dredging method was outlined by Major Walter H. Gahagan, President of the Gahagan Construction Corporation, and he also spoke interestingly of other airports and their construction, details and development, and pictured the great opportunities and prestige Boston will have when this airport is completed.

EVERETT N. HUTCHINS, *Secretary*

SANITARY SECTION

APRIL 5, 1944.—A joint meeting of the Sanitary Section, B.S.C.E. and the Northeastern University Section was held in Room 228 of the New Building at Northeastern University at 7:30 p.m. this evening, following an informal dinner gathering at the Old France Restaurant. Forty-two persons attended the meeting, with thirty-four at the dinner.

Chairman Gibbs called the meeting to order and immediately surrendered the chair to Chairman Bishop of the Northeastern University Section, who presided during a business meeting of that section.

Chairman Gibbs then introduced the speaker of the evening, Mr. Joseph A. McCarthy, Chief of Laboratory, Lawrence Experiment Station, Massachusetts Department of Public Health, who gave an interesting talk on 'Sewage Treatment Experiments—Sometimes They Work." Mr. McCarthy discussed some of the experiments on sewage treatment which have been carried on at the Lawrence Experiment Station. Among the treatment processes discussed were trickling filters—with and without effluent recirculation—sand filters and septic tanks. After a considerable discussion, the speaker was given a rising vote of thanks and the meeting adjourned about 8:30 p.m.

GEORGE C. HOUSER, *Clerk*

DESIGNERS' SECTION

APRIL 12, 1944—The meeting of the Designers' Section was held in the Society rooms April 12, 1944, starting at 6:45 P.M., with Chairman Lawrence M. Gentleman presiding. The report of the previous meeting was approved as read. The Chairman requested suggestions

from the members regarding subjects to be discussed at future meetings and announcement was made regarding tickets for the joint meeting of the Northeastern Section of the American Society of Civil Engineers and the Boston Society of Civil Engineers on April 20.

The speaker for the evening was Mr. O. H. Ammann, Consulting Engineer, New York City, who gave an illustrated talk on "Bridges of New York." Following the paper, Mr. Ammann discussed questions brought up by members and guests.

The meeting adjourned at 9:40 P.M.

FRANK L. LINCOLN, *Clerk*

MAY 10, 1944—A meeting of the Designers' Section was held in the Society rooms on May 10, 1944, following an informal dinner at the Ambassador Restaurant. The meeting started at 6:45 p.m., with Chairman Lawrence M. Gentleman presiding. The report of the previous meeting was approved as read.

The speaker for the evening was Mr. A. H. Hadfield, Assistant Chief, Airways Engineering Division, Civil Aeronautics Administration, Washington, D. C. Mr. Hadfield discussed the design of both rigid and flexible type airport pavements, stressing particularly the soil classifications, the thicknesses of pavements and bases, and the methods of drainage advocated by his department. The formal paper was followed by informal discussion of questions by members and guests. There was an attendance of 56 members and guests.

The meeting adjourned at 9:00 p.m.

FRANK L. LINCOLN, *Clerk*

HYDRAULICS SECTION

MAY 3, 1944.—A regular meeting of the Hydraulics Section was held in the Society rooms, conducted by the Chairman, Allen J. Burdoin. The speaker was Mr. Scott Keith, of Metcalf and Eddy, who, with the aid of slides, presented a paper entitled "Hydraulics of the Park River Conduit, Hartford, Conn." Mr. Keith reviewed the basic hydraulic allowances and commented upon the salient features of design of this important flood control project at Hartford

Professor Leslie J. Hooper of the Worcester Polytechnic Institute in a prepared discussion went into some phases of the model study made in connection with the project, particularly with regard to the hydraulic jump occurring in the conduit under certain conditions. The model investigation was illustrated by a movie. Mr. Charles W. Cooke, Engineering Department, City of Hartford, exhibited a most interesting film relating to the construction of the conduit.

Thirty-seven members and guests attended the meeting.

HAROLD A. THOMAS, *Clerk*

Northeastern University Section

MARCH 28, 1944.—A Noon Meeting of the Northeastern University Section of the Boston Society of Civil Engineers was held with thirty-one persons present including students and four faculty members.

President Henry J. Bishop opened the meeting at 1:15 p.m., with the reading of the notices for future meetings. Mr. Bishop then introduced the speaker, Mr. James E. Jagger, Acting Assistant Secretary for the American Society of Civil Engineers, whose subject was entitled "Maintenance."

