JOURNAL *of the*
BOSTON SOCIETY
OF
CIVIL ENGINEERS

109 YEARS
1848-1957

JANUARY - 1957

VOLUME 44 NUMBER 1

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1956-1957

PRESIDENT

JOHN G. W. THOMAS

VICE PRESIDENTS

ARTHUR CASAGRANDE (Term expires March, 1957)

WILLIAM L. HYLAND (Term expires March, 1958)

SECRETARY

ROBERT W. MOIR

TREASURER

CHARLES O. BAIRD, JR.

DIRECTORS

THOMAS A. BERRIGAN
ERNEST L. SPENCER
(Term expires March, 1957)

GEORGE G. BOGREN
EDWARD C. KEANE
(Term expires March, 1958)

PAST PRESIDENTS

CHESTER J. GINDER
MILES N. CLAIR
EDWIN B. COBB

SANITARY SECTION

DARRELL A. ROOT, Chairman — CLAIR N. SAWYER, Clerk

STRUCTURAL SECTION

AUGUSTINE L. DELANEY, Chairman — RICHARD W. ALBRECHT, Clerk

TRANSPORTATION SECTION

JOHN CLARKESON, Chairman — LEO F. DEMARSH, Clerk

HYDRAULICS SECTION

JOSEPH C. LAWLER, Chairman — JAMES W. DAILY, Clerk

SURVEYING AND MAPPING SECTION

LLEWELLYN T. SCHOFIELD, Chairman — GEORGE A. McKENNA, Clerk

Editor—CHARLES E. KNOX

88 Tremont Street, Room 715, Boston 8, Massachusetts

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 44 JANUARY, 1957 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
715 Tremont Temple, Boston, Massachusetts

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

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 44 JANUARY, 1957 Number 1

COMPETITIVE WATER USES: SANITARY ENGINEERING ASPECTS

BY RICHARD HAZEN*

(Presented at a meeting of the Sanitary Section, B.S.C.E., held on March 7, 1956.)

THE words "water competition" usually suggest the classic struggles for water in the arid and semi-arid states of the west. The water supplies there are simply inadequate to go around, and water is accordingly valuable. The fights for water have been bitter. The situation in the west has been aggravated by the fact that many of the uses are consumptive uses, and much of the water does not return to the stream or ground.

In the eastern states the water supply is more abundant and consumptive uses have been relatively small. The conflict between water users has been a matter of quality. However, in spite of this, the east has had its own disputes over diversion, as evidenced by the Pawtuxet River diversion to serve Providence, the Ware and Swift River diversions for metropolitan Boston, and the Delaware River diversion for New York City.

Probably the most important consumptive water use in the country is irrigation, and in this respect the east is approaching more rapidly the western conditions than is generally realized. All along the Atlantic seaboard, from South Carolina to New England, supplemental irrigation is growing by leaps and bounds. Well water has been used extensively on Long Island and New Jersey for many years. Surface water is impounded in literally thousands of farm ponds in Virginia and the Carolinas. The growth of these farm ponds has already had an appreciable effect on municipal water supplies

*Hazen and Sawyer, New York City.

taken from the rivers further downstream. In states predominantly agricultural in character, political considerations are likely to favor the farmer, and the municipalities will have to scratch to get the necessary supplies. To date the emphasis has been on individual irrigation developments, but it is reasonable to expect that group action through irrigation districts will follow in years to come. As evidence of the growing importance of farm use, the Corps of Engineers has estimated water requirements to be met from the Potomac River in 1970 as follows:

Water supply purposes—	375	million	gallons	per	day
Supplemental irrigation—	160	"	"	"	"
Pollution abatement—	650	"	"	"	"
Total	1,185	"	"	"	"

In North Carolina, where there are many textile mills in the relatively small cities surrounded by farming activity, all of these mills are faced with waste treatment in the next few years. The suitability of the treated effluent for use as irrigation water is likely to determine the degree of treatment in many areas.

The reverse effect, that is, the effect of irrigation on water quality in the streams, is not likely to be significant in the eastern states for many years to come. In arid countries, however, the return water from irrigated lands is always more saline than the original supply, and in some places the total solids become so high as to make the water unsuitable.

The importance of irrigation in particular areas is illustrated by a case which recently has come to my attention. In the Beaufort, S. C., area, ground water has been used for many years to serve the local population, the Marine Corps camp on Parris Island, the Naval Hospital and the Naval Air Station. In recent years a large truck farming activity has been added for which the water requirements are substantial, perhaps enough to jeopardize the adequacy of the existing ground water supply. It has been suggested that the irrigation water be taken from a deeper strata which is known to yield water high in fluorides in order to preserve the upper strata for domestic purposes. Information is being collected on the existing water supplies in the area for an appraisal of the safe yield and determination of the future program.

Future irrigation requirements are stressed here because in so many places they are beginning to have an important bearing on our eastern water supply developments. The next 10 or 15 years will see many changes.

For the sanitary engineer, conflicts between water uses develop in respect to both the quantity and quality of water. The conflicts between water users most frequently involve one or more of the following:

(1) Diversion of water from one drainage area to another. This reduces the flows below the diversion and may result in inadequate water supplies, inadequate dilution of wastes, and greater salt water intrusion from the ocean.

(2) Pollution of water supplies, fish and shellfish areas, and recreational facilities by sewage and industrial waste.

(3) Salting of fresh water bodies by the construction of navigation channels at the lower end of coastal rivers.

The pollution of ground water resources with sewage or industrial wastes, and the salting of ground water resources by changes in adjacent stream channels for navigation and other purposes are important in some sections of the country and frequently are difficult to assess and control properly. However, the scope of this paper is limited to surface water problems.

Where topography permits, upstream impoundments and diversions have been the most favored methods of obtaining municipal and industrial water supplies throughout the east. This has been especially true where the drainage area was such that the water could be used without filtration. Further developments along these lines become less favorable because of the cost and difficulty of finding suitable reservoir sites, because it is usually necessary to release a substantial flow of water during the dry season to meet downstream uses, and finally because the water frequently must be filtered in any event.

The release of water for downstream uses increases substantially the cost of impounding reservoirs. In connection with some industrial water supply studies in North Carolina, we investigated the possibility of storage reservoirs on several of the coastal rivers. In each case release of water downstream below the reservoir during periods of drought would be needed not only to take care of existing water uses, but more important in order to provide dilution water for the waste that would be discharged from the proposed mill. Generally, in such situations even though adequate waste treatment is proposed, the dilution water requirements are harder to meet than the water supply requirements.

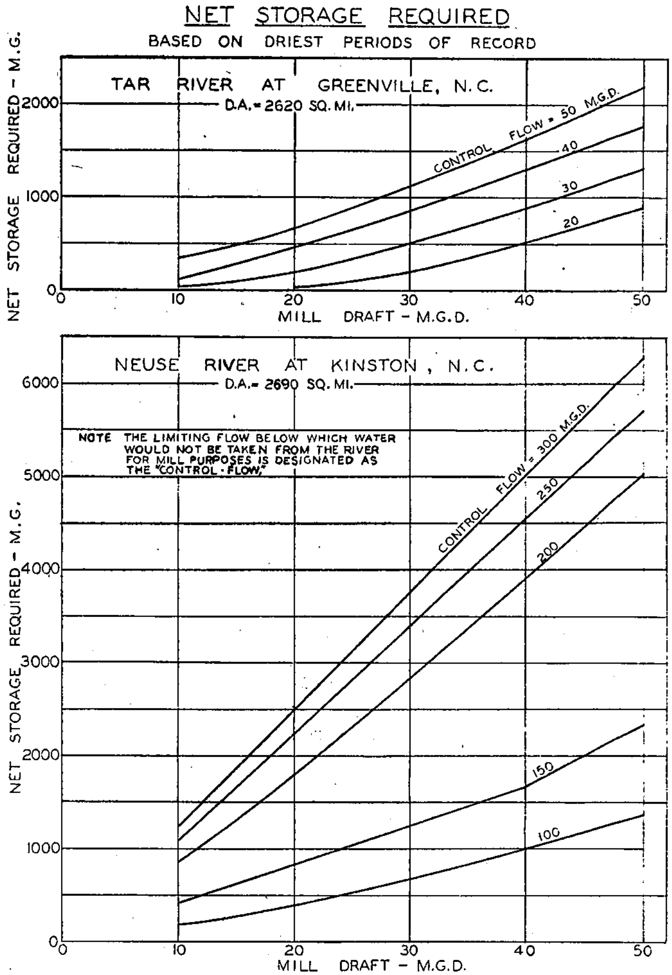


FIG. 1.

Figure 1 shows the effect of downstream releases on the quantity of storage to be provided. It will be noted from these graphs that, for example, a 40 m.g.d. supply from the Tar River with no diversion when the natural flow was less than 20 m.g.d. would require a 500 million gallon storage reservoir. If the downstream limit were increased to 40 m.g.d. the storage requirements would be 1,300 million gallons, or 2½ times as much. The data for the Neuse River shown at the bottom of Figure 1 indicates even greater storage to meet downstream requirements. Similar studies for rivers in Pennsylvania, Maryland, and New York show comparable ratios.

