JOURNAL of the BOSTON SOCIETY OF CIVIL ENGINEERS

1848
BOSTON
SOCIETY
OF CIVIL
ENGINEERS

111 YEARS

1848-1959

JANUARY - 1959

VOLUME 46 NUMBER 1

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JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 46 JANUARY, 1959 Number 1

PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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

Second-Class mail privileges authorized at Boston, Mass.

Published four times a year, January, April, July and October, by the Society
20 Pemberton Square, Boston, Massachusetts

Subscription Price $5.00 a Year (4 Copies)
$1.25 a Copy

JOURNAL OF THE
BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 46 JANUARY, 1959 Number 1

HYDRAULIC MODEL STUDY OF PROTECTIVE WORKS FOR FLEET BERTHS IN NARRAGANSETT BAY

BY PETER S. EAGLESON,* Member

(Presented at a meeting of the Hydraulics Section of the Boston Society of Civil Engineers, held on May 7, 1958.)

INTRODUCTION

THE model study reported on here had as its purpose, determination of the optimum size and location of a breakwater to protect the existing and future berthing facilities of the United States Navy's Atlantic Destroyer Fleet in Coddington Cove, Narragansett Bay, Rhode Island.

The existing and proposed piers in Coddington Cove are intended to be used as berthing facilities for the repair and refitting of the ships and the relief of their crews. Continued maintenance of seaworthiness is currently necessary in order to permit evacuation at the threat of high seas which normally accompany north and northwest winds. Examination of annual average wind records shows winds from the northwest quadrant to exceed 20 knots, 20% of the time.

The objective of the study was to determine the best size and location of a breakwater compatible with:

1. Maximum protection of ships and structures from wind waves.
2. Minimum interruption of the existing harbor "flushing" created by tidal surface currents.
3. Minimum interruption of littoral sand transport.

A detailed description of the entire study may be found in Reference (1).

* Assistant Professor of Hydraulic Engineering, Massachusetts Institute of Technology.

THE PROTOTYPE

Coddington Cove is an embayment on the western coast of Aquidneck Island near the mouth of Narragansett Bay. It can be found at the lower end of the shaded portion of Figure 1.

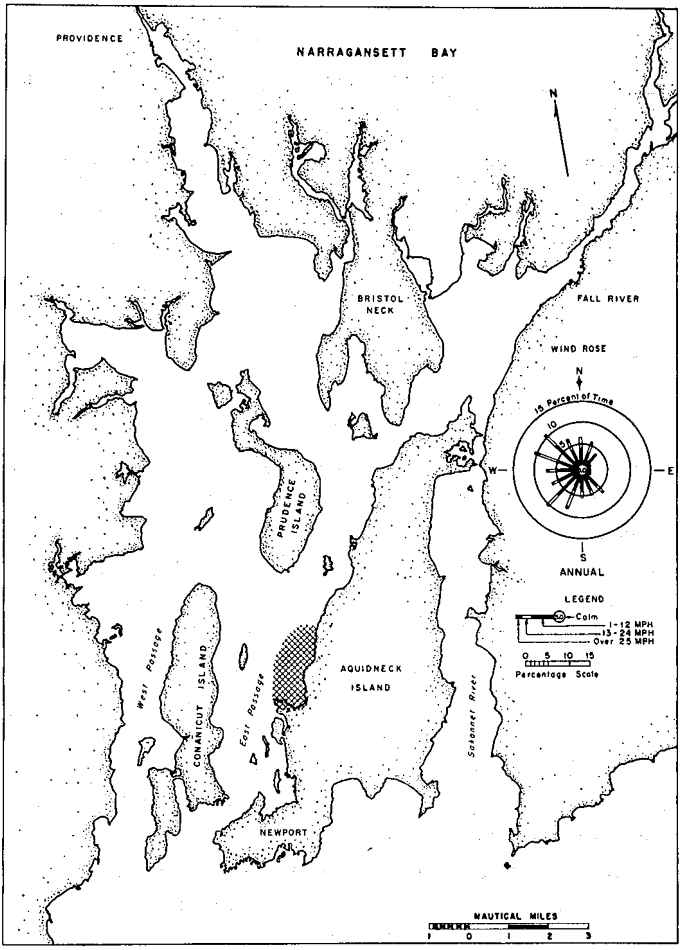

FIG. 1.—LOCATION MAP.

Harbor Structures

During World War II the Coddington Cove area became a part of the expanding U.S. Naval Base at Newport, Rhode Island and various harbor structures were constructed. These structures and existing topography along with proposed improvements are shown in Figure 2.

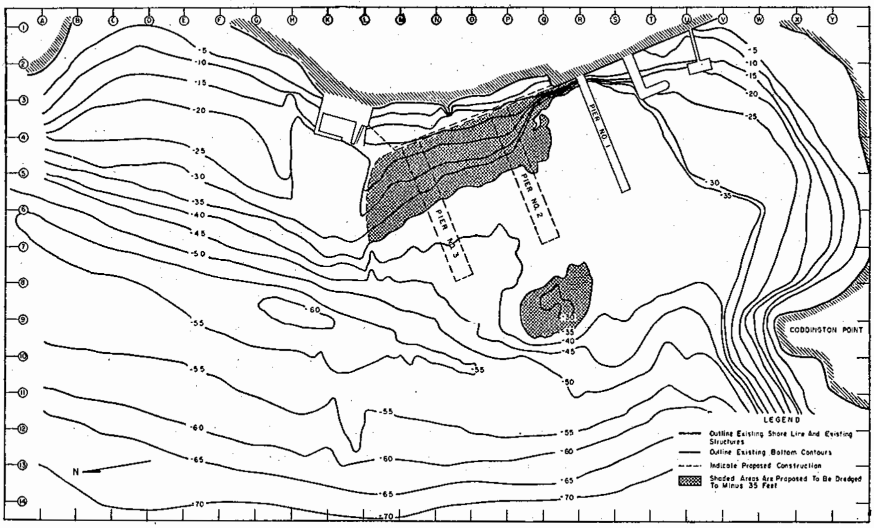

FIG. 2.—DETAILED MAP OF MODELED AREA.

Starting in the bay and working northward along the coast the first structure seen is a dock and transit shed herein called the "south basin." Both arms of this structure are of impermeable construction. Pier No. 1 is 1350 ft. long and 100 ft. wide and rests on 1638, 16 in. monotube piles. Proposed piers 2 and 3 will each be 1525 ft. long and 200 ft. wide resting on 14″ cylindrical piles spaced 10 ft. on centers. Both of these piers are designed to be built with sheet pile bulkheads along their northern side to yield protection for ships berthed in their lee until such time as a breakwater can be constructed. The northernmost structure is a dock and ferry slip herein called "north basin" and has a substructure of timber piling. The proposed rubble breakwater has a 10 ft. top width, side slopes of approximately 1¼ to 1 and is to be faced on the weather side with 7 ton stones.

Available Oceanographic Data

1. Detailed records of wind direction, intensity and duration have been obtained by personnel of the U.S. Weather Bureau's Airport Station at Hillsgrove, Rhode Island, since 1949.

2. Tidal current measurements have been made at several points in the Coddington Cove area and are listed by the U.S. Department of Commerce (2).

3. Visual inspection of the Cove and adjacent coast showed an absence of fine sand and comparison of sets of serial photographs indicated no appreciable coastal changes since the beginning of World War II.

4. The only wave data available for Narragansett Bay are those by Hicks (3) which represent a very few observations of height and direction in the East Passage.

Because of the need for information concerning significant wave periods as well as wave heights, a capacitive type prototype wave gage was constructed and installed in 35 feet of water at the end of Pier No. 1. Unfortunately, the model study was completed before the statistical analyses of the resulting measurements were available. Following, therefore, is a description of the techniques used to obtain from the available wind data, estimates of the waves which have occurred in the harbor area and may thus be expected to occur again.

Wave Statistics by Hindcasting Procedures

Hindcasting may be defined as the application of wave forecasting techniques to past meteorological data in order to determine the wave conditions which existed at that time.

A very brief description of the two most commonly used techniques, their differences and a definition of terms is in order.

Figure 3 depicts the variation of water surface elevation with time at a given point for two different conditions. Figure 3a shows the pure or "classical" wave form which repeats itself exactly every T seconds and is governed, for small H, by the familiar equation:

$$C = \frac{L}{T} = \left[\frac{qL}{2\pi} \tanh \frac{2\pi d}{L} \right]^{1/2}$$

in which:

C = wave celerity in ft/sec
T = wave period in sec.
L = wave length in ft.
g = gravitational constant, 32.2 ft/sec^2
d = stillwater depth

For deep water (d > L/2) this wave can be described completely by the height, H, and either the length or period.

The wind never generates such well defined wave trains with uniform heights, periods, or wave lengths. The energy supplied by the wind to the waves creates surface disturbances of widely varying period, length, height, phase (and also direction) with each individual

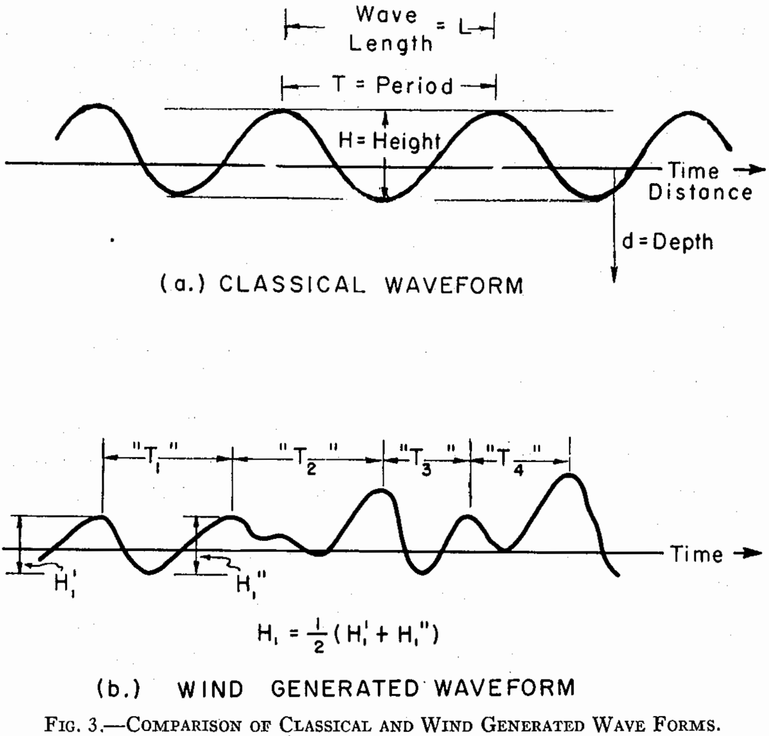

FIG. 3.—COMPARISON OF CLASSICAL AND WIND GENERATED WAVE FORMS.

disturbance following the classical laws. The composite of all these various periodic components is known as the "sea" (Fig. 3b) or the "spectrum" of ocean waves.

It is difficult to describe this wind generated wave pattern by means of only two parameters such as the "wave height" or "wave period" of some fictitious wave of classical form.

Two approaches to deep-water forecasting have grown out of an attempt to supply a simple yet adequate description of the true sea surface. Both methods begin with the equations of motion describing the transfer of energy between wind and water and both use the same data on observed wave characteristics to bridge the gap between theory and a workable forecasting technique.

The Sverdrup-Munk method (4, 5), as modified by Bretschneider (6) describes the sea in terms of a "significant" wave which is determined once the following data are available:

a. The fetch or distance over which the generating wind blows.
b. The wind direction, speed and duration within the fetch.
c. Wind conditions which may exist between the fetch and the target.
d. The decay distance from the fetch to the target.

The "significant" wave is defined by:

a. "Significant" wave height: the average height of the highest one-third of all observed waves larger than some arbitrary minimum.
b. "Significant" wave period: the average period of the one-third highest waves.

The Pierson-Neumann-James approach (7) utilizes statistics to forecast the distribution of the square of the wave height as a function of the period of the classical components making up the sea. This distribution is called the energy spectrum and its time derivative is the power spectrum. This technique also provides the two parameter description of the former method. However, it utilizes the average of all waves greater than the chosen minimum to define a representative mean height and mean "period." Component or true periods are denoted by T and significant or mean "periods" by "T."

These techniques were developed for generation in deep water and since the average depth of Narragansett Bay to the northwest from the Cove is approximately 30 feet, all generated waves having periods of 3.5 sec. or more will be shallow water waves ($d < L/2$).

Bretschneider (8) has studied shallow water generation and has found from theoretical considerations that both the significant "period" and height of the shallow water wave will be less than those of that wave which is generated by the same wind over the same fetch in deep water.

Using both laboratory and field data, however, Sibul (9) found the significant shallow water wave height to be slightly larger than the deep water value for high winds and the depth to have only a slight decreasing effect on the significant wave "period" for the range of fetch, wind speed and depth of interest in this study.

It was thus decided to use the accepted deep water hindcasting technique for prediction of significant wave height and "period."

The state of growth of waves due to wind of a given intensity has been stated to be a function of both fetch and duration. As a wind begins to blow with a given intensity across a body of water, the wave height at any given distance from the upwind shore will increase with time and reach an essentially steady state at a time denoted as the minimum wind duration for that intensity and fetch. If one then examines the steady state conditions as successively greater fetches, one finds the spectrum to be correspondingly broader. There is a certain minimum fetch and a minimum duration for this wind speed for which all possible wave components in the spectrum are present with their maximum energy. In this state, the sea is called "fully arisen."

Application

The annual wind rose for Airport Station, Hillsgrove, R. I., is presented in Fig. 1 as compiled by Fay, Spofford and Thorndike, Inc. Two directions are seen to be important, northwest and southwest.

Examining the location map the following maximum fetches are obtained:

TABLE I
Wind Fetches at Coddington Cove

Direction	Fetch to end of Pier No. 1 (Nautical Miles)
N 8° E	10.1
N 22° W	10.1
S 42° W	3

Examination of the wave data for the dates on which damage occurred (10) yields:

TABLE II
Wind Conditions Leading to Reported Damage

Date	Intensity (Knots)	Duration (hrs.)	Direction
Nov. 20, 1955	25	18	NW
Mar. 16-17, 1956	35-40	12	NNW

It is apparent from the above figures that the north and northwest directions are critical. Islands shelter the Cove from Westerly winds and the sinuosities of the East Passage apparently filter all but the shortest components of a S.W. ocean wave.

A fetch of 10 nautical miles was thus chosen for the study of expected waves.

In Fig. 4 the relationship given by Sverdrup and Munk (5) defining the minimum wind duration necessary for steady state conditions of wave growth is plotted as a function of wind intensity for a fetch of

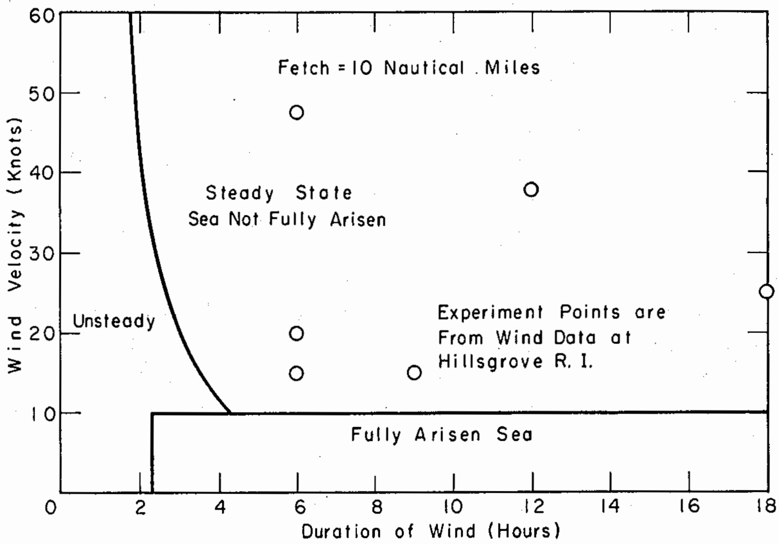

FIG. 4.—WIND SPEED VS. MINIMUM DURATION FOR STEADY STATE DEVELOPMENT.

10 nautical miles. The relationship of Pierson, Neumann and James (7) defining the maximum wind intensity and minimum duration allowable for a fully arisen sea to be obtained at the end of the fetch is also shown.

Some of the wind data including the two storms for which damage was reported (10) are plotted on Fig. 4, and clearly indicate the fetch and not the duration to limit the growth of the sea. For the winds of interest therefore, the state of the sea will be steady but not fully arisen.

Using the maximum fetch of 10 n.m. wave heights and periods were hindcast by both techniques described above. The resulting characteristics are shown in Fig. 5 as a function of wind intensity. Shown on the plot are:

(a) Significant height and "period" by Sverdrup-Munk-Bretschneider (6).
(b) Average height and "period" by Pierson-Neumann-James (7).
(c) True period, T_{max}, at which the maximum spectral energy is concentrated, by Pierson-Neumann-James (7).