Mr. Jagger pointed out that while the Engineer need not worry about the maintenance of a structure after it has been finished, his education is an entirely different matter. If one is to be a good engineer, he must not let his

education lag after graduation. There is plenty of material available from which the ambitious engineer can gain information on new subjects.

The meeting adjourned at 2:30 P.M.

PHILIP A. FRIZZELL, *Clerk*

APRIL 5, 1944.—Joint meeting of the Sanitary Section of the Boston Society of Civil Engineers and the Northeastern University Section of the Boston Society of Civil Engineers.

Dinner was held at the Old France Restaurant at 6:00 P.M. Thirty-four persons were present. including twelve student members. After dinner, the meeting adjourned to room 228 Northeastern University.

The meeting was opened at 7:30 P.M. by Mr. Fred Gibbs, Chairman of the Sanitary Section, who conducted the business for that section. The meeting was then turned over to Henry J. Bishop, Chairman of the Northeastern University Section. A nominating Committee for future officers was elected. This committee was composed of the entire Senior Civil Group.

Mr. Gibbs then introduced the speaker for the evening, Mr. Joseph A. McCarthy, Chief of Laboratory, Lawrence Experiment Station, Massachusetts Department of Public Health. Mr. McCarthy's subject was "Sewage Treatment Experiments—Sometimes They Work."

Mr. McCarthy dealt with experiments that he had worked on in the past, such as, ozone, filters, study of sludges, septic tanks, and the trade waste problem. Trade wastes have become one of the greatest problems of the present day. Many new inventions are appearing in this field. Great possibilities for research in this field are now appearing since it has become necessary to stop the pollution of the streams; also valuable material can be salvaged from these wastes, if economical means of doing so can be found. After his talk, a lengthy discussion period was held in which all members were free to ask questions.

The meeting of forty-three persons, including sixteen students adjourned at 8:30 P.M.

PHILIP A. FRIZZELL, *Clerk*

APRIL 12, 1944.—Noon meeting of the Northeastern University Section of the Boston Society of Civil Engineers.

Thirty-four persons were present including students and faculty members.

President Henry J. Bishop opened the meeting at 1:00 P.M. by reading the notices of future meetings. The report of the nominating committee was read by Mr. Piper. The following members were nominated for office.

President—Philip A. Frizzell
Vice President—Samuel Patterson
Secretary—Arthur Hebert
Treasurer—William Maravel

Mr. Bishop then introduced the speaker for the meeting. Mr. Diebert of the W. and L. E. Curley Company.

Mr. Diebert gave an illustrated talk on the anatomy of a transit. He also showed the best way to clean and adjust an instrument.

The meeting adjourned at 1:45 P.M.

PHILIP A. FRIZZELL, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[July 20, 1944]

The By-Laws provide 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 mem-

ber 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 consideration. 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

CHARLES W. BOWEN, Providence, Rhode Island. b. January 15, 1914, Gloucester, Mass.) Graduated from Northeastern University in June 1936, with B. S. degree in Civil Engineering. Experience, June 1936 - Sept. 1936, assistant engineer for the Donnelly Electric & Neon Company, Boston, Mass., duties consisting of drafting and detailing of steel trusses and supports for signs. Sept. 1936 - May 1938, engineer for John Donnelly & Sons, Boston, Mass., duties, design and drafting of steel trusses and supports for advertising signs, detailing steel connections, determining safe roof loads, earth bearing capacity and wind loads; May 1938-May 1939, consulting engineer. Principal employer, F. H. Birch Co. (Advertising) Boston, Mass., for whom I did the same type of work as listed above; May 1939 to date, U. S. Engineer Office, Providence, R. I.; May 1939 - Feb. 1941, engineering draftsman, duties consisted of making finished drawings from original design on dams, dikes, pumping stations and equipment, stoplog structures, etc.; Feb. 1941 - June 1941, Senior Draftsman, design of steel reinforcement for concrete and making original drawings of the same; June 1941 - May 1942, Principal Draftsman, and from May 1942 to Nov. 1943, Asst. Engineer. Design and layout of all utilities for Westover Field, Chicopee Falls, Mass., Nov. 1943 to date, Assistant Engineer. Design and layout of dams, dikes, conduits, pumping stations and other flood control projects. Refers to, *C. O. Baird, J. C. Dingwall, E. A. Gramstorff, J. B. McAleer, H. B. Shumway.*