It should be noted that the previous calculations indicate the additional storage needed if no diversions are made when the natural flow drops to specified limits. If releases are required from the reservoir to maintain downstream flows equal to these limits, at all times, the storage reservoirs must be larger—usually 25 to 50 per cent.

As concrete evidence of the importance of downstream releases, one may cite the Delaware River development of New York City now under construction. Approximately 1/3 of the reservoir capacity is reserved specifically in order to maintain low flows in the Delaware River.

The required downstream releases are sure to increase in the future, and provision for them will add substantially to the cost of upland water supplies.

The effects of pollution on water supplies, shellfish, commercial and sport fishing, and recreational facilities need no comment. These are competing water uses. However, in nearly all parts of the United States, unrestrained pollution is no longer tolerated, and some degree of treatment is required. Only rarely is it necessary to approach clean-water conditions in sewage and waste treatment, because of the natural purification available in receiving rivers and lakes. This is fortunate because of the high cost of providing truly complete treatment. Economy dictates that we reserve a part of our water resources to assist in waste disposal.

The importance of dilution water may be illustrated by the situation in Greensboro, N. C., where we are presently engaged in the development of additional water supply and waste disposal facilities. This is a sizeable project with many facets, and will require major additions to existing works. The two sewage treatment plants will have to be enlarged, and it is expected that one of the plants will be a combined sewage-textile waste treatment plant. Pilot plant studies are now underway, with the assistance of Camp, Dresser & McKee, Consulting Engineers of Boston.

Greensboro is located at the extreme upper end of the Cape

Fear River basin. All of the nearby streams are small, and there is little water available for water supply and waste disposal (see Figure 2). The existing water supply is taken from Lake Brandt, an 800 m.g. reservoir impounding the runoff from a 70 sq. mi. drainage area. The safe yield is approximately 12 m.g.d. A second reservoir

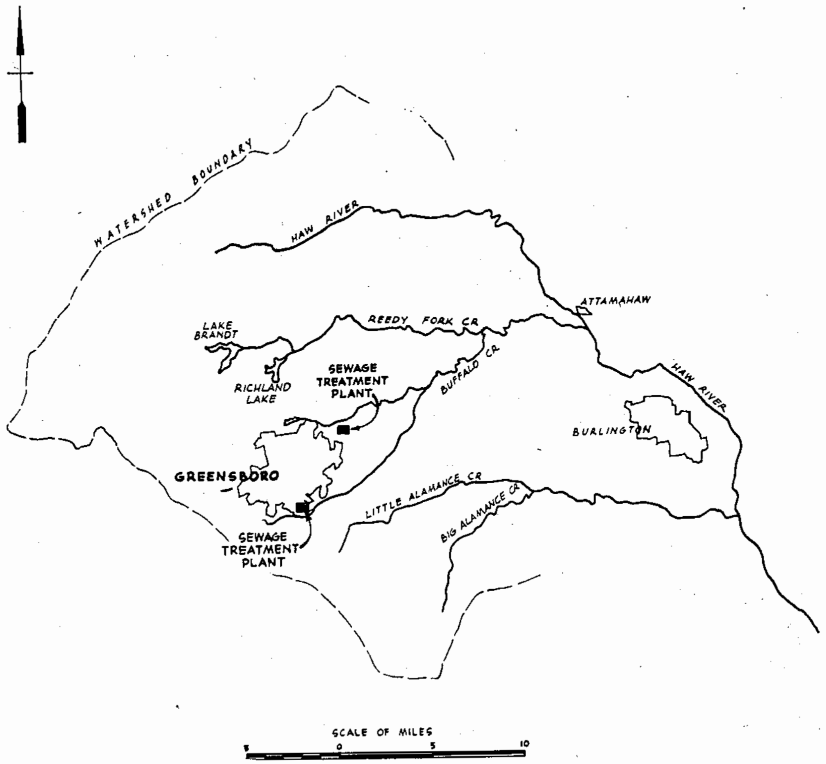


FIG. 2.—REGIONAL MAP, GREENSBORO, N. C.

now under construction will increase the yield to 15 or 16 m.g.d., but is recognized as a stop-gap measure. Richland Lake, also in the Reedy Fork drainage area, is owned by Cone Mills, a large textile concern, and furnishes 4 to 5 m.g.d. industrial water for mill use.

The wastes from the city and much of the outlying area are treated at two plants: one on North Buffalo Creek and the other on South Buffalo Creek. Both plants are overloaded, and conditions in Buffalo Creek and the lower part of Reedy Fork Creek are far from satisfactory. Aside from biological considerations, the color of the textile dye wastes in North Buffalo Creek is sometimes a source of complaint. The pilot plant studies are directed toward ultimate treatment of the textile mill waste in the North Buffalo plant. This will reduce the BOD loading, but no satisfactory answer to the color problem is in sight.

The section of water most seriously affected by pollution is the lower (7-mile) stretch of Reedy Fork, between Buffalo Creek and the Haw River. It is generally recognized that the most useful function of Buffalo Creek and its two branches is to carry off the waste of the Greensboro area. The treatment to be provided must be adequate only to prevent nuisance here. Reedy Fork Creek, however, is in a somewhat different category, and farmers along it have urged the N. C. State Stream Sanitation Comm. to see that the water is good enough for supplementary irrigation. The Cape Fear River basin is now under study by the Stream Sanitation Committee, and classification of the tributaries will be completed within a year or two. How far the Committee will go will undoubtedly depend upon the pressure exerted by downstream owners and the feasibility of meeting fully their demands.

Stream surveys in years past, and especially during the dry summer of 1954 showed severe oxygen deficiencies in the lower part of Reedy Fork Creek. The Haw River joins Reedy Fork Creek below Attamahaw, almost doubling the drainage areas, and conditions between there and Burlington are reasonably satisfactory. The dry-weather flows of all of these rivers are small, as indicated by probability plots of minimum monthly flows for two streams in the vicinity of Greensboro (Figure 3). The low monthly flows in an ordinary year are only about 0.2 c.f.s. per sq. mi., and every 10 or 15 years the minimum monthly runoff may drop to 0.1 c.f.s. per sq. mi. Furthermore, low flows are likely to persist for several months.

The present waste loads and the water available for dilution at various points along the stream are summarized in the following tables.

From the preceding data it is evident that even with high treatment efficiencies, difficulty must be anticipated in Buffalo Creek, and even the lower part of Reedy Fork will suffer from time to time. This is of prime importance to Greensboro because one of the most promising methods of increasing the City's water supply would be to build additional reservoirs on Reedy Fork. Our studies have shown that by utilizing a drainage area of 100 square miles and increasing the total storage capacity to perhaps 4,700 m.g., a safe yield of 35 to 40 m.g.d. could be obtained. However, if this is done, dilution water in Reedy Fork will be decreased still further.

A. SEWAGE AND WASTE LOAD

Plant Influent	*(1) North Buffalo Creek*	*(2) South Buffalo Creek*	*(3) Total*
Municipal sewage m.g.d.	5.6	2.3	7.9
Industrial waste m.g.d.	5.6	1.7	7.3
Total	11.2	4.0	15.2
Population equivalents (BOD)	215,000	70,000	285,000

B. RECEIVING WATER AVAILABLE

	(1) At North Buffalo S.T.P.	*(2) At South Buffalo S.T.P.*	*(3) At Mouth of Buffalo Creek*	*(4) At Mouth of Reedy Fork Creek*
Drainage area—s.m.	23	30	100	259
Estimated mean annual flow—c.f.s.	25	32	107	250*
Estimated dry monthly stream flow—c.f.s.:				
Once every 2 years	5	6	21	40**
Once every 5 years	3	4	14	27**
Once every 10 years	2.5	3.5	11	21**

*Less 22± c.f.s. diverted for water supply purposes by Greensboro and Cone Mills.
**Assuming no releases from Lake Brandt.

On the other hand, if the additional storage were used not only for water supply but also to supplement dry-weather flows of Reedy Fork Creek, we could obtain for a few years both water supply and waste disposal benefits. We have estimated that for several years the lower part of Reedy Fork Creek can be maintained in good condition if minimum flows are controlled to approximately 30 c.f.s., or 20 m.g.d. in addition to the plant discharges. The storage required to provide this flow under ordinary conditions (say 90 per cent of the time) would be approximately 900 m.g. or 20 per cent of the anticipated water supply requirements. A reservoir built now to meet water needs several years hence would have surplus capacity that could be used to advantage in the immediate future for waste dilution.