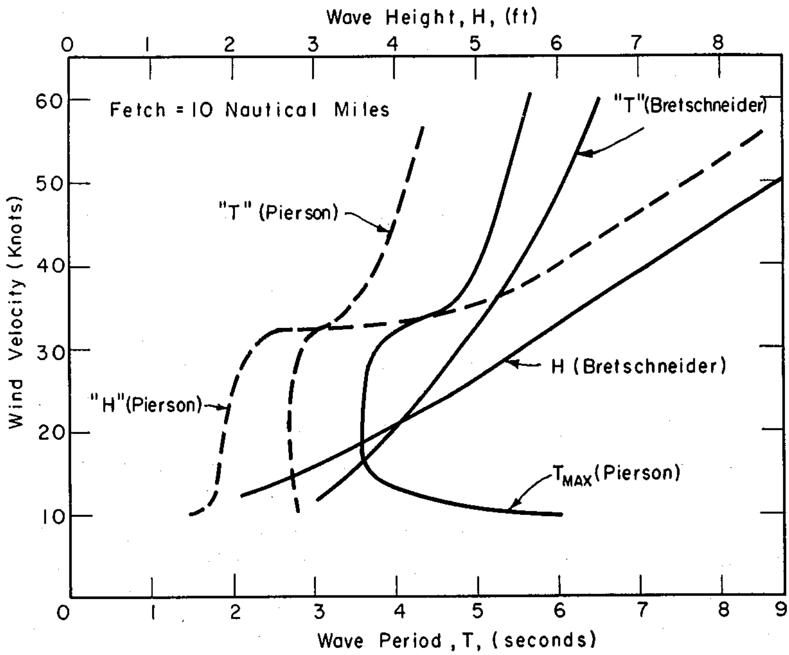

FIG. 5.—HINDCASTS OF WAVE HEIGHT AND PERIOD.

Since the experimental equipment to be used can generate only pure waves it is desired to model, for each wind speed, a wave with the period, T_{max}, at which most of the spectral energy is concentrated, and having the significant height.

Because of its broader background of successful usage the Sverdrup-Munk-Bretschneider technique was selected as the predictor. It was recognized that T_{max} will probably be larger than the given deep water "T" and also that the actual shallow water "T" will be somewhat less than that shown in Fig. 5. With these compensating factors in mind the apparent "periods" of Bretschneider were chosen as most nearly representing the component wave at which most of the spectral energy is concentrated.

Two basic waves were chosen as representative of moderate and severe storm wind conditions:

TABLE III

Prototype Characteristics of Chosen Representative Waves

Wave No.	Wind Velocity (knots)	Wave Period (sec.)	Wave Height (ft.)
1	30	5.0	5.5
2	60	6.5	10.0

The Model

Scale Selection

Model size requirements and laboratory space restrictions dictated a horizontal scale ratio:

$$L_r = \frac{L_{model}}{L_{prototype}} = 1/400 \qquad [1]$$

Since the mean depth in the vicinity of the piers is only 40 feet an undistorted model would thus have only 0.1 ft. of water in the anchorage. This is undesirably small for several reasons. In order to appreciate these reasons the basis for similitude is briefly reviewed.

The basic requirement for a satisfactory model is that those of the basic forces (gravity, viscosity, surface tension, elasticity) which are important in determining the prototype behavior be important to the same relative degree in the model. It is practically impossible to satisfy this requirement exactly in any model study. A compromise solution is to reproduce to proper scale the one (or possibly two) basic forces causing the primary prototype phenomena. Similarity is thus sacrificed to a more or less predictable degree for those secondary phenomena dependent upon other forces.

For the model study under consideration, the primary phenomena, waves and tidal surface currents are both gravity phenomena. The "Froude Law"[1] will therefore govern the model-prototype relationships yielding:

[1] By "Froude Law" is meant the obtainment of dynamic similarity where gravity forces predominate through equality of Froude Numbers, F, at geometrically similar points in model and prototype. $F = \frac{V}{\sqrt{gy}}$ where V is the local fluid velocity and y is a characteristic length (often the local depth).

$$\text{Velocity ratio} = V_r = \frac{V_{\text{model}}}{V_{\text{prototype}}} = L_r^{1/2} \qquad [2]$$

$$\text{Time ratio} = T_r = L_r V_r^{-1} = L_r^{1/2} \qquad [3]$$

Because of the presence of a free water surface, reduction of scale to model size may make surface tension forces important whereas their role in the prototype is infinitesimal.

To a vertical scale of 1:400 a 4 foot prototype wave would be only 0.1 in. high which is of the order of the meniscus at the measuring element. Accuracy of vertical measurement would thus be lost.

Dissipation of the energy of these small waves at the beaches, in the breakwater and among the pilings would be controlled by the large surface tension forces.

Similarity with respect to tidal currents is dependent upon the effects of viscosity being small in the model; i.e., model Reynolds numbers[2] must remain large. Reduction of velocities by the Froude Law produces model Reynolds numbers considerably less than prototype values.

A solution to these problems, but one which introduces new inaccuracies of its own, is to give the horizontal and vertical scales separate consideration and construct a distorted model. Such distortion must not, however, for reasons of its own, destroy the similarity between model and prototype.

Examining Equation [1], the familiar relationship governing the celerity of small amplitude gravity waves, we find that the celerity is a function of both depth and wave length (i.e. a vertical and a horizontal dimension) for $0.5 > d/L > .025$. The significant prototype range of the latter parameter is $.55 > d/L > .19$ for the two waves chosen thus we must write:

$$C_r^2 = \left[\frac{C_{\text{model}}}{C_{\text{prototype}}}\right]^2 = \frac{L_m}{L_p}\,\frac{\tanh \dfrac{2\pi d_m}{L_m}}{\tanh \dfrac{2\pi d_p}{L_p}} \qquad [4]$$

Distortion of a model concerned with shoaling waves is treacherous in this range of d/L since it will delay the effect of the bottom until

[2] Reynolds numbers, R, should be equal at geometrically similar points in model and prototype to insure dynamic similarity if viscous forces are important. $R = \frac{Vy}{v}$ where v is the kinematic fluid viscosity and the characteristic local length is the depth in this case.

the wave is closer to shore and will thereby modify the refraction pattern.

The effect of scale distortion on the refraction pattern was investigated theoretically as described below. A distortion of approximately 3 to 1 was selected as producing tolerable departures from the true refraction pattern and yet yielding measurable values of wave height.

The maximum variance of the celerity ratio at the edge of the model from its deep water value,

$$C_r^2 = L_r, \qquad [5]$$

may be seen by substituting into Equation [4] conditions accompanying the 60 knot prototype wave in 70 feet of water. This yields

$$C_r^2 = 1.03\ L_r. \qquad [6]$$

For the purposes of this study, therefore, the celerity ratio is assumed given by Equation [5].

A summary of the transfer relationships used in this study follows:

TABLE IV
Model-Prototype Transfer Relationships

Quantity	Relationship	Magnitude
Horizontal Scale	L_r	1:400
Vertical Scale	Y_r	1:120
Wave Period Ratio	$L_r^{1/2}$	1:20
Tidal Current Velocity Ratio	$Y_r^{1/2}$	1:11

Reliability

As was mentioned above, the effect of vertical distortion is to keep the model wave a deep-water and thus unrefracted wave farther inshore than in the prototype or in an undistorted model.

The effect of the distortion on the refraction pattern is cumulative thus refraction diagrams were plotted for the longest wave in order to evaluate the magnitude of the error introduced. In Fig. 6 two sets of refraction lines (wave crests) are shown; one is for normal depths as given by the contours of Fig. 2; the other is for depths 3.33 times the normal. Comparison of the results shows how the model refraction (solid lines) differs from prototype refraction (dashed lines). Serious discrepancies appear only very near the shore for the important case of northwesterly waves.

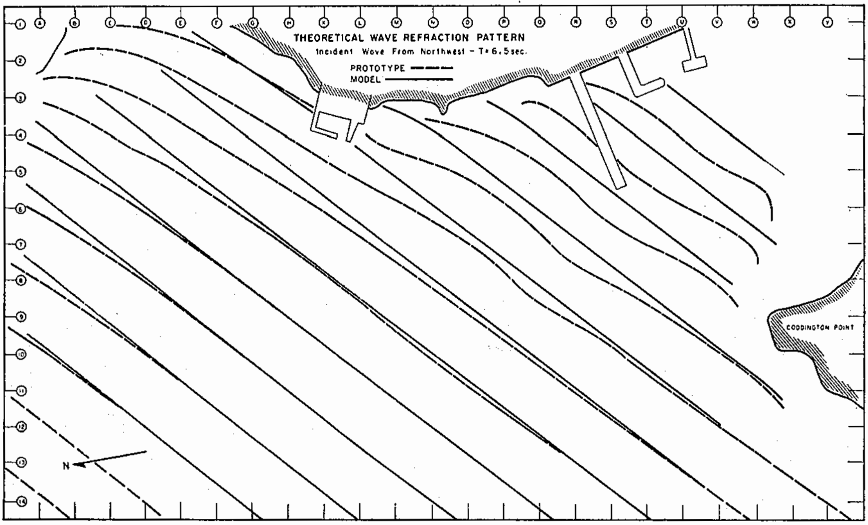

FIG. 6.—WAVE REFRACTION DIAGRAM — EFFECT OF MODEL DISTORTION.

Further detailed considerations of reliability are given in Ref. (1); however, a summary of the additional important conclusions follows:

1. Model wave lengths may be up to 16% too long in the zone of interest.
2. Periods at which abnormal wave heights are observed as a result of local resonance may be in error by as much as 8% for prototype periods up to 7 seconds.
3. Model wave heights tend to be from 10 to 15% too low but the error in estimation of significant height may be ± 15%.
4. Surface current patterns should be accurate in so far as the patterns are independent of current strength.

EXPERIMENTAL EQUIPMENT AND PROCEDURES

The experimental model basin consists of a rectangular area 50 feet long and 28 feet wide enclosed by a cement block wall 17 inches high. Waterproofing of the basin was achieved through a .040 inch coating of vinyl plastic applied to floor and walls.

The basin is filled and emptied from below through a 5″ line cut into the basin floor.

Circulation of water within the basin for tidal current simulation is provided by a small portable pump.

Model topography was laid out on female templates spaced with concrete blocks and was brought to grade by carefully trowelled cement.

All three model piers and the structure of the north basin were constructed of clear plastic using 0.035 inch diam. copper wires to simulate piling. The south basin was constructed of cement.

The model breakwater was built in six inch sections in order to facilitate revisions in its location. That portion below Elev. —10 ft.

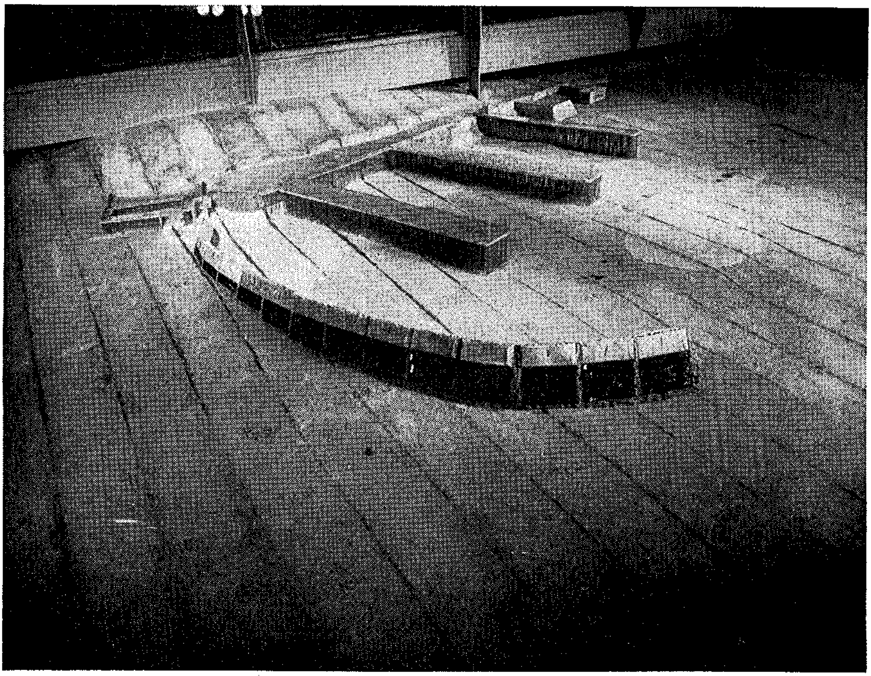

FIG. 7.—MODEL HARBOR STRUCTURES.

was made impermeable and was cast out of plaster of paris. From Elev. —10 to +10 ft. the sections consisted of wire baskets filled with 0.2 in. diam. gravel which corresponds to the 5 ton prototype stone reduced by the average scale ratio. In order to prevent the exaggerated damping accompanying the shoaling of the model waves at the very low Reynolds numbers of an undistorted model it was decided to distort the breakwater slopes as well.

Model harbor structures are shown in Fig. 7.

Waves were generated by means of a movable generator of the vertical plunger variety (see Fig. 8) having continuously variable speed and stroke.

Determination of wave characteristics was accomplished using variable capacitance wave probes as the active arm of wheatstone bridge circuits, the outputs of which were fed to a multiple channel recording oscillograph.

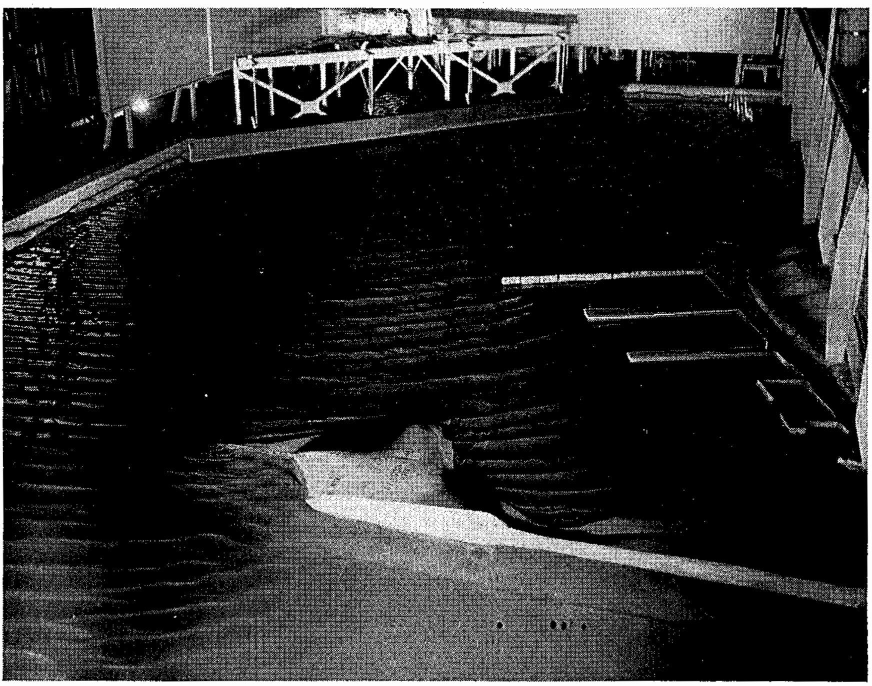

FIG. 8.—VIEW OF WAVE GENERATOR AND MODEL BASIN.

As shown in Fig. 9, the model was gridded at 1 foot intervals to provide reference points for wave height measurement. The wave generator was adjusted until the desired wave was obtained at the reference station, C-14.

The original and proposed structural and hydrographic configurations were investigated with respect to waves and tidal currents for the various breakwater locations shown in Fig. 9 and in the presence and absence of a solid bulkhead on the north side of Pier No. 2.

In order to determine the tendency of any harbor configuration toward resonant response to wave excitation, wave height measurements were taken at several selected spots (see Fig. 9) in the model while the period of the incident wave was varied in finite intervals over a wide range.

With the exception of tests concerning overtopping of the breakwater, all experiments were performed at MLW. Absence of a suitable

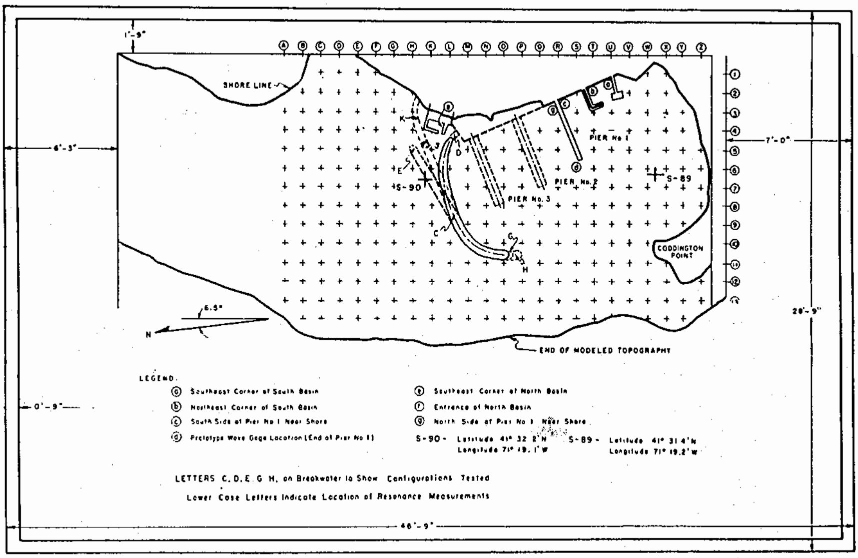

Fig. 9.—Model Configurations Tested.

tide generating mechanism precluded generation of the time varying tidal current, however, two U.S.C. and G.S. current stations lay within the area of modeled topography and as a substitute a steady ebb or flood current was provided by a pump and movable walls were positioned in order to simultaneously bring the mean current magnitude and direction to the proper value at these points. Current measurements were made by observing the motion of floating cork wafers.