RUSSELL C. CHASE, Stoneham, Mass. (b. February 5, 1907, Boston, Mass.) Northeastern University, 1929. Experience, City of Medford to 1931; Metropolitan District Commission to 1942; Fay, Spofford & Thorndike (Resident Engineer Arctic Base) to January, 1944; Shell Oil Company (Division Asphalt Representative) to date. Refers to *C. Cann, C. A. Farwell, F. Heaney, W. L. Hyland.*

CHARLES H. NORRIS, Lexington, Mass. (b. March 30, 1910, Pendleton, Oregon). Graduated in 1931 from University of Washington with degree of B.S. in Civil Engineering; attended Massachusetts Institute of Technology as a graduate student receiving degree S.M. in Civil Engineering in 1932 and SC.D. in 1942. Experience, 1932-1933, teaching fellow in Civil Engineering at University of Washington; 1934-1936, assistant in Civil Engineering at Massachusetts Institute of Technology; 1936-1938, Mechanical Engineer and Assistant to Chief Mechanical Engineer of American Steel Foundries, Chicago, Illinois; 1938-1941, instructor in Civil Engineering at Massachusetts Institute of Technology; 1941 to date, Assistant Professor of Structural Engineering at Massachusetts Institute of Technology. Refers to *C. B. Breed, J. B. Babcock, J. B. Wilbur, D. W. Taylor, J. D. Mitsch.*

Transfer from Grade of Junior

WALTER A. FORD, Quincy, Mass. (b. November 21, 1890, Staten Island, New York). Graduate, Mechanic Arts High School, Boston, in 1910. Experience, 1911-1918, Boston Transit Commission —rodman, instrumentman, inspector, senior draftsman. (Plots, studies, contract and report plans, etc.) 1918-1932, Union Savings Bank, Boston; 1932-1933 Gardner Adding Machine Company, Boston; 1933-1938, City of Quincy, and U. S. Navy, Hingham and Squantum, Mass.,—foreman, field inspector, draftsman, chief of party (Plots, studies, contract and record plans, surveys and field construction, etc.); 1938-1941, Metropolitan District Commission, Sewerage Division. (draftsman, similar to above); 1941-1943 Stone & Webster Engineering Corporation (draftsman, structural and mechanical); 1943-to date, Jackson & Moreland, Boston, Mass., (appraisal engineer). Refers to *J. J. Casey, W. W. Davis, J. B. Flaws, R. W. Loud.*

Transferred from Grade of Student

LAWRENCE I. PIPER, Jamaica Plain, Mass. (b. March 28, 1922, Cambridge, Mass.) Graduated from Jamaica Plain High School in June, 1939. Went back for a post-graduate course and finished in June, 1940. Graduated from Northeastern University June 3, 1944, with B.S. Degree in Civil Engineering. Cooperative work while at Northeastern University was as follows; June-September, 1941, chainman for the Boston & Albany Railroad, rodman's work in connection with laying out spiral, lines and grades; 1941-1943 (part time) transitman for Whitman & Howard, Engineers, had general field experience with transit and rodman's work in connection with land, airport, and construction surveys; June, 1944, to present have been employed by the Grumman Aircraft Engineering Corporation of Bethpage, Long Island, New York as a trainee for their Engineering Department. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

ADDITIONS

Members

LEO E. GAUDETTE, 88 Forest Street, Winchester, Mass.

NORMAN D. HALL, 38 Pitman Avenue, Greenwood, Mass.

CHARLES G. HUNT, JR., 721 Kensington Avenue, Plainfield, N. J.

FRANK K. PERKINS, 127 Waverley Avenue, Newton 58, Mass.

JAMES N. PRIEST, 316 St. Paul Street, Brookline 46, Mass.

JOHN F. SHEA, Cushing General Hospital, Framingham, Mass.

Juniors

ROBERT H. BROWN, Main Street, Chatham, Mass. Box 163.

RICHARD M. BURBANK, 25 Hartsuff Street, Rockland, Mass.

PAUL W. CAMERON, 59 Tamworth Hill Avenue, Greenwood, Mass.

ERNEST A. DEVEUVE, 77 Augustus Avenue, Roslindale, Mass.

GEORGE B. FAY, 14A Whittier Street, Dover, N. H.

JOHN F. HEANEY, JR., 31 Chase Avenue, Lexington 73, Mass.