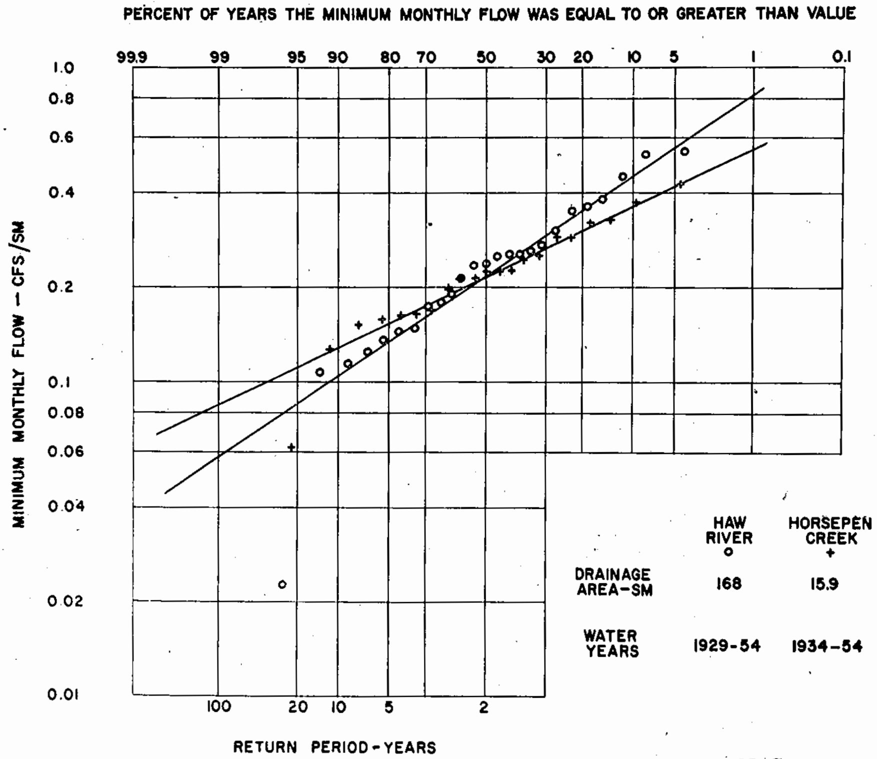


FIG. 3.—PROBABILITY PLOT OF MINIMUM FLOWS NEAR GREENSBORO, N. C.

Ultimately, Greensboro and several of the adjacent cities probably will have to obtain water from a larger river, some distance away. The Yadkin River, west of Greensboro, seems the most likely source, and the possibilities of a regional development are being studied. For Greensboro, not only the relative cost of a regional supply, but the ultimate solution of its waste disposal problem must be considered. A supply from the Yadkin will involve inter-basin diversion and reduction of stream flows already used for hydropower development. Regional water supplies, involving a number of cities and varied interests often take years to develop, and the final outcome cannot be anticipated. However, Greensboro is fortunate in that whatever it builds on Reedy Fork Creek can be used to good advantage ultimately to improve waste disposal if the regional water supply scheme is realized. The construction of low-flow regulating reservoirs for pollution abatement costs more than waste treatment to accomplish the same results. In this case, where natural flows are so low, both measures may be needed.

An entirely different situation exists in the Detroit, Michigan, area, where a practically unlimited supply of water is available from the Detroit River (see Figure 4). The Detroit River connects Lake St. Clair with Lake Erie. The flow past Detroit ranges from 104,000 to 241,000 c.f.s., and averages 177,000 c.f.s. The Detroit water supply is taken from the upper end of the river. Other municipal intakes are located in the river at Windsor, Wyandotte, and Amherstburg, and in Lake Erie near Point aux Peaux for Monroe.

Windsor and Amherstburg do not have sewage treatment plants. On the U. S. side most of the sanitary sewage receives primary treatment and chlorination at the Detroit plant before it is discharged into the Detroit River just above the mouth of the Rouge River. There are two smaller sewage treatment plants in Dearborn which discharge into the Rouge River, and two Wayne County plants which discharge into the Detroit River below Wyandotte. The principal sources of pollution in the Detroit River are the sewage treatment plant effluents and storm overflows from combined sewers on the U. S. side, raw sewage discharges on the Canadian side, and industrial waste discharges on both sides of the river. It is obvious that the Detroit River plays an essential part in carrying off the wastes of this large metropolitan and industrial area. At the same time it serves as a primary shipping artery of the nation, as an important recreational area, and as the source of public and industrial water supplies. That the lower Detroit River can be used for water supply purposes with safety is due to the unusual stratification of the water. The tremendous volume of water and negligible lateral mixing protect the water in the center of the stream from shore pollution.

We have just completed an investigation of the feasibility of supplying water to the area south and west of Detroit from an intake in the lower Detroit River or western Lake Erie. This area is growing rapidly and additional works are needed. The cost of bringing

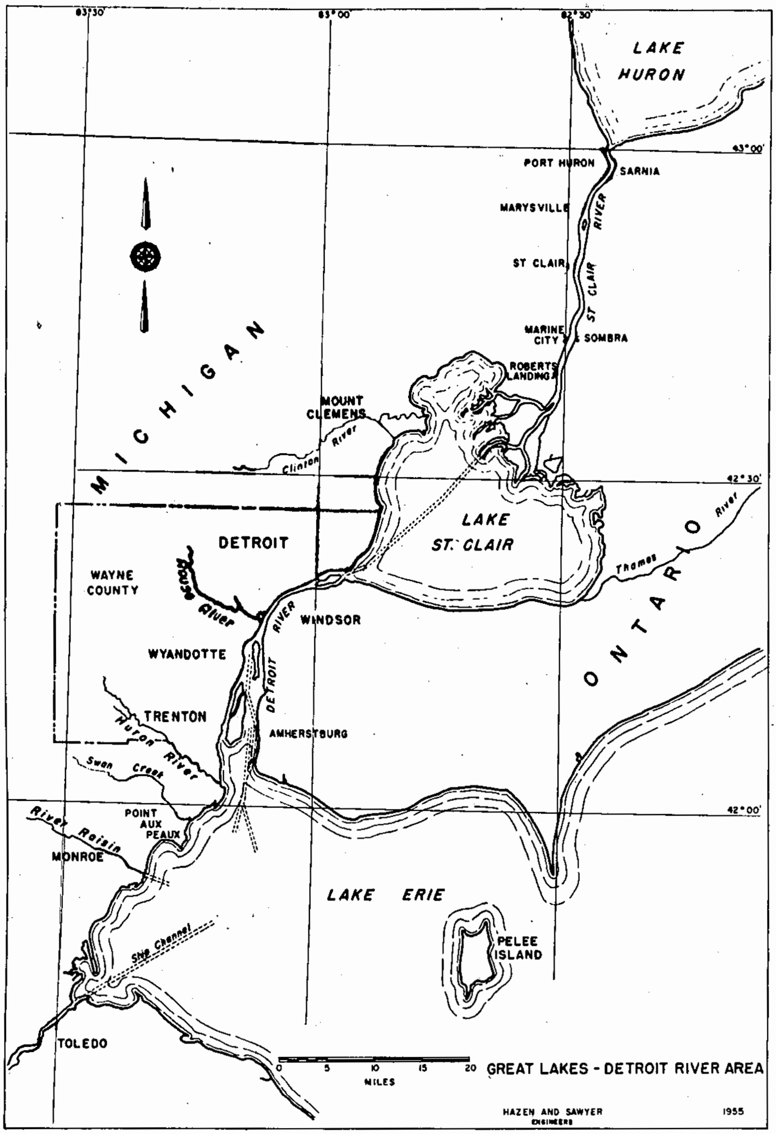

83°30'
83°00'
82°30'
43°00'
42°30'
42°00'
LAKE HURON
PORT HURON
SARNIA
MARYSVILLE
ST CLAIR
ST CLAIR RIVER
MARINE CITY
SOMBRA
ROBERTS LANDING
MOUNT CLEMENS
Clinton River
MICHIGAN
LAKE ST. CLAIR
ONTARIO
Thames River
DETROIT
WAYNE COUNTY
Rouge River
WINDSOR
DETROIT RIVER
WYANDOTTE
TRENTON
Huron River
AMHERSTBURG
Swan Creek
POINT AUX PEAUX
River Raisin
MONROE
LAKE ERIE
PELEE ISLAND
Ship Channel
TOLEDO
0 5 10 15 20
MILES
GREAT LAKES - DETROIT RIVER AREA
HAZEN AND SAWYER ENGINEERS
1955

FIG. 4.

water from the upper part of the river, or still further north, through the city to the area to be served would be extremely high, and the development of a nearer supply would have many advantages. Our report has not yet been released, and I am not free to discuss the findings. However, the two illustrations following show clearly the segregation of clean water from polluted water, and areas in the river where an intake could best be located.

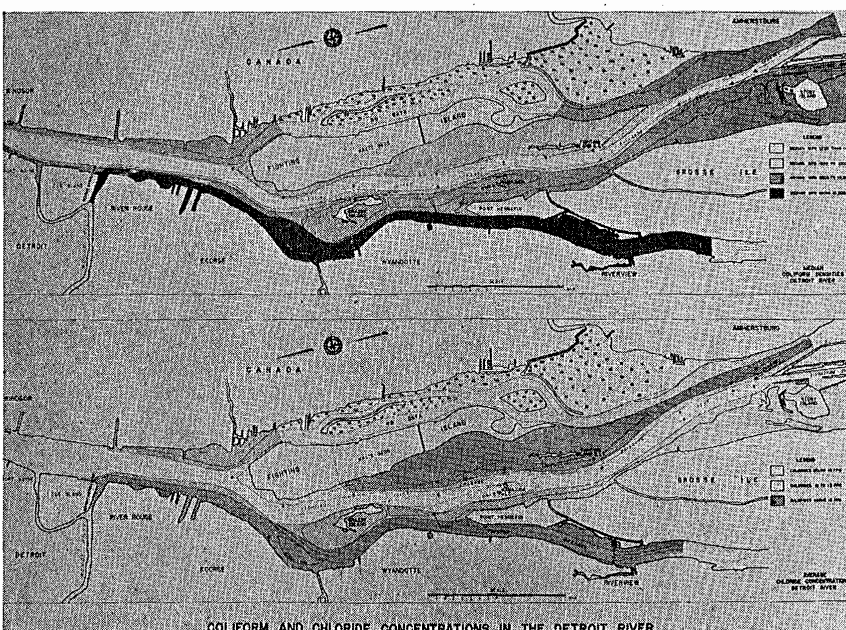


FIG. 5.