Summary of Results

In the limited space available here it is not possible to present more than a brief summary of the results obtained.

Tidal Surface Currents

In Fig. 10a the ebb flow pattern of mean tidal surface currents is presented for the unimproved configuration. The main observations to be made are the presence of a large, single-celled circulation system in the Cove and the absence of observable motion through and south of Pier No. 1.

In Fig. 10b the effect of a typical breakwater configuration is shown. The pattern is now double-celled and much weaker while the area between piers is essentially stagnant with or without the Pier 2 bulkhead. Similar results were obtained for all other breakwater configurations tested.

It became apparent that the presence of the body of an effective breakwater, no matter what the disposition of its ends, destroys any slight cleansing current. It thus appears that if wave motion is to be reduced, a pollution problem exists which must be solved separately.

None of the mean current magnitudes observed were large enough to cause any navigational difficulties. However instantaneous magnitudes may, particularly near the tip of the breakwater. It must be remembered that wind and wave induced currents will also be present and may, in fact, control the net surface current pattern. Determination of their magnitude was not within the scope of this study. However, their effect on the pollution question is apparent. The prevailing winds and thus the waves are from the northwest. Thus all surface refuse will be piled up in the stagnant area south of Pier 1. The negative conclusions from tidal current studies conducted in the absence of wave motion are therefore conservative from the pollution standpoint.

Wave Height

All data presented in this section will be for the wave resulting from the 60 knot northwest wind (herein called "Northwest No. 2").

A map of the water surface in the unimproved case is shown in Fig. 11a.

For comparison Fig. 11b shows the effect of the breakwater for an improved configuration. As might be expected the waves diffract around the outer end of the breakwater and decrease in amplitude as their energy spreads out over a longer crest. The breakwater length can thus be seen to be critical in determining the degree of protection afforded the outer berths of piers 1 and 2. Wave heights outside the breakwater are increased in the presence of the breakwater due to wave reflection. This tendency is somewhat exaggerated in the model due to the distorted slopes.

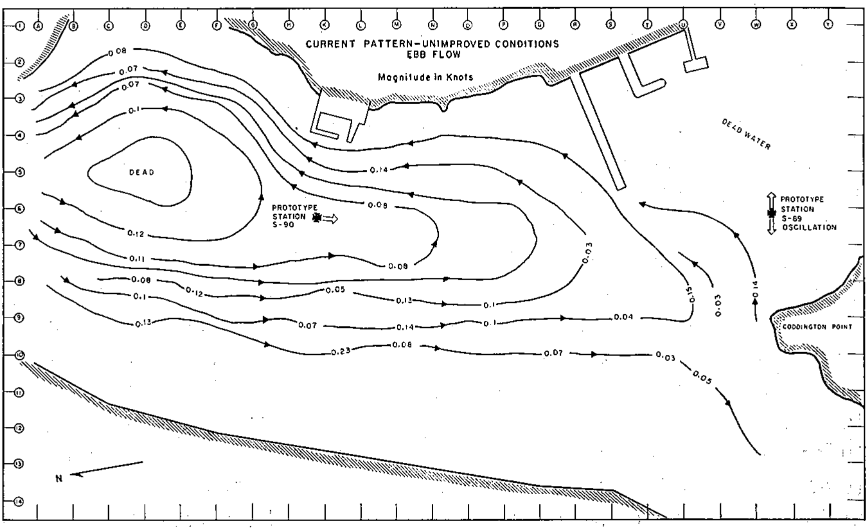

FIG. 10a.—MEAN TIDAL SURFACE CURRENTS, EBB FLOW, UNIMPROVED.

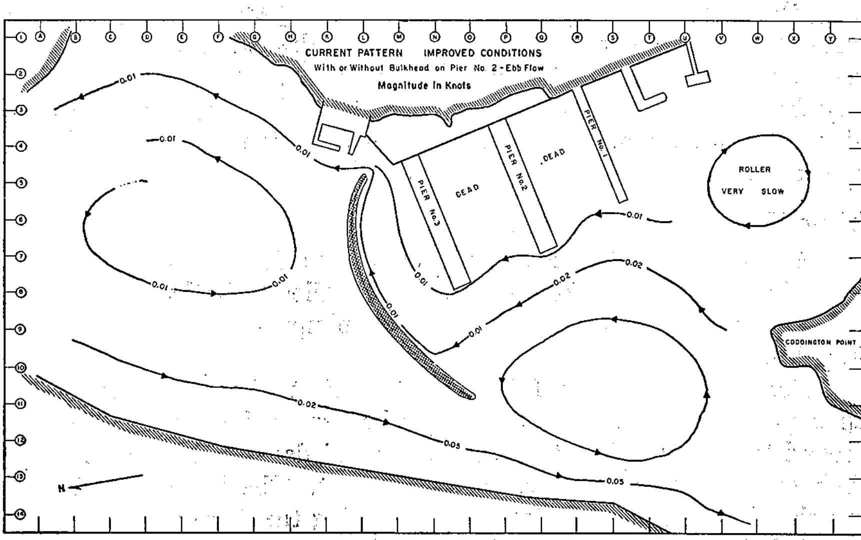

FIG. 10b.—MEAN TIDAL SURFACE CURRENTS, EBB FLOW, IMPROVED CONFIGURATION.

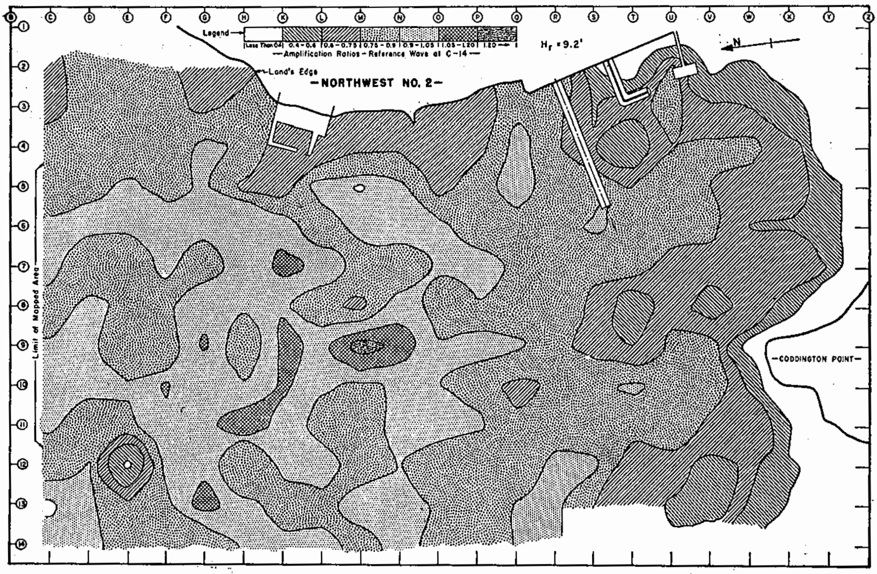

FIG. 11a.—WATER SURFACE MAP, UNIMPROVED CONFIGURATION.

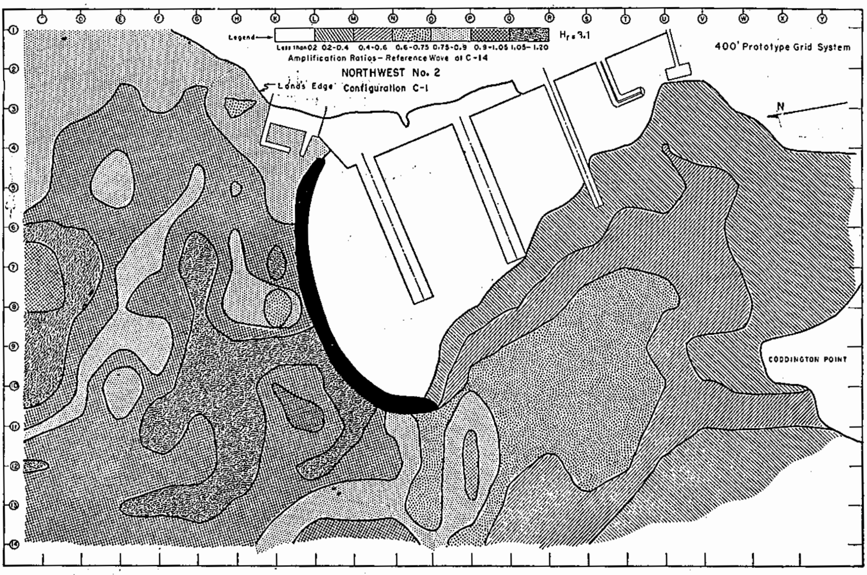

FIG. 11b.—WATER SURFACE MAP, IMPROVED CONFIGURATION.

In Fig. 12 the effects of the sheet pile bulkhead along the north side of Pier 2 are shown clearly. The bulkhead side of the pier is rendered untenable due to reflections but conditions at berths in the lee of this bulkhead are considerably improved. Also discovered but not shown is the fact that enough of the wave energy reaching the bulkhead even in the presence of a breakwater is reflected northward to double the heights experienced at Pier No. 3.

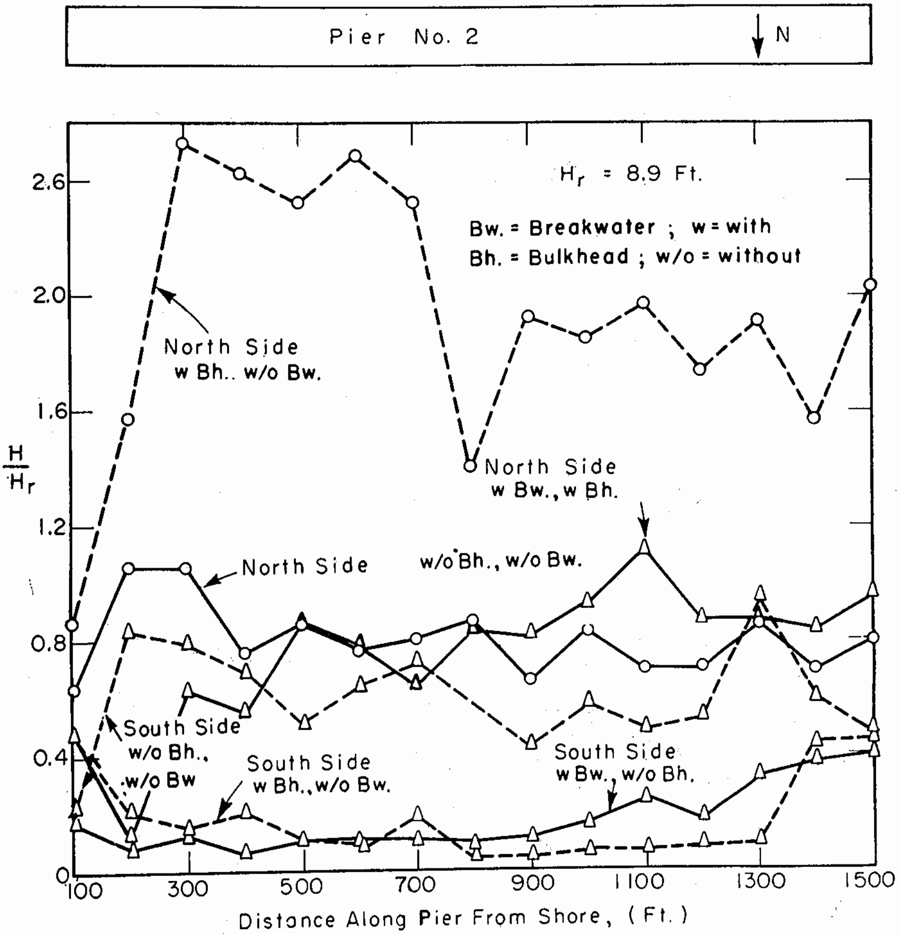

FIG. 12.—SOME EFFECTS OF BREAKWATER EXTENSION ON WAVE HEIGHT IN THE OUTBOARD BERTH OF PIER NO. 1.

Also shown in Fig. 12 is the improvement in wave height along Pier No. 2 due to the presence of the breakwater in Configuration D. Similar results are obtained for the other two piers with the exception that the decrease in protection toward the outboard end of the pier is greater for Pier 1 and less for Pier 3.

In order to decrease wave heights in the outboard berths of Piers 1 and 2 the breakwater was extended into deeper water. Figure 13

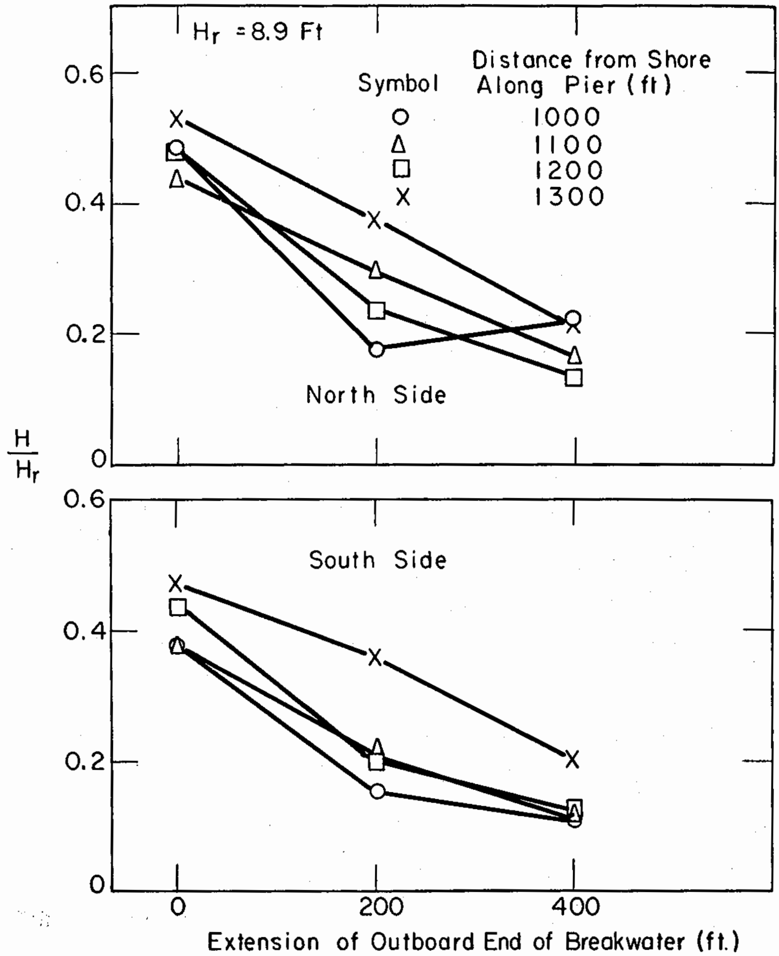

FIG. 13.—EFFECT OF BREAKWATER EXTENSION ON WAVE HEIGHT IN THE OUTBOARD BERTH OF PIER NO. 1.

shows the resulting continual reduction in wave height obtained at various points along Pier 1. Similar results were obtained for Pier 2.

It was determined from personnel of the U. S. Navy that a pierside wave height of 2 to 4 feet was the maximum that could be withstood without evacuation of ships. On the basis of reducing all pierside wave heights to a maximum of 3 feet for a 60 knot wind wave the optimum configuration tested was "K" of Fig. 9 with a 200 foot extension on the outboard end.

A fairly detailed study of the frequency response of the various geometric elements indicated serious resonance problems only in the north and south basins at wave periods less than 10 seconds. In a non fully developed sea the period at which the most energy is concentrated (T_{max}) is essentially the maximum period in the spectrum. It is not likely therefore that in this case any significant wind wave energy exists at periods greater than 10 seconds.

Since the mean depth of water along the proposed breakwater centerline is about 40 feet for configuration K the possibility of an economy through lowering the breakwater height was studied.

The only adverse hydraulic feature of overtopping of the breakwater by waves will arise when enough energy passes the breakwater to cause objectionable ship motions at the piers.

For a given incident wave height as the breakwater height is reduced the energy passing over the breakwater will pass through a current producing stage and into a wave producing stage. It is doubtful that the currents so produced will be objectionable with respect to either ship motion or navigation.