RICHARD P. HEFFERNAN, 36 Arbella Street, Salem, Mass.

CHARLES G. HUNT, JR., 721 Kensington Avenue, Plainfield, N. J.

GEORGE O. HUSE, 133 Vernon Street, Newton, Mass.

JOHN W. IRELAND, 36 Newhall Street, Lynn, Mass.

FREDERICK B. JACKSON, 128 Rowe Street, Auburndale, Mass.

JOHN G. JARNIS, 29 Warren Avenue, Waltham, Mass.

CARL E. JOHNSON JR., 152 Mary Street, Arlington, Mass.

ESA M. KHOURY, 135 Hudson Street, Boston, Mass.

PAUL D. KILLAM, 19 West Avenue, Salem, Mass.
HENRY A. KINGSBURY, 1 Curve Street, Medfield, Mass.
JOSEPH C. LAWLER, 89 Empire Street, Lynn, Mass.
JOSEPH W. LAVIN, 21 Dunlap Street, Dorchester, Mass.
EDWARD F. LOBACZ, 161 Salem Street, Wakefield, Mass.
RALPH L. METCALF, 450 Plymouth Street, Bridgewater, Mass.
RICHARD A. MORSE, 602 June Street, Fall River, Mass.
WILLIAM C. PETERSON, 1265A Mass. Avenue, Arlington, Mass.
RICHARD J. SPENCER, JR. 492 Central Street, Saugus, Mass.
LOUIS P. VUONA, 13 Groton Place, Worcester, Mass.
JOHN W. WHITE, Cross Street, South Ashburnham, Mass.

DEATHS

THEODORE B. PARKER, April 27, 1944.
C. LEONARD BROWN, April 30, 1944.
JAMES R. GIBSON, May 13, 1944.
ROBERT J. GREER, May 12, 1944.
NATHAN C. BURRILL, May 26, 1944.
FREDERIC H. FAY, June 5, 1944.
ERNEST R. GALLAGHER, June 22, 1944.
HERBERT C. KEITH, May 2, 1944.
ALBERT E. KIMBERLY, April 21, 1944.

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.

Vol. XXXI JULY, 1944 No. 3

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

INDEX TO ADVERTISERS

COMMITTEES

1944-1945

NOMINATING COMMITTEE

Past Presidents (Members of the Committee)

Karl R. Kennison — Gordon M. Fair — Arthur L. Shaw

George N. Bailey
Walter N. Fife
Frank L. Lincoln
(Term expires March, 1945)

Raymond W. Coburn
Lawrence M. Gentleman
Donald W. Taylor
(Term expires March, 1946)

SPECIAL COMMITTEES

Program

Samuel M. Ellsworth, *Chairman, ex officio*

Edward H. Cameron
William F. Covil
George H. Delano

Frank L. Flood
Everett N. Hutchins
Herman G. Protze

Truman H. Safford
Ralph M. Soule
Donald W. Taylor

Publication

Everett N. Hutchins, *Chairman*

J. Stuart Crandall — Gordon M. Fair — John B. Babcock

Library

Allen J. Burdoin, *Chairman*

Burtis S. Brown
Thomas C. Coleman
Eugene Mirabelli

J. Donald Mitsch
Ralph M. Soule
Frederic N. Weaver

Hospitality Committee

John H. Harding, *Chairman*

Edward S. Averell
Frederick S. Gibbs

Frederick M. Joy, Jr.
Edward Wright

Relations of Sections to Main Society

Frederick S. Gibbs, *Chairman*

Charles O. Baird
Henry J. Bishop

Allen J. Burdoin
Lawrence M. Gentleman

Welfare

Ralph W. Horne, *Chairman*

Desmond Fitzgerald Award

Harold K. Barrows, *Chairman*

James E. Lawrence — Arthur L. Shaw

Sanitary and Hydraulics Section Award

Harold K. Barrows, *Chairman*

Harvey B. Kinnison — Thomas R. Camp

Designers Section Award

Arthur L. Shaw, *Chairman*

J. Donald Mitsch — Frederick N. Weaver

Highway Section Award

James E. Lawrence, *Chairman*

Alexander J. Bone — Parker Holbrook

Subsoils of Boston

Donald W. Taylor, *Chairman*

Miles N. Clair
Irving B. Crosby

Stanley M. Dore
Lawrence G. Ropes

Investment Committee

Chester J. Ginder, *Chairman*

Samuel M. Ellsworth — Harry P. Burden