The upper part of Figure 5 shows the distribution of median coliform density in the Detroit River between the Rouge River and upper end of Grosse Ile. The white area shows the part of the river with a median MPN of less than 1,000 p.p.m.; the most heavily shaded area represents median MPN's greater than 10,000. Extreme values, represented by the 90% value and A.M.D. are somewhat higher, but follow the same pattern. The data collected in 1955 confirm the results of the I.J.C. surveys in 1913 and 1946-48, and numerous other studies to show the distribution of pollution.

The lower part of Figure 5 shows the chloride distribution over the same stretch of the river. The chloride concentration parallels the sewage contamination as would normally be expected. However, much of the chlorides are due to industrial waste, especially along Fighting Island, where there is leaching from chemical waste beds on the island.

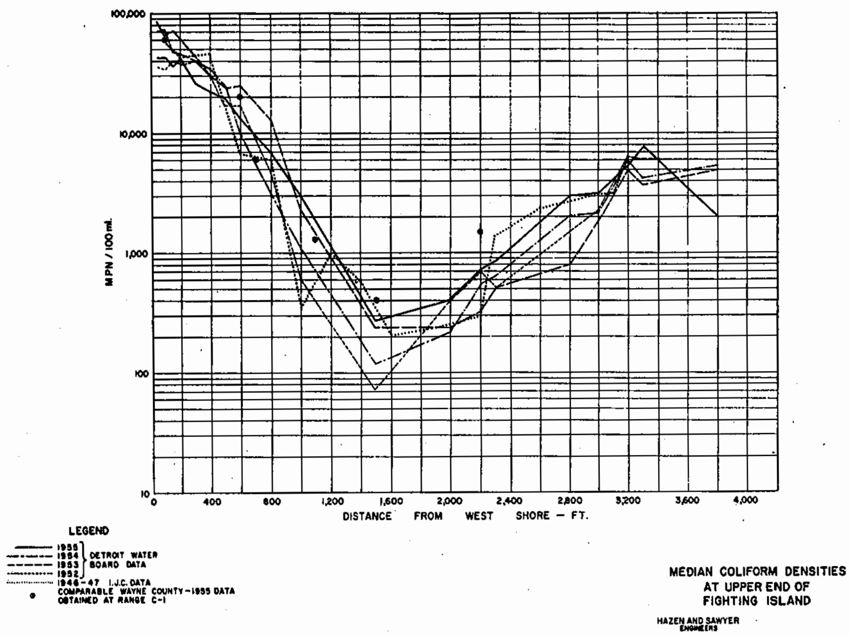


FIG. 6.

The lateral distribution of coliforms is brought out sharply by the profiles shown in Figure 6 representing the median MPN values at the head of Fighting Island as determined by surveys over six different years.

The situation at Detroit is an unusual one. It shows that water below a large city is not necessarily bad, and that by taking proper precautions provision can be made for both water supply and waste disposal. As a practical matter, the most serious water supply problem—including the City's water supply—is the severe taste following occasional discharges of phenol-bearing wastes upstream. The Detroit, Wyandotte, Windsor, Amherstburg and Monroe water purification plants have consistently produced safe water, but taste control is still a problem.

The conflict between water supply and navigation requirements is reflected frequently by the encroachment of salt water caused by the deepening and widening of shipping channels near the mouth of coastal rivers. Wilmington, N. C., for years obtained its water supply from the Cape Fear River within 2 miles of the city. By 1941 channel improvements permitted the salt to reach many miles upstream, and a raw water line had to be extended to the first navigation lock 22 miles away. At Savannah, Ga., the same thing occurred, except that until about 1940 the City used wells and was not dependent upon the Savannah River. When a surface supply was built, the intake had to be located several miles inland.

In considering several of the North Carolina rivers as a source of industrial water supply, the existing or proposed navigation requirements made it necessary to allow for locating the intake several miles upriver to avoid salt. The necessary pipelines, or canals in some instances, represented a substantial part of the estimated cost. In some cases, the actual navigation benefits were nil, or extremely small. It would seem time to consider the possibility of abandoning the rarely-used ship channels and the installation of low dams to prevent salt water encroachment. In many areas the water supply benefits would more than make up for losses in shipping.

Lake Maracaibo in Venezuela has been an important source of fresh water down through the years. The total solids and salinity in the lake water reached limits higher than desirable from time to time, but the water was usable. Enlargement of the shipping channel near the mouth, combined with low stream runoffs, have aggravated the situation in recent years, and the lake is fast losing its value as a source of water supply. Ground water is limited and the lake supply will be missed.

Perhaps the most important case in recent years involves the proposed dredging of a deep-water channel in the Delaware River, above Philadelphia, to the Fairless Works of U. S. Steel. The effect of salt water encroachment on water supplies in the area is of major importance, and several investigations have been made to determine whether or not the channel enlargement would be harmful. These investigations, including extensive model studies at Vicksburg, have indicated that the channel can be built safely.

The few instances discussed, and countless others, demonstrate the competitive situation into which water supply and other water-using activities are moving. From this certain conclusions as to future trends and needs seem obvious:

1. the growing demands for water for all purposes—and especially for irrigation—are going to tax the water resources in many eastern states unless reservoirs are built to store flood flows for use later in the dry season. Our water law, based largely on the riparian doctrine, is in for rough sledding. A sudden switch to the appropriation doctrine of the west is not likely—and in itself will produce no more water—but a movement in that direction seems inevitable. Organization of public water districts, with the power to condemn lands and rights of way, is receiving support in many states where it is recognized that regional action is needed.

2. Fewer and fewer cities will enjoy the benefits of pure, impounded mountain water. Present-day water treatment processes permit the use of water contaminated beyond limits previously established. Emphasis will have to be placed on reliability of treatment and the ability to take shock loads. Included in this are more frequent and more rapid methods of raw water analysis so that trouble can be detected soon enough to do something about it.

3. There is great need for fully effective and reliable taste and odor control. The public is more influenced by taste than any other factor. If we can learn to consistently produce a safe water that tastes good, many raw water supplies heretofore dismissed can be used successfully.

4. On the other side, we are going to have to insist on continuous and certain treatment of sewage and wastes. Our water treatment methods are based upon extreme conditions, and a normally good source of supply may be rejected because at rare intervals upstream waste treatment fails. When the regularity of sewage and waste treatment operation reaches that of most water purification plants, shock loads will be at a minimum. The added cost of providing regularity and reliability is not excessive in most instances.

5. In industrial waste treatment, particular emphasis must be placed on the elimination of toxic and taste-producing substances. In some instances money so spent would be far more useful than "primary treatment" or "complete treatment" according to some statewide formula.

SITE PRELOADING ELIMINATES PILES FOR TWO OIL STORAGE TANKS

BY HARL P. ALDRICH, JR.,* *Member*

(Presented at a meeting of the Structural Section, B.S.C.E., held on November 14, 1956.)

SYNOPSIS

The Metropolitan Coal Company proposed to construct two fuel oil storage tanks 120 ft. in diameter and 60 ft. high at its branch in Chelsea, Massachusetts. Soil conditions at the site consist of approximately 13 ft. of miscellaneous granular fill underlain by from 6 to 13 ft. of soft organic silt and inorganic silty fine sand. Stiff yellow clay, sand, gravel, and boulders occur below this compressible stratum.

The problem was one of selecting a suitable foundation treatment for the oil storage tanks. These site conditions would normally require a pile foundation but after considerable study it was concluded that a site preloading operation could be successfully carried out at less than half the cost of a pile foundation. Therefore, approximately 28,000 tons of sand and gravel were placed first at one site, then moved to the second site for the purpose of precompressing the underlying soil to minimize foundation settlement.

This paper describes the soil engineering investigation for the design and control of site preloading for these oil storage tanks.

INTRODUCTION

Site preloading consists of applying a dead load or surcharge over the site for a proposed structure, generally equal to or greater than the total weight of the structure. After compression of the underlying soil has occurred under the preload, which is usually an earth fill, the preload is removed and the structure is built. Thus, the purpose of site preloading is to develop settlements before construction and therefore to minimize the structural settlement.

The theory behind the principle of preloading, which involves the nonelastic nature of soils, is now well known to soil engineers. Briefly,

*Assistant Professor of Soil Mechanics, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

when a soil stratum first experiences a given load it compresses over a period of time under that load. If the load is removed, the stratum will usually expand a small percentage only of the compression it underwent. Finally, if the load is reapplied the recompression will be small, perhaps only slightly larger than the expansion. The relative magnitudes of initial compression, expansion, and recompression depend on the soil type, thickness of the stratum and time of loading. The most complex problems occur where organic soils are encountered.