Since the worst condition will arise at high tide, these tests were run with a higher still-water level than the previous tests. The U.S.C. and G.S. (12) gives the average spring tide range to be 4.4 feet at Newport.

In addition, however, the wind tide should be included. Assuming the bay (along the northwest fetch from Coddington Cove) to be a bay of reasonably uniform depth and width, the wind tide as given by Sibul (11) is approximately 0.75 feet for the 60 knot wind.

Thus for these tests the still-water level was held at +5.15 ft. and wave heights were measured along the north side of Pier No. 3.

To note the approximate effect of reducing the breakwater height by 4 feet the same test was performed with the still-water level at +9.15 ft.

In Figure 14, some of the results of the overtopping studies are presented. The wave heights along the north side of Pier No. 3 can be seen to increase as the breakwater freeboard is decreased. When interpreting these results it must be remembered that the model breakwater is distorted so that its slopes are three times as steep as the prototype. This will increase the percentage of wave energy reflected but should also decrease the percent dissipated and also increase slightly the percentage transmitted. It is believed that the results are slightly conservative. It can be said with assurance that the breakwater height cannot be reduced to Elev. +6 without making the north side of Pier 3 untenable for 60 knot wind-waves.

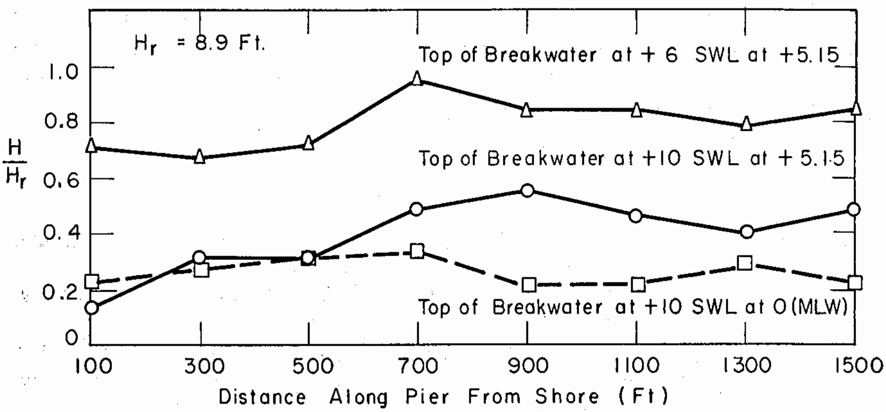

FIG. 14.—EFFECT OF BREAKWATER HEIGHT ON WAVE HEIGHT ALONG PIER NO. 3—CONFIGURATION D.

CONCLUSIONS

A. The optimum breakwater location tested appears to be configuration K (Figure 9) with the addition of a 200 foot extension on the outboard end.

B. Presence of a solid bulkhead on the north side of Pier 2 protects only the inboard two thirds of Pier 1 and the south side of Pier 2.

C. Presence of a solid bulkhead on the north side of Pier 2 is detrimental to use of the north side of Pier 2 in all waves and increases wave heights along Pier 3.

D. The top of the breakwater cannot be lowered to Elev. +6 without making Pier 3 untenable in a 60 knot northwest wind.

E. Presence of the body of an effective breakwater so disrupts tidal surface currents as to present a serious pollution problem requiring independent solution.

F. Littoral sand transport is not significant and need not be a consideration in location of the inboard end of the breakwater.

Acknowledgments

The model investigation reported herein was sponsored by the Department of Public Works, First Naval District, U. S. Navy. Fay, Spofford and Thorndike, Inc., was the engineer-architect.

The investigation was carried out at the Hydrodynamics Laboratory of the Department of Civil and Sanitary Engineering, of the Massachusetts Institute of Technology under the direction of the author. The experimental work was performed by Messrs. Thomas A. Marlow and Cheng-tien Luke, Research Assistants, and Mr. Francis J. Turpin, Research Engineer.

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SUBURBIA: SATELLITE OR SPRAWL

BY ROLAND B. GREELEY*

(First of two papers presented at a meeting of the Sanitary Section, B.S.C.E., held on October 1, 1958.)

MY MAJOR objective in this paper is to raise with sanitary engineers some basic questions in the borderline zone between sanitary engineering and comprehensive planning. City planners, and the political entities which they serve, are making major decisions on assumed answers to these questions every day. I believe our assumed answers are, at least, in the right direction; but I am not sure. If the sanitary engineers can give us reliable information we may be able to avoid very serious mistakes. As a minimum, these mistakes, if they be mistakes, will cost our metropolitan areas many millions of dollars in the future. As a maximum—well let us just say that the answers to these questions could quite reasonably dictate the future patterns of our urban areas for generations.

I shall present the questions in some detail later on. For now, let it suffice for me to say that they all relate to the devices which may be available to carry water to, and remove sewage from, homes, businesses, and factories of varying density patterns in metropolitan areas of the future.

During the past century we have evolved from a dominantly rural to a dominantly urban society, from a country of farms and relatively small industrial and cultural centers to a vast complex of metropolitan areas, where, at least in the Northeast, the rural farm areas are barely incidents in a vast urban sprawl.

Advances in technology, in leisure time, in standards of living, in attitudes toward nature and open space have made this possible, or according to some, have forced this pattern upon us.

When the factory was tied to the prime mover by belts and pulleys, and the worker's home had to be within reasonable walking distance of the factory, our cities had to be small—10 or 15 square miles in area. Relatively high density of settlement was essential if the city was to grow to a population of 100,000.

As ease in transportation of fuel, and worker-commuting by rail

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developed, the effective size of the city became multiplied by a factor of ten, or more. The city could expand to a population of a quarter or half a million with a significantly lower density of land use—with much more open space—without increasing inconvenience and without impairing the essential functions of the urban area.

Then along came electric power, the automobile and the bus, the transportation of goods and materials by truck, and the effective area of the City, or metropolis, was again multiplied by a factor of at least ten.[9] We can now have a metropolis of ten million just as easily as we could have a city of 100,000 a century ago. Or we can have a metropolis of 2½ million, as in Boston, at only one-fourth the average density of the city of 100,000 at the time of the Civil War.

As economic and industrial activities in our Metropolitan areas continue to decentralize, and as the speed and flexibility of commuting increases, we approach a situation where most people could live at what have been considered essentially rural densities and still be within reach of virtually all of the advantages which we have associated with urban centers. For example, all of the population increase forecast for the entire United States in the next fifty years could be housed on one-acre lots and still be within an hour's drive of one or more of the major existing urban centers along the Atlantic Seaboard—from Portland and Albany to Richmond and Norfolk.

Such a pattern may be utterly ridiculous. But the fact remains that significant numbers of us want to live on large lots in the outer suburbs; and in nearly all respects it is quite possible for us to do so.

"In nearly all respects." The qualification recognizes, perhaps most significantly of all, the fact that technological changes in the field of water supply and sewage disposal tend more and more to favor high rather than low densities. The changes have all tended to increase the demand for water, and the load of sewage; but these changes have resulted in greater, rather than less, difficulty in serving relatively low-density areas.

There are several factors involved here, but I want to emphasize only one: the means of sewage disposal in low-density residential areas.

The Massachusetts Department of Commerce has recently conducted a study of "The Effects of Large Lot Size on Residential Development" (published by Urban Land Institute, 1958). A major element of that study was the analysis of the variation in cost of development lots with size of lot. As lots increase in frontage and area the cost does not increase as a straight-line function, since certain elements of cost can be reduced—width and weight of pavement, sidewalks, etc. The largest single increment is the cost of the public sewer system. Roughly speaking, the lot which must have public sewers will cost about $1000 more than the lot which can provide its own sewage disposal (the breaking point in this Report is assumed to be when the lot attains a size of about one acre). Assuming a "reasonable" raw-land cost of $1000 per acre, the capital cost of a half-acre lot with public sewers is slightly more than the cost of a one-acre lot with adequate septic tank. But if the one-acre lot turns out later to need public sewers, then the gross cost exceeds the cost of a two-acre lot, if we assume that the larger lot would never need public sewers.

You may say, rightfully, that I am getting excited about a problem which, at most, only amounts to a couple of thousand dollars per lot. My retort is that whether this amont of money be large or small, it is one of the most significant factors determining our suburban patterns today; that guesses as to whether a lot of a given size will, or will not, need public sewerage are dictating, time after time, our zoning policy, our subdivision policy, and therefore our basic plans of streets and land use for a matter of generations or even centuries.

In the highly critical band of residential densities ranging from two or three families per acre to only half a family per acre, we are confronted with the following premises:

(1) Any layout that really permits use of private sewage disposal means for each lot is likely to be cheaper (capital cost) than any layout which does not.

(2) Any layout which is most favorable for private means of disposal is likely to be the most expensive to convert to sewers at a later date.

(3) If a sewage system is to be needed in the anticipated future, then some form of clustering (regardless of the over-all density) is more economical.

(4) Unless really small public disposal systems are likely to be feasible, then all clusters must be so laid out that they will fit into a single massive sewerage system.

Obviously many factors other than the cost of sewage disposal do, and should, enter into decisions as to what the basic pattern of suburban residence should be. Nevertheless, with emphasis currently on the desirability of low-density, and with marginal costs of land a highly-important factor in the competitive market, the economics of sewage disposal actually plays a dominant role. Many decisions are being based on possibly false assumptions as to what type of disposal will work in the long run; many of the decisions might be significantly altered if it were known what the sewerage pattern will eventually have to be.

This makes the following questions, to which I referred above, highly important:

(1) What types of sewage disposal, at what density patterns, can be relied upon?

(2) Can we know enough about long-range costs of sewerage to make realistic gambles?

(3) Are treatment plants (such as "package plants") for small clusters of homes (or industry or business or schools) going to be reasonably inexpensive, and dependable?

(4) Can we hope to understand soil conditions well enough, *before* we build, to prophesy safely where private disposal systems will work?

(5) Is there reason to assume that sanitary engineering technology will evolve fast enough so that septic tanks or cesspools installed correctly now will continue to be useful indefinitely, assuming only such maintenance measures as can be enforced?

(6) What are the criteria governing present correct installation, in terms of soil types, lot sizes, and over-all densities?

I repeat what I said at the outset: Decisions are being made, and will have to be made continuously, which depend upon the answers to these questions. Regardless of whether the decisions are made by planners, health officials, or sanitary engineers, it is up to the latter to provide the technical answers. These answers may be the determining factor as to whether our suburbs expand in a pattern of really low-density sprawl, of many small clusters surrounded by open space, or of tentacle-like extensions from the main core. Whether they are the determining factor or not, they should be available to the planners, and even more important to the public, so as to permit the orderly and efficient growth of our suburban areas.

Let me close with a couple of simple analogies—simple but I think fair:

The highway engineers might readily conclude that the only safe intersection is a full clover-leaf; and that lacking this the next best bets are either stop-lights or a policeman. Still it is recognized that for relative safety and efficiency we can reasonably design simple crossroads with such inexpensive safety and regulatory devices as signs and painted lines, for the vast majority of our worst intersections.

Similarly, most sanitary engineers of twenty years ago plugged for "pure" streams. Then, beginning in the early 1940's, there evolved the concept of basing pollution control on relative rather than absolute objectives, and the systems of stream classification were worked out which are, I understand, now being applied. A class "C" stream is not absolutely healthy, or absolutely safe, or absolutely satisfactory; but it is "acceptable" in many actual situations.

These are both practical compromises with the ideal, recognizing that society seems to prefer to take calculated risks rather than pay for ideal conditions. We are now taking risks, so far as sewage disposal and health are concerned, in laying out new subdivisions, new suburbs. But in most cases these risks are based on intuition rather than calculation. The aid of the sanitary engineers is gravely needed to help calculate the risks—to assess the relative dangers and relative costs of the several practical alternates—and thereby give planners and developers a more reliable basis for the all-important decisions which must be made.

SUBURBIA: SATELLITE OR SPRAWL

BY WORTHEN H. TAYLOR*

(Second of two papers presented at a meeting of the Sanitary Section, B.S.C.E., held on October 1, 1958.)

A SAFE and adequate water supply is the first requirement of suburban development. Such a supply is usually obtained by extension of existing public mains, development of a new common supply or the installation of individual local wells. Developers prefer to utilize existing supplies wherever possible. Consideration must be given to the adequacy of the source to meet the increasing demands of the present population and the expected population of the proposed development. Of nearly equal importance is the capacity of the distribution system. New developments are generally outside of the area presently served. Approaching the ends of distributing lines the ability to maintain adequate pressures is frequently difficult and thus the added load may require strengthening of the entire system. There are few more exasperating events than to find no water on the second floor as company comes to warm the new house. Many developers are willing to extend existing mains but are not in a position to go back into the distribution system to increase pipe sizes or provide interconnections which may be necessary.

The development of a new supply in this State offers many problems, both technical and administrative. There are relatively few remaining sources of surface water of satisfactory quality available in Massachusetts without providing considerable treatment as most of our surface water resources have been appropriated already for water supply or other uses. In instances where a source of surface water supply is available in the vicinity of a suburban development it is quite probable that treatment in excess of plain chlorination would be required because of direct pollution or development on the watershed. Many of the ground waters of Massachusetts are not satisfactory for direct sources of water supply due to a high content of iron and/or manganese. Thus even if a ground water source of adequate quantity were available there is considerable probability that treatment would be required. Frequently a developer would find it un-

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reasonable to set aside a sufficient area around a well for proper development and sanitary protection. In instances where treatment would be required, either a surface supply or a ground water supply, the State Department of Public Health would feel that the operation of such facilities should be in the hands of a responsible public agency, such as a municipality, a water supply district or a water company operating under the jurisdiction of the Department of Public Utilities. Water districts may be formed only with the approval of the Legislature and the residents of the area. When thus established authority is granted to issue bonds for construction and to raise funds by taxation and other means available to municipalities. Thus the continued maintenance and operation of the facilities are guaranteed and the Department of Public Health maintains its normal jurisdiction over the quality of the water and the sources of supply.

The possibility of providing a water supply from individual wells with sewage disposal on the same lot is restricted to areas where the lot size will physically permit a considerable distance between such facilities. Under favorable conditions it has been felt generally in Massachusetts that such a minimum distance should be 50 feet. With the advent of synthetic detergents it is felt that this minimum may not be adequate and it may be necessary to increase minimum recommendations to 100 or more feet. It should be emphasized that such requirements are minimum, not optimum or adequate in many instances.

Disposal of sewage most frequently offers problems to suburban development. Where possible to do so, sewage should be disposed of by means of the municipal sewerage system. Consideration should be given to the adequacy of the sewerage system and the sewage treatment facilities. Federal grants-in-aid are available to municipalities for the construction or extension of sewage treatment facilities, but no such funds are available for common sewers. Federal aid is forthcoming for the construction of interceptors and pumping stations in certain instances. A survey of the practices of municipalities indicates that the developer is usually required to bear the entire cost of sewer extensions to serve the newly developed area. The operation of the system thereafter may be placed on the general tax levy or a sewer usage charge directly against the resident.

Where it is not possible to connect each individual home directly to the municipal sewerage system, consideration should be given to providing a common sewerage system and either pumping the sewage from a central point to an existing sewerage system or providing treatment facilities at a central location. In either instance the Department of Public Health requires adequate assurance of the proper maintenance and operation of facilities. Where only pumping facilities are required the municipality will frequently assume the title and thereafter operate and maintain the works if the system meets local engineering requirements. In other instances proper operation may be by means of formation of a sewerage district similar to a water district or by means of trust deeds where a responsible agency, such as a large banking institution, guarantees proper maintenance and operation of works. The General Laws of the Commonwealth do not provide for the operation of sewerage or sewage treatment facilities as public utilities.

The use of individual sewage disposal facilities on each lot is dependent very considerably on the size of the lot, character of the soil, the elevation of ground water, and the proximity of sources of water supply, buildings and property lines. Such works generally consist of septic tanks and subsurface leaching works, although the use of cesspools is approved in certain favorable instances within the Commonwealth. Of great importance are the character of the soil and the elevation of ground water. Developers find it necessary to utilize more and more marginal land and thus provide housing in clayey soils and swampy areas which heretofore have been considered unsatisfactory for building purposes.

No method of local sewage disposal has been developed to date that may be considered suitable for an unlimited period of time. Our present local disposal facilities are generally designed to operate for a period of some 20 or so years. It is anticipated that they must be replaced, at least in part, at some future date and thus ample room for at least duplicating the original facilities should be provided.

Of utmost importance is the maintenance of the facilities, particularly the septic tank, which must be cleaned from time to time. The State Department of Public Health does not look with favor upon the installation of sand filters for individual households but does consider the installation of such for schools and other public buildings. In such an event the effluent must be discharged to a suitable body of receiving water.