While preloading for structures is relatively uncommon, the procedure was used for bridge approach fills and other engineering projects long before the advent of soil mechanics. Four years ago, Stanley Wilson, then Assistant Professor of Soil Mechanics and Foundation Engineering at Harvard University, gave a paper before the Structural Section on the "Control of Foundation Settlements by Preloading."[1] Mr. Wilson described in detail the fundamental load-settlement characteristics of soil and then described several projects where preloading had been applied successfully. Site preloading for a cathedral in Baltimore was reported in the Engineering New Record, April 28, 1955. Dr. Arthur Casagrande was consultant on this project. In recent years reports on various forms of preloading and overloading in connection with earth embankments for highways have become common.

I would like to describe the soil engineering investigation for the design and control of site preloading for two large oil storage tanks. This has been one of the most interesting and challenging projects in which I've participated largely because of the nature and scope of the field data which were obtained. This was possible only through the complete cooperation of the owner, the engineer, and the contractors, all of whom recognized the importance of the undertaking.

Preliminary Considerations

The Metropolitan Coal Company proposed to construct two oil storage tanks, 120 ft. in diameter and 60 ft. high, at its Chelsea, Mass. branch on Broadway Street near the north end of the Mystic River Bridge. Mr. Everett C. Hunt of Hunt and Slayter, was engineer for the 120,000-barrel tanks. I served as soils consultant to Mr. Hunt.

---

[1]Journal of the Boston Society of Civil Engineers, January, 1953.

Site Conditions:

Figure 1 is a photograph of the site taken from the Mystic River bridge. The open area where the tanks will be constructed, is studded with concrete piers which supported an elevated track used to convey coal to the area. The track and supporting timber structure were demolished in preparation for the new tanks shortly before this pic-

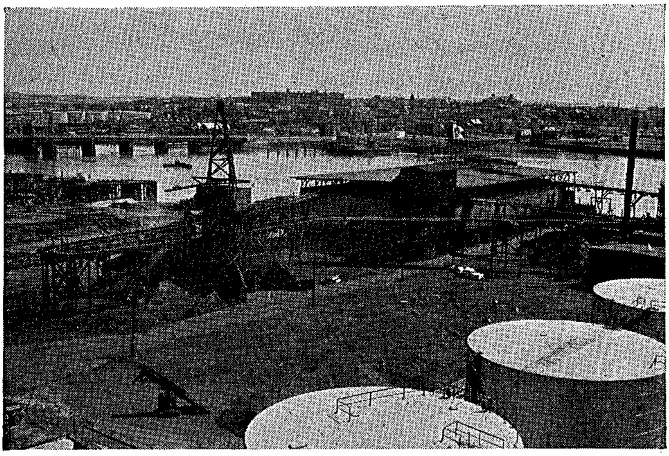


FIG. 1.—PHOTOGRAPH SHOWING PROPOSED SITE.

ture was taken. The remains of the coal pile, at times as high as 33 ft., are shown in the lower central part of the picture. The small building with stack at the far right of the picture is the Company's boiler house. The Chelsea River in the background joins the Mystic River to the right of the picture.

A site plan showing the maximum extent of the coal pile is shown in Figure 2. The proposed locations for the oil storage tanks and steel safety dykes are shown by dotted circles. It can be seen that both sites have already experienced various degrees of preloading from the coal pile over a long period of time. However, since coal weighs about 60 lb. per cu. ft. only, the maximum preloading is approximately 1 ton per sq. ft. which is less than 60 per cent of the future load under the oil storage tanks. The east edge of the site has experienced no preloading.

Soil Conditions:

Eight core borings were made by the Raymond Concrete Pile Company in January and February of 1955. Locations of these borings are shown on Figure 2 by Nos. 1 through 8. Soil conditions disclosed by these borings are summarized in Figure 3 which shows cross-sections, looking north, through the center of the proposed tanks.

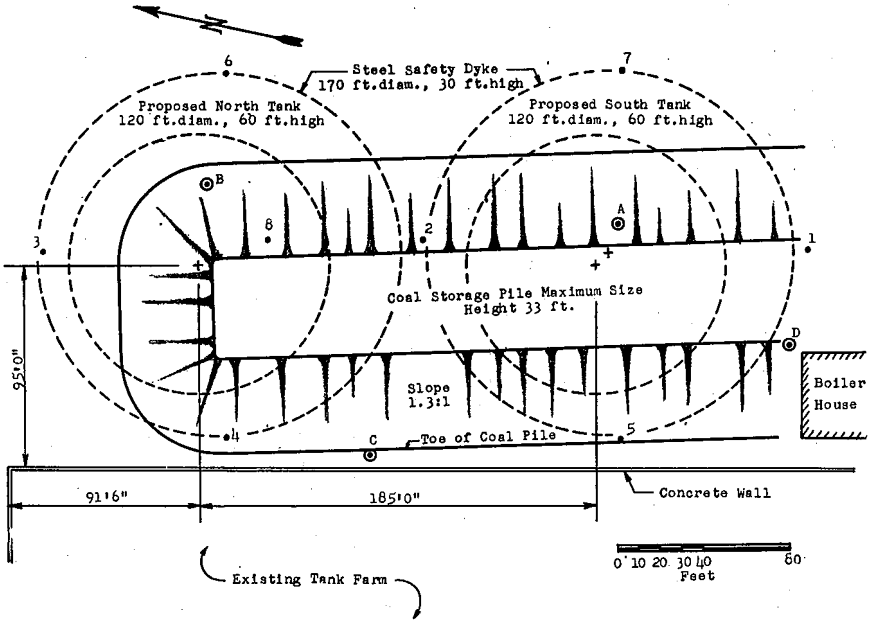


FIG. 2.—SITE PLAN SHOWING LOCATION OF BORINGS.

Approximately 13 ft. of miscellaneous granular fill having a standard penetration resistance (blows per foot on the split spoon sampler) of from 3 to 60. The original surface before filling varied from mean sea level near the boiler house to El. +3 at the North end of the site. Soil conditions in this former tidal flat vary from soft grey organic silt and silty fine sand to grey inorganic silty fine sand. The thickness of this compressible stratum is 13 ft. at the South tank site tapering off to about 6 ft. at the North tank site. Stiff yellow clay, sand, gravel and boulders occur below the compressible stratum. Some evidence of soft blue clay, peat, and medium yellow clay was observed in various borings. It should be noted that the vertical scale in Figure 3 is exaggerated. In reality, the compressible strata are thin relative to the tank size.

These soil conditions combined with the fact that the tank sites had experienced varying degrees of preloading positively eliminate the possibility of constructing the tanks without piles or some special foundation treatment.

The Preloading Decision:

Three possibilities for providing adequate foundations for the proposed tanks were studied at various times by Mr. Hunt, by Mr. Charles C. Ladd[2] and by myself. These foundation treatments were:

1. The use of bearing piles, driven into the hard yellow clay, to support the entire load.
2. The removal of existing fill and all other compressible soil and replacing it with compacted granular fill.
3. Preloading the sites to precompress the underlying soil for the purpose of minimizing differential tank settlements.

Several general requirements were observed during these studies. First, the size and location of the tanks could not be changed. The base of each tank should not be below El. +15 to provide adequate safety against excessively high tides. Finally, one tank would store No. 2 domestic fuel oil weighing 53 lb. per cu. ft. while the second tank would store No. 6 fuel oil weighing 60 lb. per cu. ft. Therefore, the design uniform loads for tanks 60 feet high were 1.6 and 1.8 tons per sq. ft. The weight of the welded steel tank is negligible.

Pile Foundation: Creosoted wood piles driven into the stiff yellow clay would vary from 25 to 35 feet in length and could be designed for 16 tons per pile. The cost to the pile foundation with a reinforced concrete pile cap was estimated to be from $200,000 to $250,000 for the two tanks. This is only slightly less than the cost of the two steel tanks erected on prepared foundations.

Excavation of Compressible Soil: If the depth of compressible soil is not excessive it is frequently economical to remove the soil and replace it with granular fill compacted under carefully controlled conditions. In this case excavation would have to be carried to a maxi-

[2]"Design and Comparative Analysis of an Oil Storage Tank Foundation," S.B. Thesis, M.I.T. May, 1955, unpublished.

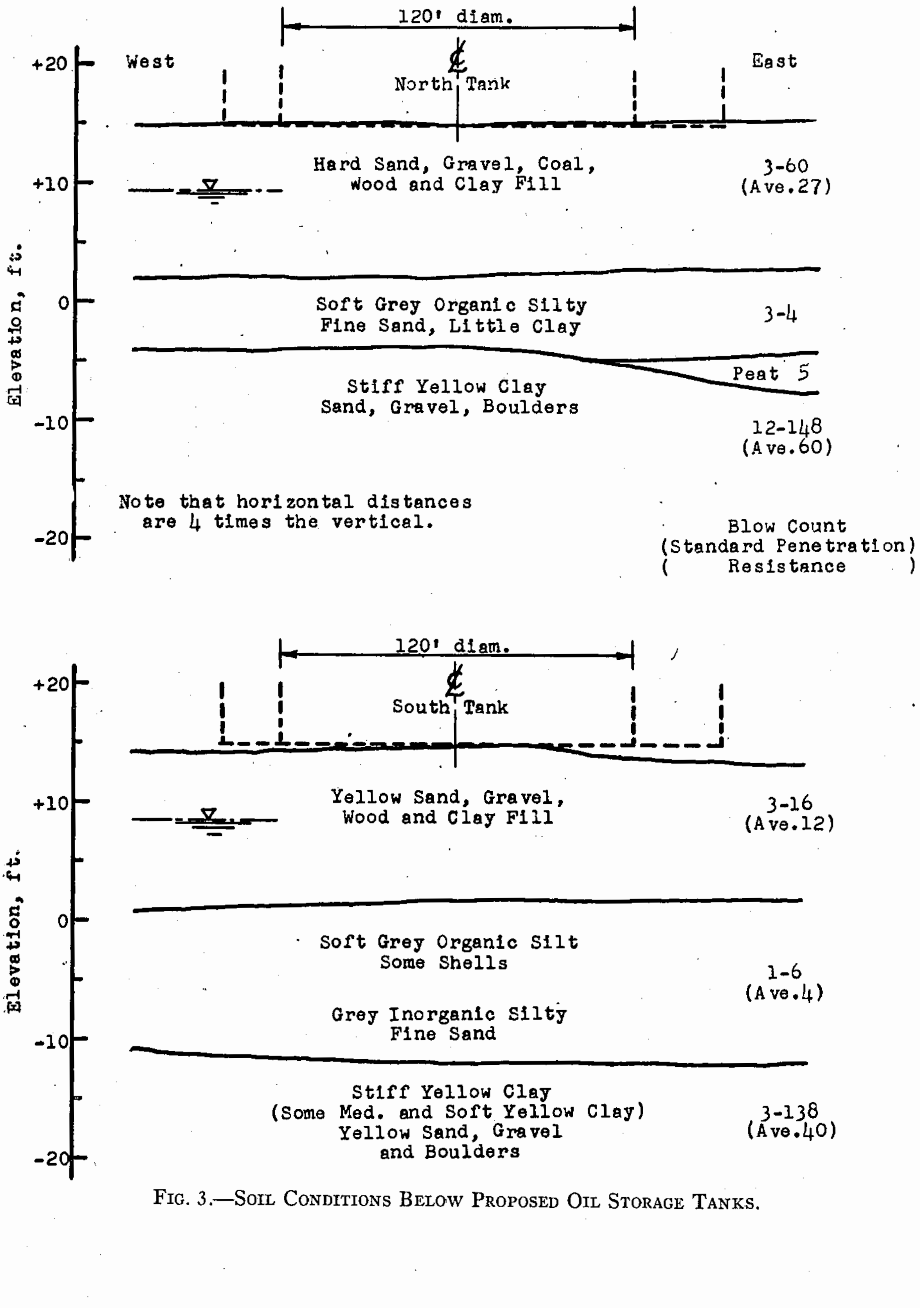


FIG. 3.—SOIL CONDITIONS BELOW PROPOSED OIL STORAGE TANKS.

mum depth of 28 ft. or 20 ft. below ground water level. Therefore, because of the confined area and soil conditions, anchored steel sheet piling would probably be required. Although this possibility was not explored in detail, a rough estimate of the cost of sheeting, excavation of fill and other compressible soil, and the cost of a compacted granular backfill exceeded $200,000 for the two tanks.

Site Preloading: A minimum preload of 1.8 tons per sq. ft. on the foundation soil would require a mound of earth at least 27 ft. high. If the soil were piled in the shape of a truncated cone covering most of the tank area, about 16,000 cu. yds. would be required. One site could be preloaded immediately while a sandy gravel pad was placed at the second site. The preload could then be moved to the second site while the first was prepared for its tank. After all preloading was completed, a portion of the material would be used to dress the area while the remainder would be sold.

The estimated cost of the preloading operation plus reinforced concrete rings below the tank sheets and additional site preparation for the tanks was less than $100,000 for both tanks. Therefore, a saving in excess of $100,000 was indicated if the preloading procedure could be carried out successfully. Several important questions arose, however. How long would each site have to be preloaded? How much settlement would the tanks experience after the sites were preloaded? Would the tanks be adequately safe against a shear or displacement failure in the foundation soil? Answers to these questions appeared favorable from the available data but additional subsoil exploration and laboratory tests on undisturbed samples were indicated.

Laboratory Tests on Undisturbed Samples

In February, 1955, the Raymond Company was directed to make four borings for the purpose of obtaining undisturbed samples by means of the 3-inch fixed-piston sampler. The location of these borings, A through D, is shown in Figure 2. Good undisturbed samples were obtained in Borings A and D only, because of the difficulty in advancing the drill hole through the miscellaneous fill and because of shells and other foreign material in the silt stratum.

The extreme variation in soil types within the compressible stratum is evident when results of Atterberg Limits are plotted on a plasticity chart as shown in Figure 4. From these results and visual examination of numerous samples from all borings, the following general observation is apparent. The top half of the compressible stratum below the South tank is a soft organic silt of medium to high plasticity and indeed compressibility, while below a depth of 20 feet (El. —5) the soil is a non-plastic inorganic silty fine sand. Consolidation of the compressible stratum during preloading will, therefore, occur far more rapidly than if the entire 13-foot depth were organic

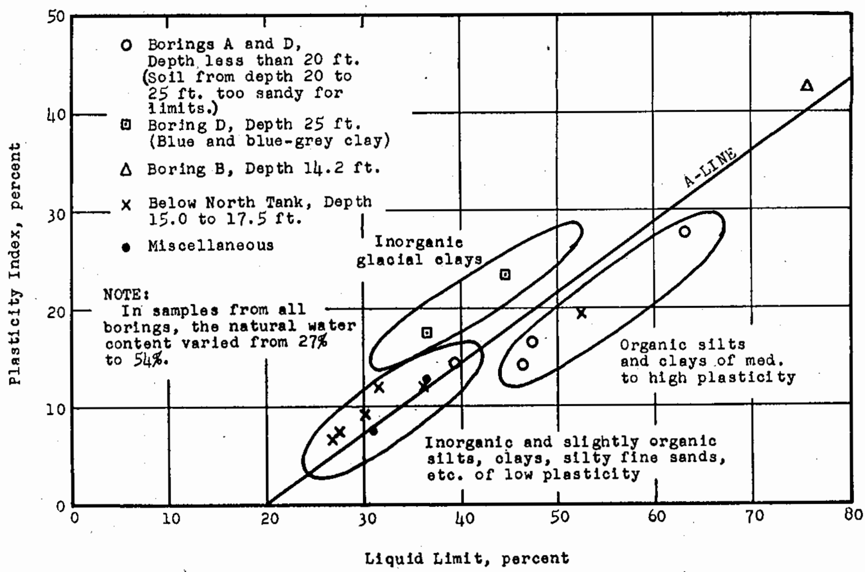


FIG. 4.—PLASTICITY CHART SHOWING EXTREME VARIATION IN ATTERBERG LIMITS FOR SOILS WITHIN THE COMPRESSIBLE STRATUM.

silt. In addition, long term secondary compression will be less. Soil within the compressible stratum below the North tank appears to fall between the extreme classifications found below the South tank.

Consolidation tests were run on seven samples from Borings A and D. Results of these tests, summarized in Table 1 show a wide variation in compressibility as expected. A measure of compressibility is given by the compression index which varied from 0.5 for samples at the top of the compressible stratum for 0.05 at the bottom. For comparison, the compression index for soft Boston blue clay is about 0.3.

A crude estimate of settlement under a 27 ft. preload fill for the

South tank can be made from consolidation test results according to the expression:

$$\rho = \frac{12\ Th}{1 + e_1} \frac{0.435\ C_c}{p_{ave.}} \Delta p$$

Where:

ρ	Settlement in inches
Th	Thickness of compressible stratum in feet
e_1	Initial void ratio oof compressible soil
C_c	Compression Index
$p_{ave.}$	Average intergranular pressure during consolidation
Δp	Pressure increment due to preload fill.

Extreme results may be obtained using the following data:

$Th = 13$ ft.
$p_{ave.} = 1.6$ tons per sq. ft.
$\Delta p = 1.8$ tons per sq. ft.

Low Compressibility:

$e_1 = 0.8$
$C_c = 0.07$

High Compressibility:

$e_1 = 1.10$
$C_c = 0.30$

which yield a settlement of from 3 to 11 inches. This result assumes no effect of preloading from the coal and does not include settlement within the existing granular fill overlying the compressible stratum or hard yellow clay below it. Indeed, when one considers these factors, settlement from local pockets of peat and soft clay and the varia-

TABLE 1
CONSOLIDATION TEST SUMMARY

Test No.	*Boring No.*	*Depth (Ft)*	*Ave. Natural Water Content (%)*	*Compression Index* C_c*
1	A	15.1	35.0	0.293
2	A	19.5	32.8	0.159
3	A	21.9	28.7	0.065
4	D	21.4	31.9	0.095
5	D	16.8	48.8	0.497
6	D	23.0	28.0	0.049
7	D	19.0	36.8	0.200

*C_c Compression Index, slope of virgin (straight line) portion of void ratio versus pressure (log scale) curve.

tion in thickness of the compressible stratum, a settlement variation of from 2 to 12 inches under the preload fill at the two tank sites is realistic. For comparison, observed settlements varied from 1.7 inches to 6.4 inches.