It is well understood that our surface waters cannot be maintained in their pristine glory. There are no longer sufficient areas to permit disposal of liquid wastes to the ground in our urban areas; thus the ultimate disposal must be to a stream or the ocean. The people generally demand that our surface waters be free of obnoxious substances and pleasing to the senses. This State believes that maximum use should be made of its water resources, taking advantage of the ability of the streams to purify themselves through time, reoxygenation and other natural processes. Since waters of various degrees of purity are required for various purposes the State, through the New England Interstate Water Pollution Control Commission, classifies its waters in accordance with their proposed highest use. Domestic water supply requires water of highest purity and thus is designated Class A. Such waters are suitable for domestic purposes with no treatment other than chlorination. Class B waters are generally suitable for bathing, game fishing and other similar uses. Class C waters, the largest group of our waters, are suitable for recreational fishing and other uses. Such waters may receive effluents of sewage and industrial waste treatment plants but must always be maintained aesthetically acceptable. Class D waters are suitable for power development, shipping, etc. but must be kept out of a nuisance condition. Streams in which nuisance conditions prevail continuously or from time to time are presently in Class E but every effort is being directed to eliminating sources of pollution or providing sufficient treatment to raise water quality to an acceptable level. Thus it may be seen that discharge of treated effluents to a stream, and the degree of treatment required is dependent on the natural ability of the receiving water to assimilate the waste within its assigned classification based on the proposed highest use.

The suburban developer should understand the powers, duties and responsibilities of various public agencies. From the public health angle the most important of such agencies are the board of health and the State Department of Public Health. When a developer proposes to subdivide land he must submit plans to the local planning board, which in turn refers matters of water supply, sewage disposal and drainage to the local board of health. No subdivision plan may be approved if the board of health disapproves any of the facilities for these purposes. The size of lot is not the direct responsibility of the local board of health except that such a board may establish a minimum size of lot consistent with the needs in regard to water supply and sewage disposal. The board of health may, after a suitable investigation, decide upon a minimum area for sewage disposal for an average size house. This area requirement would vary very considerably with the character of the soil. Generally speaking the board has a right to establish a minimum lot size which would provide for location of a private well, where necessary, at a safe distance from any sewage disposal facilities which might be needed now or in the anticipated future. In one instance a Massachusetts Superior Court rules that a board of health might reasonably assume a minimum area to be devoted for sewage disposal facilities, add 50 per cent for expansion of the family and double this resulting area to provide for duplication of all of the facilities if and when required. A minimum lot was thus established, knowing the legal requirement for setbacks from the sidewalk, average size of house, an average size sewage disposal system, providing for a local water supply at a minimum of some 50 feet from the disposal facilities and then adding enough area to provide for the rebuilding of the sewage disposal facilities. A board of health may only establish a minimum size of lot to maintain proper facilities for health and sanitation. It may adopt rules and regulations relative to the construction and location of wells and the construction, size and location of sewage disposal facilities. It may also adopt housing standards. In the absence of such standards the board of health may exercise authority in the field of housing only so far as it pertains to the general cleanliness of the premises, the conditions of floor, ceilings and such, freedom of the cellar, basement and walls from dampness, the operation of water closets and heat generating equipment.

The board of health has jurisdiction over all sources of nuisance and causes of sickness. Under this general authority it may adopt rules and regulations and cause compliance. Inadequate sewage disposal facilities are considered nuisances and thus within the jurisdiction of the board. It is of interest to note that an order of the board must be complied with before there may be review by the courts.

The State Department of Public Health in Massachusetts advises local boards of health upon request in matters of water supply, sewage disposal and drainage. The Department has authority of approval of sources and methods of treatment of public and semi-public water supplies and advises the Department of Public Utilities in such matters where the system is a public utility. The Department is required to advise cities, towns and industries relative to methods of sewerage and sewage treatment and approval of the Department is required before the establishment of such facilities. The Department is the water pollution control agency of the Commonwealth and has authority to require the abatement of sources of pollution of all streams, lakes, ponds, and other waterways.

The Department is now authorized to establish a sanitary code, the enforcement of which will be the duty and responsibility of the local board of health but appeals from actions of the board or failure of the board to act may be referred to the Department of Public Health for decision prior to court action.

Public sewerage systems may be constructed under authority granted in the General Laws but more often are constructed under authorization of special acts of the legislature. Such acts establish the maximum borrowing capacity of the municipality or responsible agency for construction purposes. Such acts also provide for considerable latitude in the establishment of fees and methods of collection.

Under Massachusetts law all municipal bonds are of the general obligation type, thus pledging the full responsibility of the municipality. Revenue bonding is not generally authorized for municipal use in this state.

There are no federal or state funds available for construction of water supply facilities but Federal aid is available as an interest-free loan to municipalities for advanced planning for public works. Such funds must be repaid to the federal government upon the start of construction. Under Public Law 660 the water pollution control act, federal aid to the extent of 30 per cent or $250,000, whichever is the lesser, is available for construction of sewage treatment facilities. Such funds are only available when the sewage or other liquid wastes receive treatment which will result in the substantial removal of the settleable solids. Such funds are available for new construction of treatment works, treatment plant, outfalls, interceptors and pumping stations but are not available for construction of common sewers or outfalls in instances where treatment is not provided.

The sanitary engineer is currently called upon to provide improved methods of sewage disposal for individual lots. Septic tanks generally result in the reduction of suspended solids but a very considerable amount of organic solids are carried into the leaching field, resulting in the clogging of the soil. Such clogging usually occurs at the interfaces between the disposal facilities and the surrounding soil. Much work is needed to develop replaceable interfaces or other devices which would greatly increase the usable life of disposal facilities. It is possible that recirculating systems may be developed so as to permit purification of the sewage and waste water and their reuse, thus providing a nearly self contained system. To date such systems have resulted in the production of large quantities of ammonia and their use has been retarded because of the lack of esthetic appeal. Prefabricated sewage treatment plants are now on the market. The standards of design and the results of the operation of such facilities are not adequately known so that many responsible agencies refuse to approve installations within their jurisdiction. Improved methods of removal of suspended solids and B.O.D. are needed.

One of the major problems that must be solved if sewage is to be disposed of by treatment plants discharging effluents to local streams is the removal and reduction of phosphates. This essential element is present in the effluents of sewage treatment plants and its presence in receiving waters generally results in the growth of algae, aquatic weeds, and in some instances obnoxious fungi and bacterial growths. Sewage and waste waters have always contained traces of phosphorous but the use of synthetic detergents has resulted in much higher concentrations than previously anticipated. It is not anticipated that use of such materials will lessen but that their use will expand both for home and industrial use. A suitable method of reduction in phosphorous in the effluents of sewage treatment plants has not been developed at the present time. Much time and study are currently being spent in a search for a suitable means of chemical control of the algae and weeds which naturally result from the presence of added phosphates. It is probable that the real answer lies in removal of the cause rather than subsequent control.

In summary, if suburban development is to provide a healthful outlet for our expanding population, ways and means of providing responsible agencies for the maintenance and operation of both water supply and sewage disposal facilities are necessary. A very searching inquiry is needed in the field of State and Federal aid for the construction of such facilities.

The sanitary engineer with the help of the laboratory can cope with the technical problems of water supply, sewage treatment and disposal, but added emphasis must be placed in fields of technical research and the administrative procedures of financing, maintenance and operation must be reviewed if suburban development is to proceed in the best interests of the general public.

PROGRESS REPORT ON DEVELOPMENT OF THE ALLEGHENY COUNTY SANITARY AUTHORITY FACILITIES AT PITTSBURGH, PENNSYLVANIA

BY EDWIN B. COBB,* Member, and HENRY R. WHEELER, JR.,** Member

(Presented at a meeting of the Boston Society of Civil Engineers, held on September 24, 1958.)

THE task of spending $100,000,000, the total cost of these facilities of the Allegheny County Sanitary Authority, presents interesting possibilities, and in this paper we will describe one way it is being done.

The privilege of addressing you should have gone to my partner, the late Frank L. Flood, a vice president of our Society at the time of his death last February. Mr. Flood had been intimately connected with the project since 1947 continuously until his death. He was the principal engineering consultant for the entire project.

The Allegheny County Sanitary Authority was formed to consider the possibility of collecting and disposing of the waste waters from the 102 communities, which include and surround the city of Pittsburgh, Pennsylvania, and more than 90 industries in Allegheny County. The Authority was made possible by the Pennsylvania Municipality Authorities Act of 1945.

The cost of constructing and operating the system will be met by sewer service charges, with no part of the revenue derived from taxation. Since the various communities were not required to subscribe to the service offered by the Authority, certain communities refused to be served, as might be expected. The system, as presently constituted, will serve 70 municipalities, with an estimated population in 1970 of 1,400,000, and 18 industries.

In this paper we will present a brief description of the project and of the organization of the Authority.

Metcalf & Eddy served as consulting engineers on the preliminary design of the facilities. Later they prepared the contract plans and specifications for the sewage treatment plant, outlying pumping facilities, and for about 30 miles of intercepting sewers of the approximate 65-mile system. The remainder of the intercepting system was designed by the Authority's own engineering staff.

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* Partner, Metcalf & Eddy.
** Project Coordinator, Metcalf & Eddy.

We are currently engaged in providing general supervision of construction for our designed contracts and advisory service under the Trust Indenture, established by the bond issues. The Authority is providing resident inspection of construction with its own forces.

Metcalf & Eddy are also under contract to supervise operation of the treatment plant during the first two years of operation.

In a paper before this Society, a description of the Authority's system was presented by Mr. Stanley M. Dore and discussed by Mr. Flood. Mr. Dore, a member of this Society, was formerly Deputy Chief Engineer of the Authority engineering staff. The paper and discussion were published in the April 1953 issue of the B.S.C.E. Journal.

A number of other papers covering various features of the project have been published in other technical publications, and some are listed at the end of this paper. Many of these papers have been prepared by Mr. John F. Laboon, Chief Engineer of the Authority and his most recent paper, titled "Controlled Submergence of Pittsburgh's Deep Sewers", was given before the October 1957 meeting of the American Society of Civil Engineers in New York.

It is difficult to be brief in reviewing the history of the Authority, without repeating material already covered by other papers. The Allegheny County Sanitary Authority was created in 1945 and incorporated in March of 1946. Preliminary work was undertaken until the middle of 1950. The design of the facilities was accomplished during the years from 1950 through 1955. Proposals for 35 construction contracts were received in the fall of 1955. The bids on 33 contracts were accepted immediately. One contract for intercepting sewer construction was readvertised, with an acceptable bid being received in January 1956. Another contract covering the 300-ft. chimney at the sewage treatment plant was readvertised twice before an acceptable bid was received in December 1957.

Currently, (September 1958) 14 of the 35 contracts have been completed. The project is scheduled for completion on March 1, 1959 and there is every indication that this completion date will be met.

The Project

The limits of the service area of the Allegheny County Sanitary Authority are shown on Fig. 1.

The Authority has contractual agreements with 70 municipalities within Allegheny County. Agreements with 18 industrial corporations have also been consummated. You will note that the city of Pittsburgh occupies a major portion of the Authority's service area.

The system of intercepting sewers is shown on Fig. 2. These sewers drain to the Pittsburgh Sewage Treatment Plant of the Authority located, as you will note, about two miles downstream on the

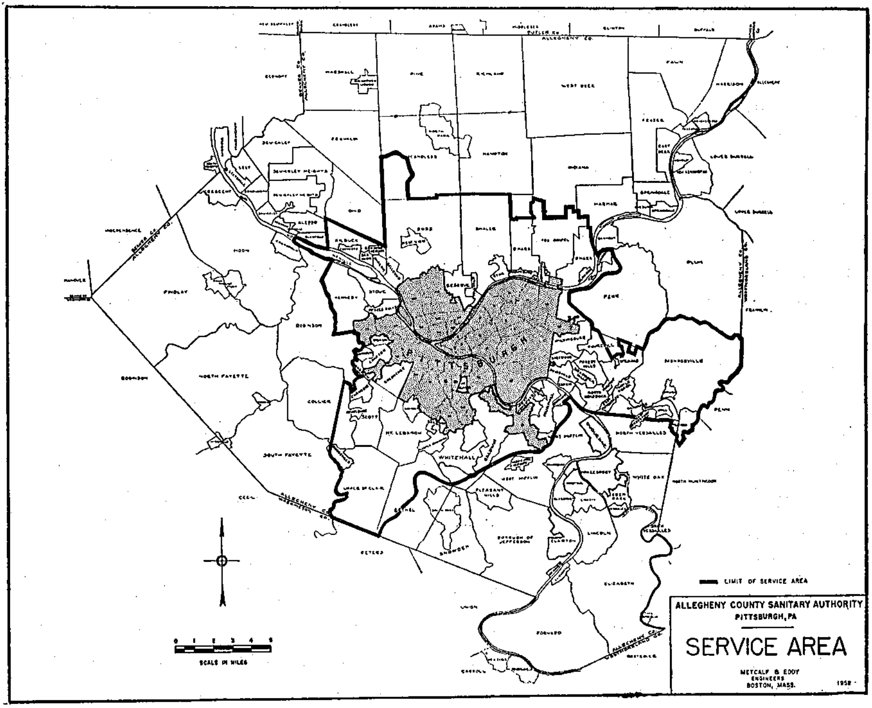

Fig. 1.

Ohio River from its origin at the confluence of the Allegheny and Monongahela Rivers.

Main intercepting sewers, constructed as tunnels, are located along the Ohio River, the Allegheny River, and the Monongahela River. Branch intersecting sewers, constructed by opencut methods, are located along Chartiers Creek and Turtle Creek.

One sewage pumping station and two pneumatic ejector stations lift the sewage from local sewers into the Authority intercepting sewers. These stations are designated Corliss, U5, and Melancthon Street, respectively.

Since the local sewerage systems are of the combined type, overflows have been provided at their connections to the Authority system. The discharge to the intercepting sewers is controlled by an automatic regulator consisting either of a gate or a fixed orifice. The automatic

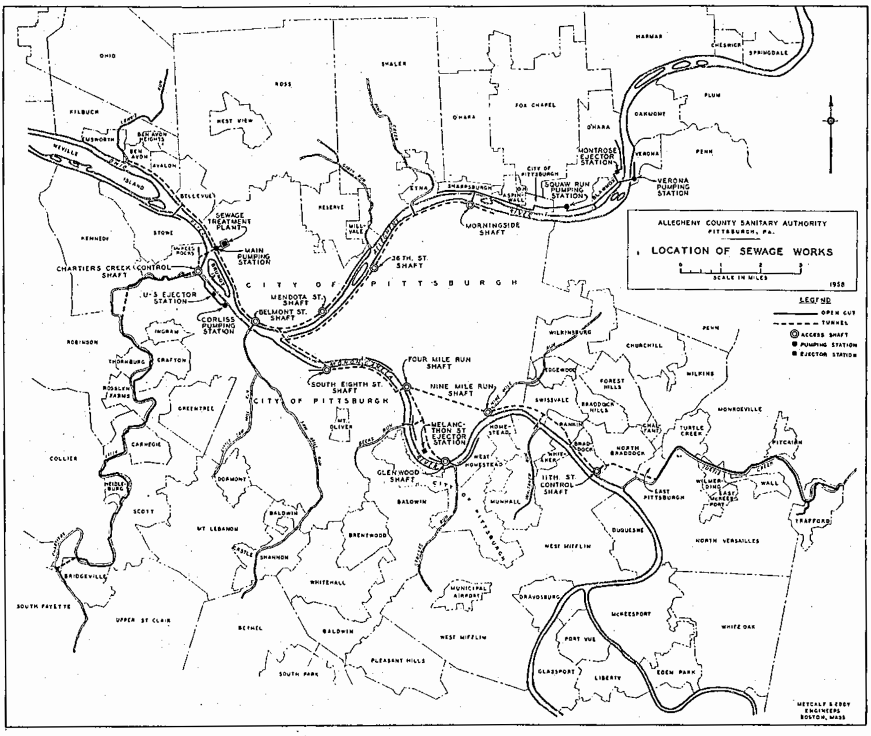

FIG. 2.

regulators of the gate-type are operated hydraulically, without floats, through water pressure on the gate itself.

Control points are located at the lower end on the Chartiers Creek intercepting sewer, where the construction changes from open-cut to tunnel methods, at Morningside, and at 11th Street on the Turtle Creek Intercepting Sewer. These structures are essentially major relief overflows, provided to permit regulation of the discharge from the local sewers to the main intercepting system. This control is necessary to prevent surcharging of the intercepting sewers and, thus, precluding downstream diversions. Control of these overflows will be effected by adjusting the water level in the wet well of the Main Pumping Station at the treatment plant.

Shafts to permit access to the tunnels for inspection and repairs are located at Belmont Street, Mendota Street, 36th Street, South 8th Street, Four Mile Run, Nine Mile Run, and Glenwood.

An extension of the intercepting sewer system above Aspinwall Dam on the Allegheny River has been called the Upper Allegheny System. This system consists of intercepting sewers, force mains, two pumping stations, and a pneumatic ejector station. Negotiations and planning are currently being made to extend the system to provide service for Oakmont Borough.