Settlement of the oil storage tanks after the sites had been preloaded will be a fraction of the settlement under the preload fill. That fraction depends on the magnitude of preload compared to tank load and on the time of preloading. More important however, is the fact that preloading will reduce undesirable diffierential settlements to a minimum. The only prediction I made prior to construction is expressed in the following statement taken from a letter to Mr. Hunt: "Preloading is expected to last from 3 to 4 weeks. The settlement of a full tank founded on the preloaded soil is expected to be fairly uniform and less than 3 inches over a period of years." While I have no conclusive information relative to allowable differential settlements for oil storage tanks, this prediction is apparently well within tolerable limits.

Attempts were made to run triaxial shear tests on samples from Borings A and D but the results scattered considerably. Difficulty was experienced in obtaining good samples for these tests. Samples were either disturbed, too soft, too granular or contained too many shells. Nevertheless, from the available data it was concluded that there was adequate safety against a shear failure or extrusion failure within the compressible stratum. The stratum was thin relative to the loaded diameter which leads to a higher ultimate bearing capacity than would be computed from conventional formulas. Furthermore, primary consolidation would occur rapidly eliminating excessive pore water pressures.

From results of the laboratory investigation and design analysis it was concluded that site preloading could be used successfully for the oil storage tanks. The Metropolitan Coal Co. and Mr. Hunt agreed then to proceed with the preloading plan.

The Preloading Plan

The preloading plan called for a fill of clean bank run gravel since a good granular soil was required for permanent compacted pads below the tanks. Settlement observation platforms and piezometers were specified to obtain field information for control of the preloading sequence. The contract for preloading and preparation of foundations for the tanks was awarded to the C. J. Maney Co. Elements of the preloading plan and foundation treatment are summarized in Figure 5.

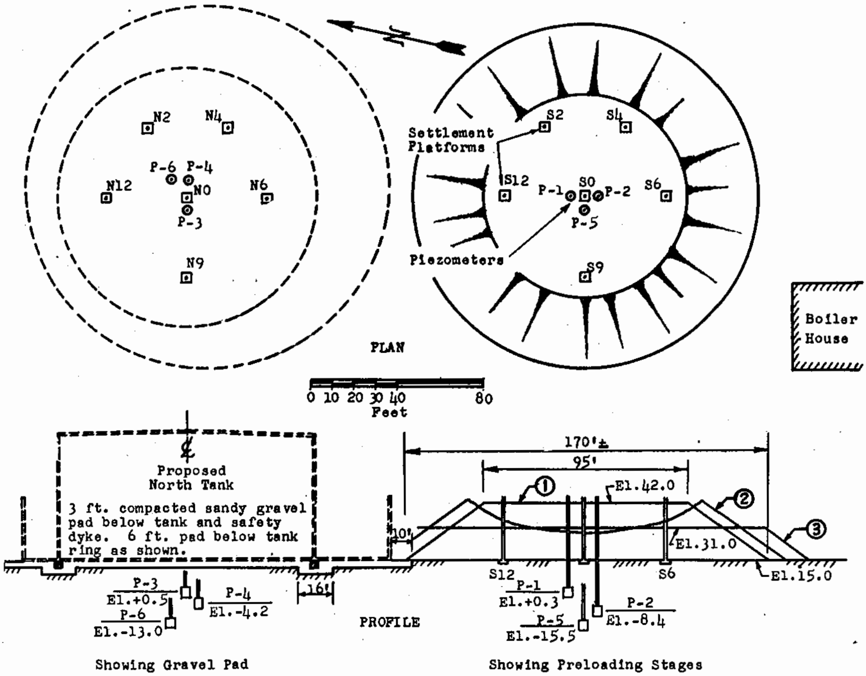


FIG. 5.—LOCATION OF SETTLEMENT PLATFORMS AND PIEZOMETERS DURING SITE PRELOADING FOR OIL STORAGE TANKS.

Settlement Observation Platforms:

Six settlement observation platforms were provided at each site, as shown in Figure 5, for the purpose of recording settlements during preloading. These platforms are numbered clockwise; for example, platform 12 is at the north edge of the fill which is defined as the 12 o'clock position. Platform 0 is at the center of the preload fill.

Each settlement platform consists of 2 ft. 6 in. square base made up of channel sections welded together. A ½ inch diameter steel rod is attached to the base and protected by a 2-inch I.D. steel pipe sleeve also mounted to the base. The base is firmly seated before preloading on a sand cushion at El. +15. The rod and protecting sleeve project upward through the fill. Elevations are taken on the tip of the rod from which settlements may be computed. Sections are added to the rod and sleeve as needed while the fill is placed.

The pipe sleeve was painted a bright yellow to warn the dozer operator. We were fortunate to have an experienced operator who appreciated the importance of the field observations. We did not lose a single settlement observation point or piezometer during the entire preloading sequence.

Piezometers:

In order to observe pore water pressure within the compressible stratum during preloading, three piezometers were installed by the Raymond Company at various depths near the center of each preload fill. These piezometers, the location of which is shown in Figure 5, are a nonmetallic type developed by Dr. Arthur Casagrande for use at the Logan Airport.[3] The piezometer point is a porous stone tube surrounded by a pocket of sand. A plastic tube, ½ inch in diameter, extends to the surface within a steel casing. Measurements of water pressure are computed from observations of water level in the plastic tube. The water level in turn is determined with an electrical sounding device.

Preload Fill:

It is generally desirable to provide a preload fill in excess of the structural load especially if organic soil exists at the site. A preload of from 125 to 150 per cent of the structural load should be used if possible. In this instance, however, the preload was designed to be 100 per cent of the maximum tank load. This would be satisfactory because the full tank load would be effective for a relatively short period of time each year. Furthermore, the tanks could withstand greater settlements than most structures. Finally, from the standpoint of economy, a 10 per cent increase in preload height maintaining the 95 ft. top diameter would require approximately 17 per cent additional earth fill.

Preloading at the South tank site was made in three stages as shown in Figure 5. In the first stage, 28,000 tons of sandy gravel were placed in the shape of a truncated cone 27 ft. high with a top diameter of 95 ft. and bottom diameter of approximately 170 ft.

[3]"Soil Mechanics in the Design and Construction of the Logan Airport," Journal of the Boston Society of Civil Engineers, April, 1949.

Twenty-five calendar days, beginning April 22, 1955, were required to bring the material to the site. Stage 1 preloading remained for 23 days before the pile was scooped out from the middle to form a dish shaped surface to give Stage 2 in Figure 5. The purpose of this preloading stage was to give additional precompression to the soil immediately below the tank ring. This stage remained 4 days before the material was leveled to El. +31 over the full area of the safety dyke. This is referred to as Stage 3.

During preloading for the South tank the foundation pad for the North tank was placed, Figure 5. An area extending 10 ft. beyond the safety dyke was excavated to El. +12 except below the tank ring where a strip 16 ft. wide was excavated to El. +9. About 8,000 tons of clean sandy gravel were imported and compacted in layers by a tractor dozer and loaded truck to bring the surface back to El. +15. On June 14, 1955, the dozer began to move the preload fill from the South tank site to the North.

The North tank preload, Stage 1, was completed in 39 calendar days and allowed to remain 16 days after which 8,000 tons from the west side, where settlements were small, were taken for the South tank pad. At the same time the remaining fill was moved upward toward the east to provide additional preloading below the future tank shell where preload settlements were very high. Sandy gravel was taken from this modified Stage 2 preloading to fill and otherwise grade various areas on the property while the excess was sold.

Field Observations During Preloading:

Settlement observations and piezometer readings during preloading were made daily by Dick Hume of Hunt and Slayer and Al Kapchus of the Metropolitan Coal Co. A summary of these observations is shown in Figures 6a and 6b.

Observation platforms 2 and 4 showed maximum settlements at both sites which was expected. It was noted earlier, Figure 2, that the east edge had not been preloaded with coal. Furthermore, some peat was encountered toward the east of the North tank site, Figure 3.

Maximum observed preload settlements at the South tank varied from 0.14 to 0.34 ft. Rebounds were from 0.028 to 0.038 ft. except observation point S12 which was 0.070 ft. In percentage of the maximum settlement, rebounds varied from 11 to 20 per cent except for S12 which was 39 per cent. At the North tank site maximum observed settlements varied from 0.17 to 0.53 ft. while the rebound was remarkably constant varying from 0.054 to 0.060 ft. only, or from 10 to 34 per cent of the maximum settlement.