Operation of System

The sewage treatment plant is designed for an average flow of 150 mgd. in 1970, and a maximum flow of 300 mgd. in the year 2000. Regulating devices along the system have been designed to limit the discharge to the intercepting sewers at the first flush from a storm to 250 percent of the average dry weather flow. Certain of the overflows are equipped with tipping gates and these will automatically cut the diversion back to 200 percent of this flow after the first flush is passed. Normally the system will be operated at as high a hydraulic profile as possible, without causing discharge through any of the numerous overflows. Upon noting impending storm conditions, the water level in the wet well will be lowered to increase the capacity of the sewers. Periodically, the wet well will be pumped down to create sufficient velocity in the intercepting sewers to flush them out.

The normal pumping heads of the Main Pumping Station will vary from a minimum of 39 ft. to a maximum of 78 ft. By maintaining the minimum head for as much of the time as possible, significant savings in the cost of power are anticipated.

Two points in the system are particularly critical hydraulically. These are the 11th Street Control and the Morningside structure. The operators at the Main Sewage Pumping Station will maintain the system hydraulic profile just below that elevation at these two structures which will prevent overflows occurring at diversion structures on tributary sewers.

Sewage levels in the various access shafts will be transmitted by audi-tone-type of telemetering equipment to an instrument panelboard located in the Main Sewage Pumping Station. This panelboard was a feature exhibit at the Federation of Sewage and Industrial Wastes Associations Convention in Detroit in October 1958.

For those of you who wish additional information in regard to the details of the diversion structures, we suggest that you refer to Mr. Dore's paper.

Pittsburgh Sewage Treatment Plant

The principal features of the Pittsburgh Sewage Treatment Plant are shown in outline form on Fig. 3.

The incoming sewage will enter the Main Pumping Station through three connections, the Upper Ohio, Chartiers Creek, and the Lower Ohio Intercepting Sewers. Three two-speed pumps and two variable-speed pumps will lift the sewage in this station to such a

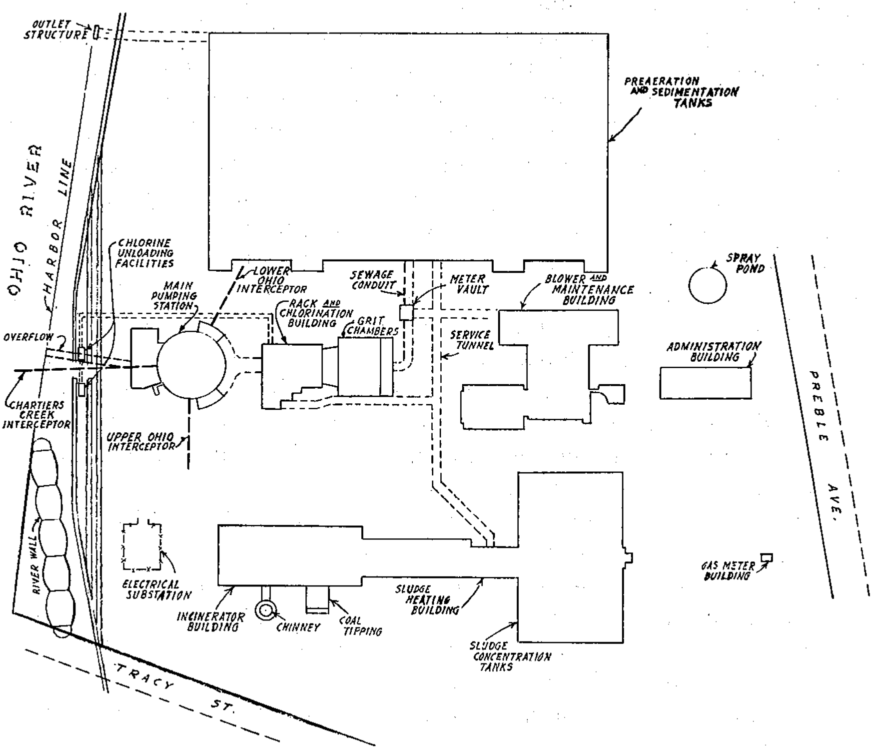

Fig. 3.

level that it will flow by gravity through the remainder of the treatment facilities.

From the pumping station, the sewage will first pass through a battery of four mechanically cleaned bar racks located in the Rack and Chlorination Building. Chlorine for pre-chlorination will be added in the main conduit ahead of these racks.

Chlorination facilities include unloading stations for four railway tank cars located to the west of the Main Pumping Station. Chlorine evaporators, chlorinators and automatic dosage control equipment are located in the Rack and Chlorination Building.

The screened sewage will next pass to four Aerated Grit Chambers. The aeration system is equipped with Walker Process type "Spargers". Grit will be removed from the chambers by clam bucket, operated from a monorail, and after draining in storage hoppers will be loaded into trucks for disposal at a dump purchased by the Authority approximately 1¼ miles distance from the plant site.

After separation of the grit, the sewage will be collected in a single main conduit and passed through the Main Sewage Meter for measurement. This meter, a magnetic flowmeter, has an internal diameter of 6 ft. and is supplied by the Foxboro Company. We are informed that this is the largest application to date of the magnetic type meter.

From the meter, the sewage will pass to the Preaeration and Sedimentation Tanks, laid out in two equal batteries. There are two preaeration tanks, each consisting of two channels and providing 44 minutes detention at the 150 mgd. design rate of flow. Chlorine for post-chlorination is applied in the influent channel of the sedimentation tanks. There are 6 sedimentation tanks, each consisting of 4 longitudinal channels. These tanks will provide 2 hours detention at the 150 mgd. rate of flow. The settled sewage will be collected in an effluent channel and, thence, discharged to the Ohio River through a submerged outlet at the river wall.

Sludge will be collected by drag-type longitudinal collectors and screw-type cross conveyors. Scum will be collected by the return pass of the longitudinal collector flights and discharged to a scum trough by revolving squeegee-type skimmers. This sludge from the tanks will be pumped through four Sludge Pumping Stations to the sludge disposal portion of the plant, which is represented by the Sludge Concentration Tanks, Sludge Heater Building and the Incinerator Building.

Sludge disposal will be by the novel Laboon process. Freshly settled sludge and scum will be pumped intermittently to equalizing tanks at the Sludge Concentration Tanks. The sludge will be pumped from these tanks at a constant rate, passed through Dorr-Oliver Disintegrators to break up all solids which might cause clogging of subsequent facilities, and then passed through sludge heaters.

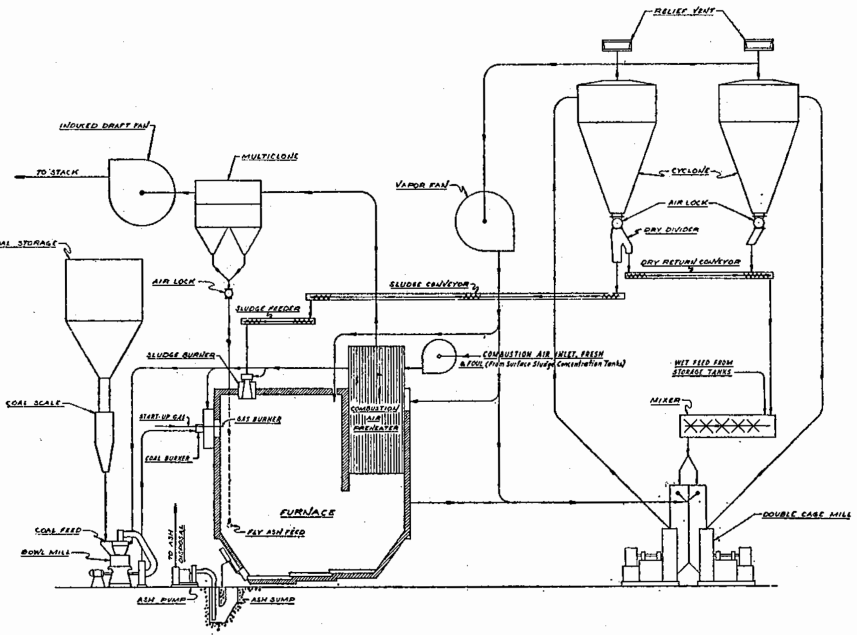

FIG. 4.

The Sludge Heater Building provides space for five initial sludge heaters and two future units with a rated capacity for each unit of 3 million Btu. per hour. These heaters are capable of raising the temperature of the sludge to 95 deg. F. After heating, the sludge will discharge to the Sludge Concentration Tanks.

There are 10 Sludge Concentration Tanks and they will be operated on a batch basis in rotation. The heated sludge when pumped to the tank is expected to average around 92 percent moisture. After the tank has filled, it will be allowed to stand quiescent for 5 days. Initial decomposition will generate gas which will be trapped in the suspended solids and cause the latter to rise to the surface. By the end of the detention period the subnatant liquid in the lower portion of the tank will be relatively clear and will then be drawn off and returned to the sewage flow at the head of the plant. The sludge solids remaining in the tank are expected to have a moisture content of only around 80 percent.

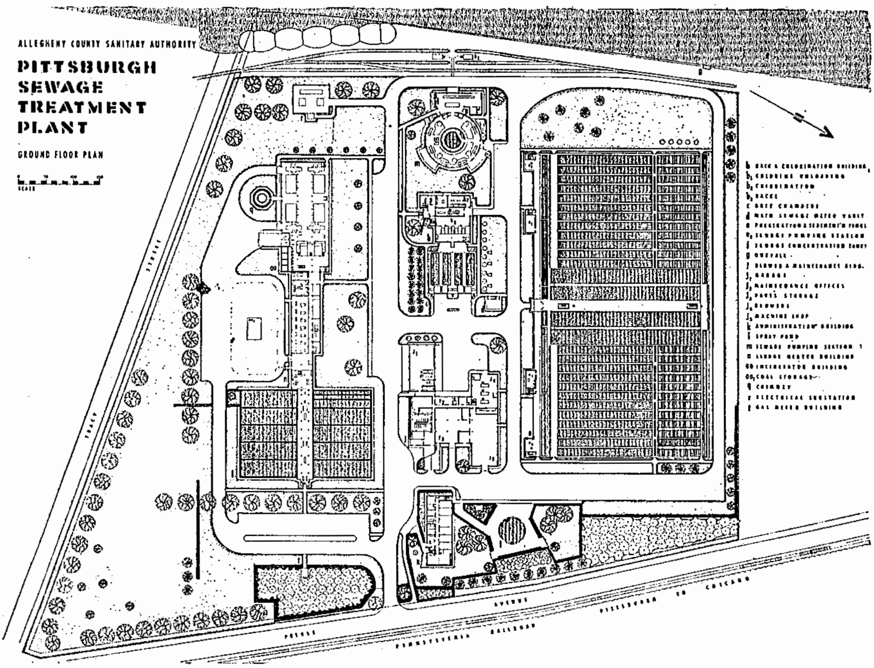

FIG. 5.

Each tank has four longitudinal hopper-type troughs, each equipped with a screw conveyor. The screw conveyors discharge the sludge to a screw-type cross-conveyor which discharges to a screw-type elevator which in turn discharges to the concentrated sludge pumps. The concentrated sludge will be withdrawn from the tanks and pumped by Moyno Pumps to the Incinerator Building.

The sludge incineration facilities consist of four Flash Drying Incinerators of the Raymond Division of Combustion Engineering. The gases are exhausted to the atmosphere through the chimney, which is 300 ft. high with an inside top diameter of 14 ft. Supplementary fuel consisting of slack coal may be used to ensure complete combustion in the incinerators. Gas burners will be used for preheating the combustion chambers. Two Combustion Engineering boilers are also housed in the Incinerator Building to provide steam for the sludge heaters and for the plant heating.

The flow diagram for the sludge incineration process is shown in Fig. 4.

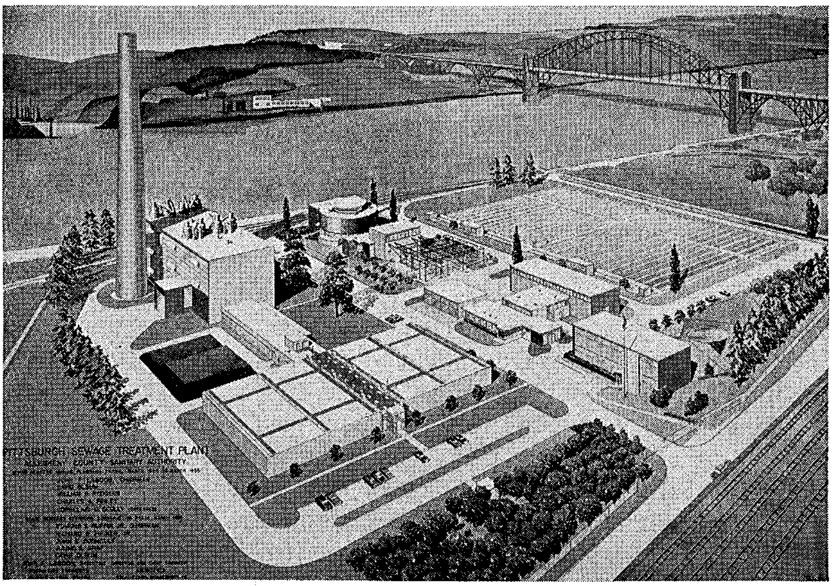

FIG. 6.

The electrical requirements of the plant will be supplied through a substation which is served by means of a 22,000-volt automatic loop type service of the Duquesne Light Company. Two 5,000 kva. oil-filled transformers, owned by the Authority, are provided in this station.

The plan of the treatment plant at ground floor level as depicted by architectural media is shown on Fig. 5.

An architectural rendering of the completed treatment plant is shown on Fig. 6. The highest type of building materials have been utilized to provide durable structures and a pleasing appearance. These materials include face brick for all buildings at the plant which are ceramic glazed in colors of yellow, red, blue, and gray.

Organization of the Authority

The Authority is governed by a board of five members, three of whom are appointed by the city of Pittsburgh, with two being appointed by Allegheny County. Current board members are as follows:

Edmund S. Ruffin, Jr.	Chairman
Julius E. Graf	Vice-Chairman
Richard B. Tucker, Jr.	Secretary
John E. Connelly	Treasurer
David Olbum	Asst. Secy.-Asst. Treasurer

The staff of the Authority is headed by Mr. John F. Laboon, who is Executive Director and Chief Engineer. During the preliminary and design stages of this project from March 1946 to March 1955, Mr. Laboon was not only Chief Engineer but was also Chairman of the Authority.

As noted previously, Mr. Stanley M. Dore, a member of this Society, was formerly Deputy Chief Engineer. This position is now held by Mr. Lawrence M. Gentlemen, also a member of this Society and well known to many members.

The office staff of the Authority is headed by the Deputy Chief Engineer. Since there are many continuing legal points arising, the Authority also has a permanent chief legal council.

The remainder of the office staff may informally be broken down into three groups: administrative, engineering, and estimating.

The field staff of the Authority is headed up by the Construction Engineer, Mr. Richard J. Dougherty. Under him are three divisions for handling the various stages of construction work under contract to the Authority. These are the treatment plant, the tunnels, and the open cut intercepting sewers. The field staff of the Authority also includes a material approval section, which passes on the acceptability of all materials and equipment as meeting the standards established by the contract plans and specifications.

The Authority is served by a number of consultants. The following are the engineering consultants to the Authority for the various phases of the work:

Metcalf & Eddy have been the consulting engineers to the Authority since 1947, carrying through with the preliminary reports, the design of the Pittsburgh Sewage Treatment Plant, and the design of the intercepting sewers along Chartiers Creek and Turtle Creek and those portions of the Ohio River intercepting sewer in open cut. Celli-Flynn, consulting architects of McKeesport, Pa., were retained by Metcalf & Eddy for reviewing the architectural planning at the plant in regard to materials, colors, and exterior and interior appearances. Betterley Associates, insurance consultants of Worcester, Mass. have

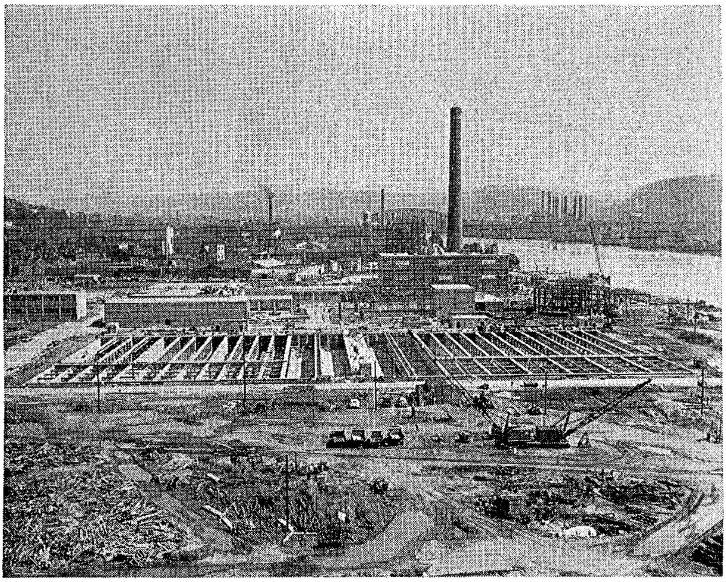

FIG. 7.—GENERAL VIEW OF PITTSBURGH SEWAGE TREATMENT PLANT WITH THE PRE-AERATION AND SEDIMENTATION TANKS IN THE CENTER, THE STRUCTURAL STEEL OF THE CIRCULAR MAIN SEWAGE PUMPING STATION ON THE RIGHT, AND THE INCINERATOR BUILDING AND CHIMNEY IN THE BACKGROUND.

advised us regarding the insurance program of the Authority during the operation of the system.

Philip S. Miller and Associates of West Orange, N.J. have advised the Authority on problems dealing with tunnel construction. On soil problems, the Authority has consulted E. D'Appolonia Associates of Pittsburgh. Thompson & Lichtner Co., Inc. of Boston have been consulted regarding concrete problems, particularly in the manufacture of concrete pipe. Problems of an electrical nature beyond the scope of the Authority's resident staff have been reviewed with Mr. E. T. Wiesmann of Pittsburgh.

Three consultants, all of Pittsburgh, have advised the Authority in financial and other related matters during the course of the construction program. Burgwin, Ruffin, Perry, and Pohl have acted as bond council to the Authority. Singer, Dean and Scribner have been financial advisors to the Authority in determining when the market was favorable for the issuance of bonds. Ebbert, Grant, and Kakel

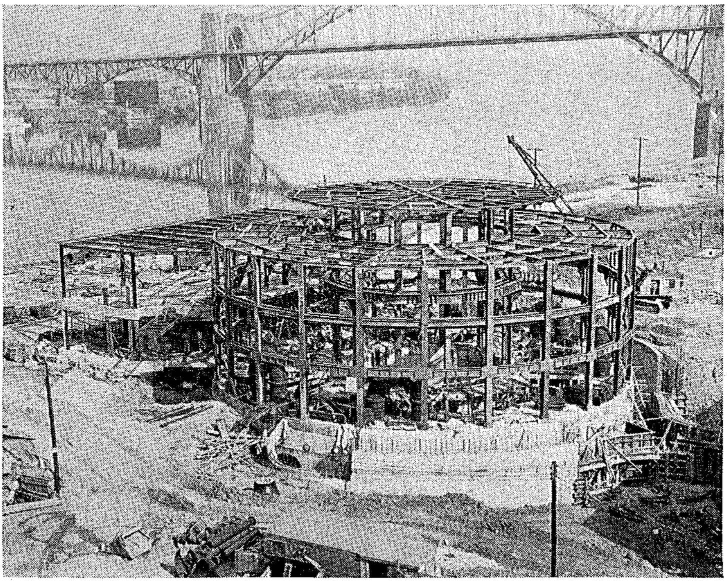

FIG. 8.—STRUCTURAL STEEL FOR MAIN SEWAGE PUMPING STATION IN PLACE, CONTROL BUILDING PORTION TO THE LEFT WITH CIRCULAR WET AND DRY WELLS ON THE RIGHT, CONCRETE WORK IN THE LOWER RIGHT FOR DISCHARGE CONDUIT TO REMAINDER OF PLANT. MCKEES ROCKS BRIDGE ACROSS OHIO RIVER IN BACKGROUND.

have advised the Authority on the acceptability of the insurance coverages required of the various contractors by the Authority as a part of their construction contracts. This firm has also acted as insurance consultants to the Authority.

FINANCIAL

The financial negotiations and arrangements for this project are interesting.

The bond market in the fall of 1955 was not overly favorable for the issuance of bonds for the Authority to start its construction program. The Authority was fortunate, therefore, in being able to arrange through the Mellon National Bank and Trust Co., of Pittsburgh for a bank loan of $100,000,000, which was consummated on October 4, 1955 for a four-year period. Twenty-four other banking institutions throughout the United States participated with the Mellon Bank in providing this bank loan.

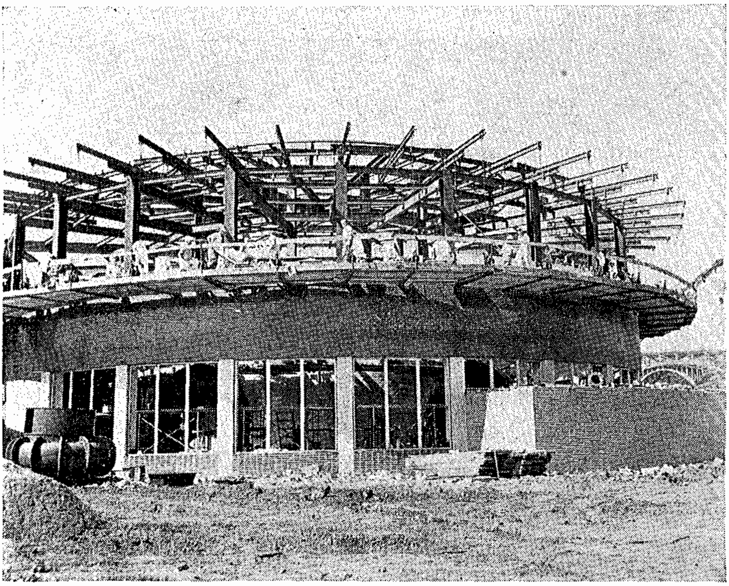

FIG. 9.—LAYING EXTERIOR BRICKWORK FOR MAIN SEWAGE PUMPING STATION. IN THE LEFT OF THE PHOTOGRAPH A 36-IN. MAIN SEWAGE PUMP, SUPPLIED BY WORTHINGTON CORP., RESTS AWAITING LOWERING INTO THE DRY WELL OF THE STATION, 110 FEET BELOW THE FINISH GRADE.

The bank loan bears two rates of interest, depending upon whether or not the money is secured by United States government bonds on deposit as collateral, or is not so secured. If it is secured by government bonds the rate of interest is 2¼ percent. For those portions not so secured, the rate increases to 2¾ percent.

Four issues of sewer revenue bonds have been made to date by the Authority. Certain details pertaining to the issuance of these revenue bonds are shown in Table 1.

The latest estimate of capital costs for this construction program of the Authority are shown in Table 2.

TABLE 1
Sewer Revenue Bond Issues

Series	Date of issue	Amount in millions	Net interest costs, %	Bond discount, %
A	May 1, 1956	$25	3.35	1.73
B	February 14, 1957	15	3.312507	1.97
C	February 4, 1958	25	3.5512	1.98
D	November 13, 1958	20	3.732725	1.99

TABLE 2
Capital Costs

I — Construction and Equipment Costs		
A. Basic intercepting sewer system	$61,607,000	
B. Upper Allegheny system	807,000	
C. Sewage treatment plant	17,500,000	
D. Maintenance and operating equipment	150,000	
E. Automotive equipment	203,000	
F. Other contracts	450,000	
		$ 80,717,000
II — Real Estate and Lands		1,773,000
III — Administrative Costs		
A. Administration and Engineering		
1. Engineering		
a. Resident supervision	2,771,000	
b. General supervision	296,000	
c. Trust indenture	109,000	
d. Other	112,000	
2. Administrative	881,000	
B. Fund to cover 1st year's operation	1,000,000	
		5,169,000
IV — Financial Costs		
A. Repayment of loans, incl. interest	3,007,000	
B. Net interest costs	3,745,000	
C. Bond discount	1,924,000	
D. Expenses in connection with Bond Issues	235,000	
		8,911,000
V — Contingencies and 6 months' interest		3,430,000
VI — Total Costs		$100,000,000

Authority Staff for Operating the System

The office staff of the Authority will consist of three major sections which could be described by their functions as being administrative, engineering, and billing.

The basis for billing the customers, in general, will be the water meter readings. These readings will be furnished to the Authority by

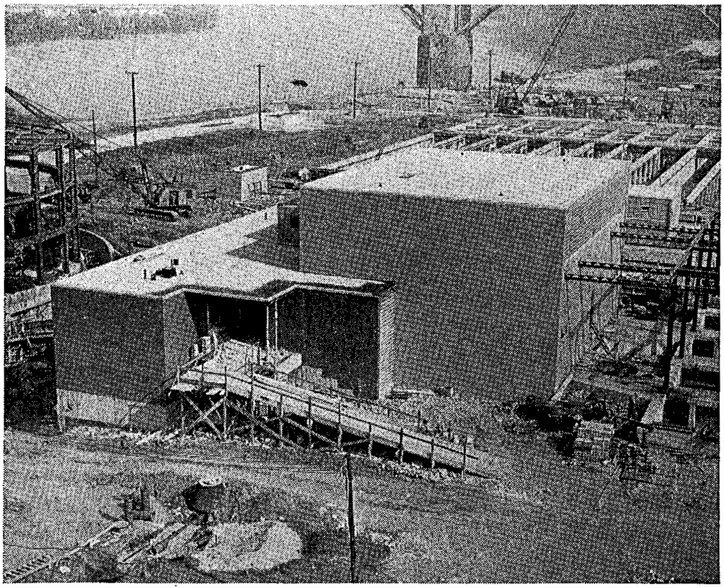

Fig. 10.—View of the south elevation of the Rack and Chlorination Building, with the chlorination facilities located under the low roof and with the racks provided with the high roof. A portion of the Aerated Grit Chambers appears in the right of the photograph.

the various municipal and private water utilities providing public water supply in the service area. The Authority is planning on utilizing International Business Machine type of billing equipment to process the meter reading data and bill the individual customers. The average sewer service charge for domestic customers, based on the water usage of 50,000 gallons per year, has been estimated to be $15 per year.

In the event the customers fail to pay their bills to the Authority, the municipalities in which the customers reside will be required by their agreements with the Authority to make a payment to the Authority to cover such unpaid bills. In this fashion, the normal municipal processes of placing liens on property may be utilized for the benefit of the Authority.

It is estimated that from 125 to 150 personnel will be required to operate the Pittsburgh Sewage Treatment Plant. The maintenance

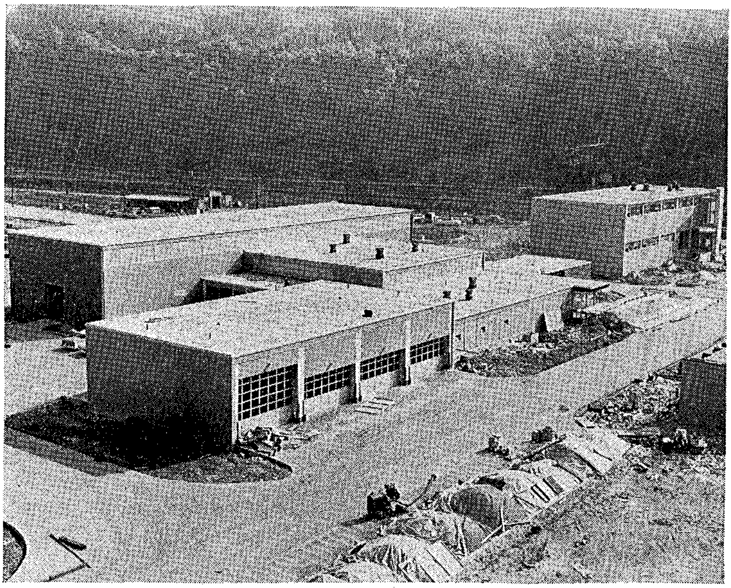

FIG. 11.—THE ADMINISTRATION BUILDING APPEARS IN THE RIGHT OF THE PICTURE. THE BLOWER AND MAINTENANCE BUILDING IS TO THE LEFT. THE LONG PORTION IN THE REAR HOUSES THE BLOWER EQUIPMENT ON THE LEFT WITH A MACHINE SHOP ON THE RIGHT. THE PLANT GARAGE IS IN THE FOREGROUND.

of the intercepting sewers will be handled by approximately 15 men. This group will be provided with a tow boat and barge for accomplishing much of this maintenance work, since many of the locations, along the rivers on the intercepting sewer system, requiring maintenance are difficult to get to, other than by water.

CONCLUSION

This project has not only been of an unusually broad scope, but also a number of unique problems of special interest have been included, several of which have been discussed. Since its conception, the Authority has been fortunate to have had the continuous service for a period of twelve years, of Mr. John F. Laboon. He has been not only a forceful administrator, but also an engineer through whose efforts the novel Laboon process of sludge disposal has been developed for use in this major sewage treatment plant. The operating results of this new plant are awaited with great interest.

A reference list of published technical papers pertaining to this project, follows:

ILLUSTRATIONS

Progress photographs of the sewage treatment plant are shown in Fig. 7-11 and represent the status of construction in the summer of 1958. These photographs illustrate type of structures provided at this plant.

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HISTORY AND BASIC THEORY OF PRESTRESSED CONCRETE

By James J. Kenney, *Member*

Let us first consider the history of prestressed concrete and its present day application in the long lines method of prestressing concrete as we know it today. The first patent was taken out by P. H. Jackson, of San Francisco, in 1886, and in 1920 and 1925 R. H. Dill, of Alexandria, Nebraska, took out patents on the method of pretensioning. The development of prestressed concrete was slow because the initial approach was made using low carbon rods having a relatively short elongation within the proportional limit of the steel used. This meant that the effect of the prestressing force was partially nullified by the effect of creep and shrinkage of the concrete.

Later in Europe, Freyssinet and others approached the problem with the use of high carbon cold drawn wire having a relatively long elongation within the proportional limits of the steel. When allowance was made for creep and shrinkage of the concrete a high percentage of the initial stress in the wire was available for inducing compressive stresses in the concrete.

In Europe such names as Freyssinet, Magnel, Billig, Hewitt and Hoyer were common with the development of prestressed concrete, while in the United States such names as Preload, Roebling, Schorer and Billner have been associated with its development. Today its use has been demonstrated on a multiplicity of structures and now it remains for American engineers and contractors to exploit its possibilities here.

Prestressed concrete is simple in its application and design and I think that we may pause for a while and consider the basic theory of prestressed concrete.

If we consider a short compression member with a force "P" applied eccentrically to one of its axis, resolve the force into a concentric force "P" and a couple with a moment equal to "Pe", draw the stress diagram for the individual force and the couple, then draw the final stress diagram for the combined effect making the assumption that the distance of "e" is taken as a special case and is equal to b/6, then the total stress at either side will be

$$S = S_1 \pm S_2$$

$$S = \frac{P}{A} \pm \frac{Pec}{I}$$

$$S = \frac{P}{A} \pm \frac{Pe\, b^2}{6}$$

Irrespective of the magnitude of "P" if placed at outer edges of middle third, stress at end will always be "o".

Draw a side view of a part of a prestressed rectangular beam and again resolve the prestress force located at the third point into a concentric force "P" and a couple with a moment equal to "Pe". Show that the fibre stress at the top or bottom of the beam is computed from a similar equation as that used for the short compression member except that the effect of the external moment "M" is included.

We now have the general equation

$$fc = \frac{P}{A} \mp \frac{Pe\text{-}M}{I}\, y(tb) \qquad \text{(Equation 1)}$$

In order to clarify the sequence of the derivations from the basic formula Figure 1 can be used:

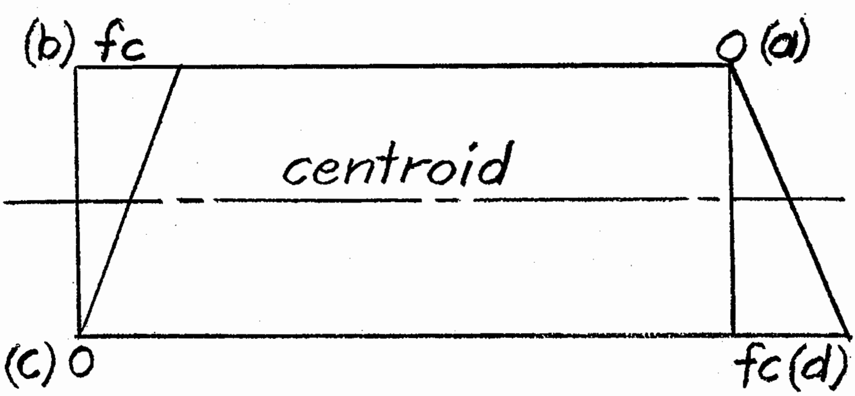

Figure 1.

Determination of Eccentricity condition (a) that no tension exists in the top fibres at the ends of the girder under critical loading conditions.

General Equation: $fc = \frac{P}{A} \mp \frac{Pe\text{-}M}{I} y\ tb$

Revised Equation for the set of loading conditions:

$$fc = \frac{P}{A} \mp \frac{Pe\text{-}M}{I} = 0\ yt\ b$$

$$0 = \frac{P}{A} - \frac{Pe}{I}\ yt$$

$$\frac{P}{A} = \frac{Pe\,(yt)}{I}$$

$$\frac{P}{A} = \frac{Pe\ yt}{I = Ar^2}$$

$$P = \frac{Pe\ yt}{r^2}$$

$$\frac{Pr^2}{yt} = Pe$$

$$e = \frac{Pr^2}{P\ yt}$$

$$e = \frac{r^2}{yt} \qquad \text{(Equation 2)}$$

"e" has been fixed so that top fibre will be = 0 from prestress alone.

Condition at (b) that the fibre stress "fc" be within that allowed by the specifications for the condition maximum "MT". Since in fixing the eccentricity "e" so that the stress in top fibre from prestress alone will be zero, the effect of prestress can be disregarded in computing "fc" at "b".

General Equation:

$$fc = \frac{P}{A} \mp \frac{Pe\text{-}M}{I}\ ytb$$

$$fc = \frac{MT}{I}\ yt \qquad \text{(Equation 3a)}$$

In the substitution of an actual problem the magnitude of "fc" will be a check on the design assumption for the size of the member selected and such changes as may be necessary should be made before proceeding farther.

Since in prestressed concrete design it is possible to add and subtract opposite stresses or superimpose stress diagrams, the tension in the bottom fibres for the MT loading will be computed for later use in making stress diagrams.

$$fc = \frac{MT}{I} yb \qquad \text{(Equation 3b)}$$

Condition at (c) for any combination of design loading, no tension is permitted in the bottom fibres which would be critical near the middle of the span where the moment would be maximum.

General Equation:

$$fc = \frac{P}{A} \pm \frac{Pe\text{-}M}{I} y\ t_b$$

$$0 = \frac{P}{A} + \frac{Pe\text{-}MT}{I = Ar^2} yb$$

$$\frac{P}{A} = \frac{Pe\text{-}MT}{Ar^2} yb$$

$$\frac{Pr^2}{yb} - Pe = MT$$

$$P\left[\frac{r^2}{yb} - e\right] = MT$$

$$P\ (\text{Final}) = \frac{MT}{\dfrac{r^2}{yb} + e} \qquad \text{(Equation 4)}$$

$$P\ (\text{Initial}) = \frac{\dfrac{MT}{0.8}}{\dfrac{r^2}{yb} + e} \qquad \text{(Equation 5)}$$

Condition at (d) under normal condition of loading the stress in the bottom fibre must be within that allowed by the specifications near the end of the member.

General Equation:

$$fc = \frac{P}{A} \mp \frac{Pe\text{-}M}{I} \, y\,t\,b$$

$$fc = \frac{P}{A} + \frac{Pe}{I} \, yb$$

$$fc = \frac{P}{A} + \frac{Pe}{Ar^2} \, yb$$

$$fc = \frac{P}{A}\left[1 + \frac{eyb}{r^2}\right] \qquad \text{(Equation 6)}$$

In order to make diagrams for various combinations of loading conditions the stresses for the effect of the dead load of the girder will be computed for use later on in drawing stress diagrams.

$$fc = \frac{MG}{I} \begin{matrix} yt \\ yb \end{matrix} \left\{ \begin{matrix} \text{compression} \\ \text{tension} \end{matrix} \right. \qquad \text{(Equation 7)}$$

With the various equations the substitution of numerical values can be made and the stress diagrams can be drawn.

The long line method of precast pretensioning has been enlarged on and is today the method used in most of our plants throughout the country.

The first application of prestressed concrete that I recall was first used in bridge construction by the construction battalions of Julius Caesar. They made large barrels using curved staves around which bronze hoops were forced from opposite ends to provide circumferential compression. The barrels were used as pontoons for floating bridges for the transport of Caesar's army and military equipment in the conquest of Britain.

In April of 1886 P. H. Jackson, of San Francisco, obtained a patent to prestress artificial stone or concrete arches which could be used as floors for buildings or making sidewalks over excavations. This work was followed by several others in Europe, and in 1908 C. K. Steiner, an American, proposed to tighten reinforcing rods against green concrete.

Early attempts to develop a practical method of prestressing failed because of the lack of high tensile steel or because the developers lacked the knowledge of the shrinkage and plastic flow characteristics of concrete. Most of the prestressing force applied was nullified because the mild steel stretched about the same amount that the concrete decreased in length.

In 1925 R. H. Dill, of Alexandria, was the first to succeed in producing prestressed concrete members by the post-tensioning method. In Dill's method high tensile strength or hard steel was coated with a plastic substance to prevent bond with the concrete. After the concrete had hardened he induced stress in the steel by tightening nuts at the ends of the members.

In 1928 French engineer, M. Freyssinet, worked on the scheme and in 1939 he introduced a practical method for post-tensioning by means of double acting jacks and anchoring the cables at the ends by means of conical wedges. At the same time, Hoyer, of Germany, had developed a practical means of pretensioning by casting the concrete around wires that were already tensioned. After the concrete had hardened small units were made by cutting up the long continuous pieces.

At this point we ask ourselves how does prestressed concrete differ from ordinary reinforced concrete? The basic theory behind both schemes is to devise a method that will compensate for the low tensile strength of concrete and prevent cracking when such stress is produced.

Ordinary reinforcement helps this condition but it remains essentially inert until load is applied to the member. As the load is increased, there is an increase in the steel stress which is accompanied with an increase in the length of the bar. This increase could be greater than the concrete can withstand. Therefore, a hair crack develops in the tension area of the concrete. At the same time only about one-third of the concrete section is effective in resisting compressive stresses.

Prestressed concrete imposes preliminary internal stresses in the member before the working loads are applied in such a way to lead to a more favorable state of stress when the external loads are applied. In other words, eliminate tensile stresses by superimposing compressive stress by mechanical means. This now leaves the whole cross-section available for compression which may be demonstrated this way.

Reinforced Concrete	*Prestressed Concrete*
$Mc = \frac{1}{2}\, fc \times \frac{3}{8}\, d\,1 \times \frac{7}{8}\, d\,1$	$M^1c = \frac{1}{2}\, fc \times d^2$
$= \frac{21}{128}\, d\,1^2\, fc$ say $\frac{1}{6}\, d\,1^2\, fc$	$M^1c = \frac{1}{2}\, fc \times d^2 \times \frac{2}{3}\, d\,2$
$Mc = \frac{1}{6}\, d\,1^2\, fc$	$M^1c = \frac{1}{3}\, fc \times d\,2^2$

For same size beam

M R/C 2500# concrete $M_c = M_c$

M (P.S.C.) 5000# concrete $M^1c = 4\ M_c$

So you see that prestressed concrete is a more efficient use of the cross-section and the neutral axis is at the bottom of the beam.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

OCTOBER 21, 1958.—A Joint Meeting of the Boston Society of Civil Engineers with the Massachusetts Section of the American Society of Civil Engineers was held this evening at Tufts University, Medford, Mass. The Student Chapters of the New England Colleges were especially urged to attend.

A dinner was held in the Jackson Gymnasium, Tufts University from 6:30 to 7:45 P.M. Student delegations were present from Tufts University, Northeastern University, Harvard, Massachusetts Institute of Technology, Brown University, Worcester Polytechnic Institute, University of Rhode Island, University of Connecticut and University of Massachusetts.

The meeting was held in the Cohen Auditorium and was called to order at 8:00 P.M., by President William L. Hyland.

President Hyland extended a cordial welcome to the students and expressed appreciation of the cooperation of the officers of the student organizations and the faculty members in making this event so successful.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership October 20, 1958:—

Grade of Member—George C. Conway, Robert G. Dean, Richard D. Field, John C. Hawley, Lawrence T. Kraynick, Harold W. Olsen, John A. Volpe, Peter B. Heidema, Allyn S. Richardson, Samuel I. Widershien, Willard O. Wilcox.

Grade of Junior—Richard A. Bernard, Alphonso J. Binda, John F. Brady, Jr., Edward J. Comeau, Elias A. Cooney, Jude T. Flynn, Mrs. Leslie B. Fuller, Donald J. Hession, Frank S. Lagrotteria, Bruce N. MacIver, Peter A. Martus, Anthony L. Ricci, Walter H. Snyder.

President Hyland introduced John J. Cusack, Secretary of Massachusetts Section, ASCE, and asked him to conduct any necessary business of ASCE at this time.

President Hyland then introduced the speaker of the evening, Dean Gordon M. Fair, Gordon McKay Professor of Sanitary Engineering, Harvard University who gave a most interesting talk on "The Civil Engineer and His Responsibility to Society."

Two hundred fifty-six members and guests attended the dinner and two hundred forty-one attended the meeting.

The meeting adjourned at 9:06 P.M.

ROBERT W. MOIR, *Secretary*

NOVEMBER 19, 1958.—A Joint Meeting of the Boston Society of Civil Engineers with the Hydraulics Section, BSCE was held this evening at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by President William L. Hyland, at 7:25 P.M.

President Hyland announced that the minutes of the previous meeting held October 21, 1958 would be published in the JOURNAL and that the reading of those minutes would be waived unless there was objection.

President Hyland announced the death of the following members:

T. Parker Clarke, who was elected a member January 26, 1921, and who died October 9, 1958.

Edwin R. Olin, who was elected a member March 16, 1904, and who died September 16, 1958.

John P. Gallagher, who was elected a member January 27, 1915 and who died in July, 1958.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership November 17, 1958:—

Grade of Member—Walter G. Baker, Raymond Donahue

President Hyland stated that this was a Joint Meeting with the Hydraulics Section and called upon James W. Daily, chairman of that section to conduct any necessary business at this time.

President Hyland introduced the speaker of the evening, John W. Leslie, Chief, Engineering Division, U. S. Army Engr., Div., New England, who gave a most interesting talk on "Development and Design of Hopkinton-Everett Flood Control Project." The talk was illustrated.

The meeting was preceded by a dinner and 69 members and guests attended the dinner. Ninety-three members and guests attended the meeting.

The meeting adjourned at 9:10 P.M.

ROBERT W. MOIR, *Secretary*

DECEMBER 17, 1958.—A Joint Meeting of the Boston Society of Civil Engineers with the Structural Section, BSCE was held this evening at the United Community Services Building, 13 Somerset Street, Boston, Mass., and was called to order by President William L. Hyland, at 7:00 P.M.

President Hyland announced that the minutes of the previous meeting held November 19, 1958 would be published in a forthcoming issue of the JOURNAL and that the reading of those minutes would be waived unless there was objection.

President Hyland announced the death of the following members:

Luis G. Morphy, who was elected a member September 16, 1908 and who died October 28, 1958.

David J. White, who was elected a member January 26, 1921, and who died December 9, 1958.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership December 15, 1958:—

Grade of Member—Zane A. Bower, Imants M. Kaupe, Joseph DiGiovonni, Robert F. McNeill, Joseph J. Randall, Donald K. Sampson, Richard J. Schoenfeld, Jr., Ken N. Shroff, Clement V. Smith, George E. Trainor, Arne J. Wolti, George C. Woods.

Grade of Student—John A. Cronin.

President Hyland stated that this was a Joint Meeting with the Structural Section and called upon Richard W. Albrecht, chairman of that section to conduct any necessary business at this time.

President Hyland introduced the speaker of the evening, Miles N. Clair who gave a talk on "Draft of Proposed Amended Part 26—Reinforced Concrete of the Boston Building Code."

A discussion period followed the talk.

The meeting was preceded by a dinner and 56 members and guests attended the dinner. Seventy-nine members and guests attended the meeting.

The meeting adjourned at 9:22 P.M.

ROBERT W. MOIR, *Secretary*

STRUCTURAL SECTION

NOVEMBER 12, 1958.—A meeting of the Structural Section was held in the Society rooms. Chairman Albrecht called the meeting to order at 7:00 P.M. The minutes of the previous meeting of October 8 were read. The Chairman introduced Dr. Ruth Terzaghi who spoke to the Section on "Geological History of New England."

Dr. Terzaghi discussed the geologic calendar showing that the time scale is based on the formation of animal or vegetable life found in fossils. The oldest measurement of age recently determined for rocks is 500 million years. The relatively recent development of radioactive data on carbon-14 gives us fairly accurate data on the age of rocks. The accuracy in determining the age of rocks is in the order of one million years since the animal or plant life of a given type usually existed for that length of time with some exceptions. Dr. Terzaghi explained how in the beginning the Appalachian region was a trough which was several thousand feet deep into which deep sedimentary deposits containing marine plant and animal life collected. Later these deposits in the form of sedimentary rocks were pushed up by high pressure which folded the earth's crust to form the White and Green Mountains of New Hampshire and Vermont in the Paleozoic period and that later volcanic eruptions took place. Later the mountains formed by folding were eroded away to reduce the magnitude of their elevation. Dr. Terzaghi explained the way in which the ice cap scoured the valleys and left deposits of various sorts which we now find in New England. The fact that New England is generally rising as a result of the release of 20,000 feet of ice after the end of the last ice age has resulted in certain earthquake activity due to the readjustment. It was pointed out that the melting of the ice cap on the North American continent caused the ocean to rise approximately 300 feet.

After a brief discussion with several of the members, the meeting was adjourned at 9:00 P.M.

The attendance was 101.

HYDRAULICS SECTION

MAY 7, 1958.—A meeting of the Hydraulics Section was held at the M.I.T. Hydrodynamics Laboratory and was called to order by James W. Daily, Chairman at 7:05 P.M.

The minutes of the meeting of February 5th were read and approved.

The Chairman introduced the speaker, Peter S. Eagleson, Assistant Professor of Hydraulics, Massachusetts Institute of Technology, whose subject was "Hydraulic Model Study of Protective Works for Fleet Berths in Narragansett Bay."

Professor Eagleson's talk covered recent hydraulic model tests, at the M.I.T. Hydrodynamics Laboratory, of breakwaters under design for the Navy by Fay, Spofford & Thorndike. He described investigations of the breakwaters which are to protect thin-skinned vessels against north-west storm damage, with the model testing covering three main subjects:

(1) Optimum size and location of breakwaters.
(2) Effect of breakwater on flushing action by surface currents.
(3) Littoral sand transport effect.

The discussions included hindcasting wave heights based on observed prototype wind velocities at a nearby weather station and consideration of the significant wave concept as compared with the wave spectrum. Excellent slides were used to give the results of the tests. Reductions in wave heights from 9 feet to 2 feet were shown on maps of the water surface in the vicinity of the breakwaters.

After a good question and discussion period the group inspected the breakwater model, along with a wide variety of other hydraulic models in operation at the Hydrodynamics Laboratory. The inspection closed an interesting evening.

Total attendance was 47.

The meeting adjourned at about 8:30 P.M.

JOHN B. MCALEER, *Clerk*

ADDITIONS

Members

Walter G. Baker, Jr., 20 Kemper St., Wollaston 70, Mass.
Zane A. Bower, 14 Watt Street, Chelsea 50, Mass.
George G. Conway, 484 Beacon St., Boston 15, Mass.
Robert G. Dean, 12 Unity Avenue, Belmont 78, Mass.
Raymond Donahue, 58 Harding Avenue, Weymouth 88, Mass.
Joseph Giovanni, 13 Bradford Avenue, Medford 55, Mass.
Lawrence Kraynick, 77 Norwich Circle, W. Medford 55, Mass.
Robert F. McNeill, 5 Edwin St., Randolph, Mass.
Joseph J. Randall, 12 Alpine Road, Norwood, Mass.
Allyn S. Richardson, 519 Pleasant St., Belmont 78, Mass.
Donald K. Sampson, 12 Priscilla Road, Reading, Mass.
Richard J. Schoenfield, Jr., 146 Intervale St., Chestnut Hill 67, Mass.
Clement Smith, 542 Canton St., Stoughton, Mass.
Eugene Smith, 303 Union St., Norwood, Mass.
George E. Trainor, 14 Warren Place, Weston, Mass.
Ken N. Shroff, 90 Fresh Pond Pky., Cambridge 38, Mass.
George C. Woods, Commonwealth Corporation, 575 Boylston St., Boston 16, Mass.
Arne J. Wolti, 115 Green St., Lynn, Mass.

Juniors

John F. Brady, 228 Parke St., Lowell, Mass.
Edward J. Comeau, 307 Beech St., Belmont 78, Mass.
Elias Cooney, 1094 Andover St., No. Tewksbury, Mass.
Jude T. Flynn, 156 Dalton Road, Chelmsford, Mass.
Anthony L. Ricci, 291 Pearl St., Cambridge, Mass.
Walter H. Snyder, 245 Pearl St., Malden 48, Mass.

Student Member

John A. Cronin, 100 Packard Ave., W. Somerville 44, Mass.

Deaths

Luis G. Morphy, Oct. 28 1958
Arthur T. Weston, Dec. 19, 1958
David J. White, Dec. 9, 1958

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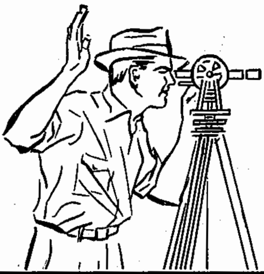

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