The piezometer readings were useful to show that primary consolidation within the compressible stratum occurred nearly as rapidly as the preload was added since very little excess pressure was observed. Only P-2 within the silty fine sand below the South tank showed a rational response to the preload filling. During each working day the water level rose as much as 2 ft. in response to the filling. As consolidation continued overnight, the piezometer level fell. Three Sundays when no fill was placed, are shown at about 9, 16 and 23 days. Piezometer P-2 rose a maximum of only 6.5 ft. which is slightly more than 10 per cent of the theoretical maximum rise of about 60 ft. The latter would occur in a saturated soil if no consolidation took place while the 27-foot preload fill was applied.

Piezometers below the South tank were preserved following preloading by extending the tubes below the tank to the boiler house where they were connected to mercury manometers. This was done so that pore water pressures could be observed while the tank was filled. Since the tanks can be filled in less than 24 hours, pore pressures could be considerably higher than during preloading.

Tank Settlement

Following preloading, final site preparation for the oil storage tanks was carried out. A reinforced concrete ring, 3 by 3 ft. in cross section, was provided below the tank shell to give additional stiffness. The welded steel tanks and safety rings were provided and erected by the Hammond Iron Works.

Twelve settlement observation points were established around the exterior of each tank. Numbers were again assigned clockwise with the twelve o'clock position at the north edge of each tank. One-inch steel cubes were welded to the tanks and elevations were made on steel pins inserted horizontally into the cubes. The maximum and minimum settlements for each tank are shown in Figure 7.

Each tank was test filled with sea water before it was used to store oil. Since foundation conditions were more favorable at the North tank site, the heavier No. 6 fuel oil was stored in this tank. Test filling with sea water therefore exceeded the maximum fuel oil load by approximately 7 per cent in the North tank and 20 per cent in the South tank.

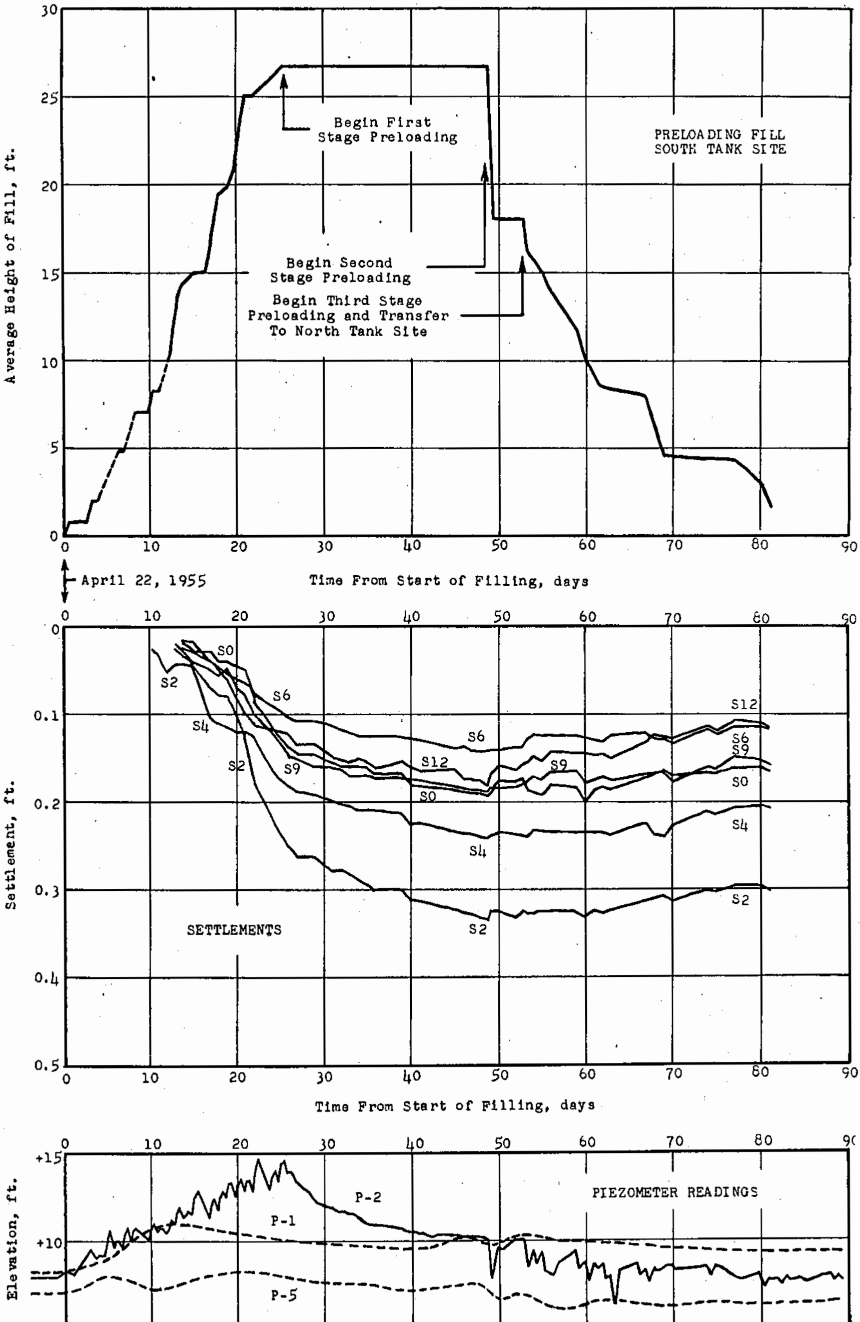


FIG. 6A.—SETTLEMENT OBSERVATIONS AND PIEZOMETER READINGS DURING SITE PRELOADING FOR SOUTH TANK.

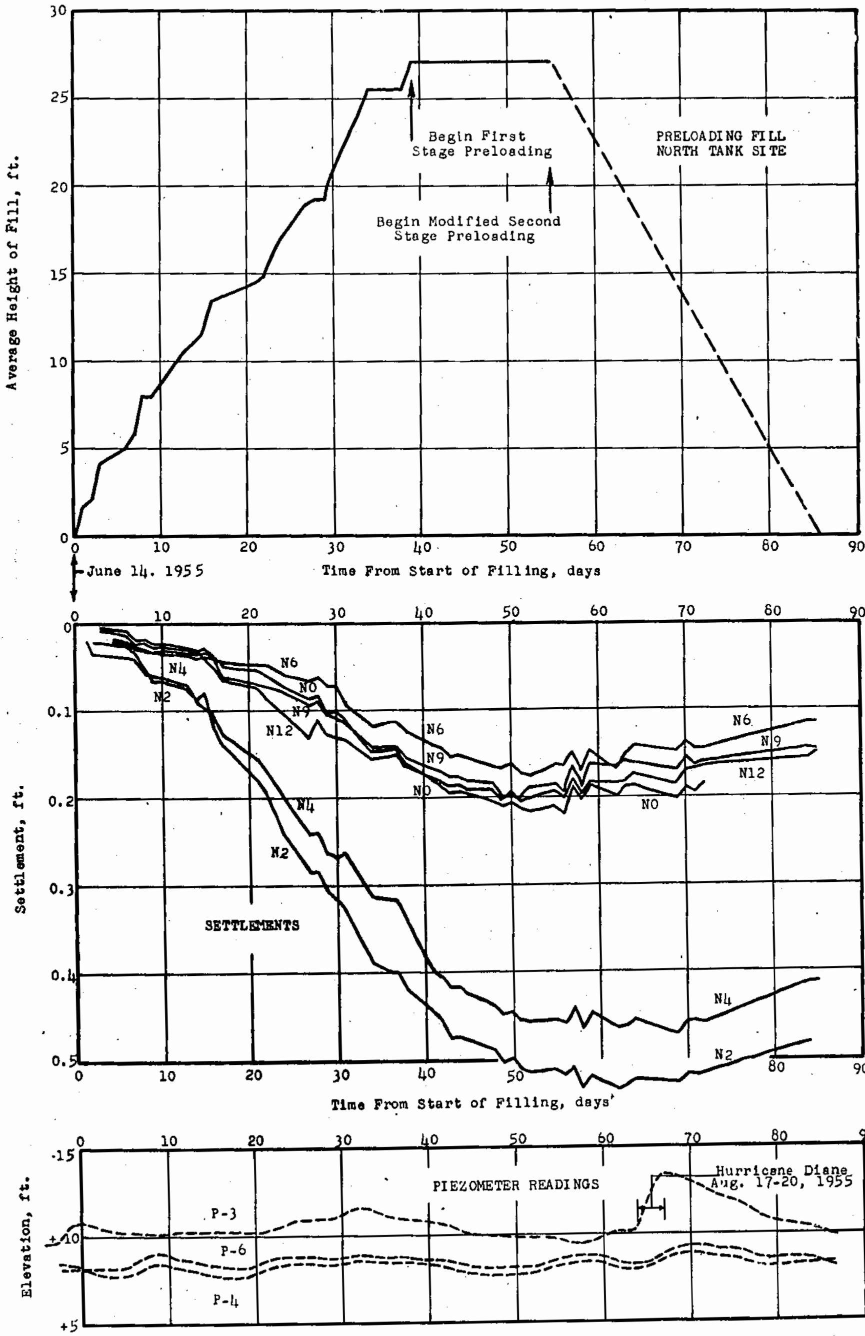


Fig. 6B.—Settlement Observations and Piezometer Reading During Site Preloading for North Tank.

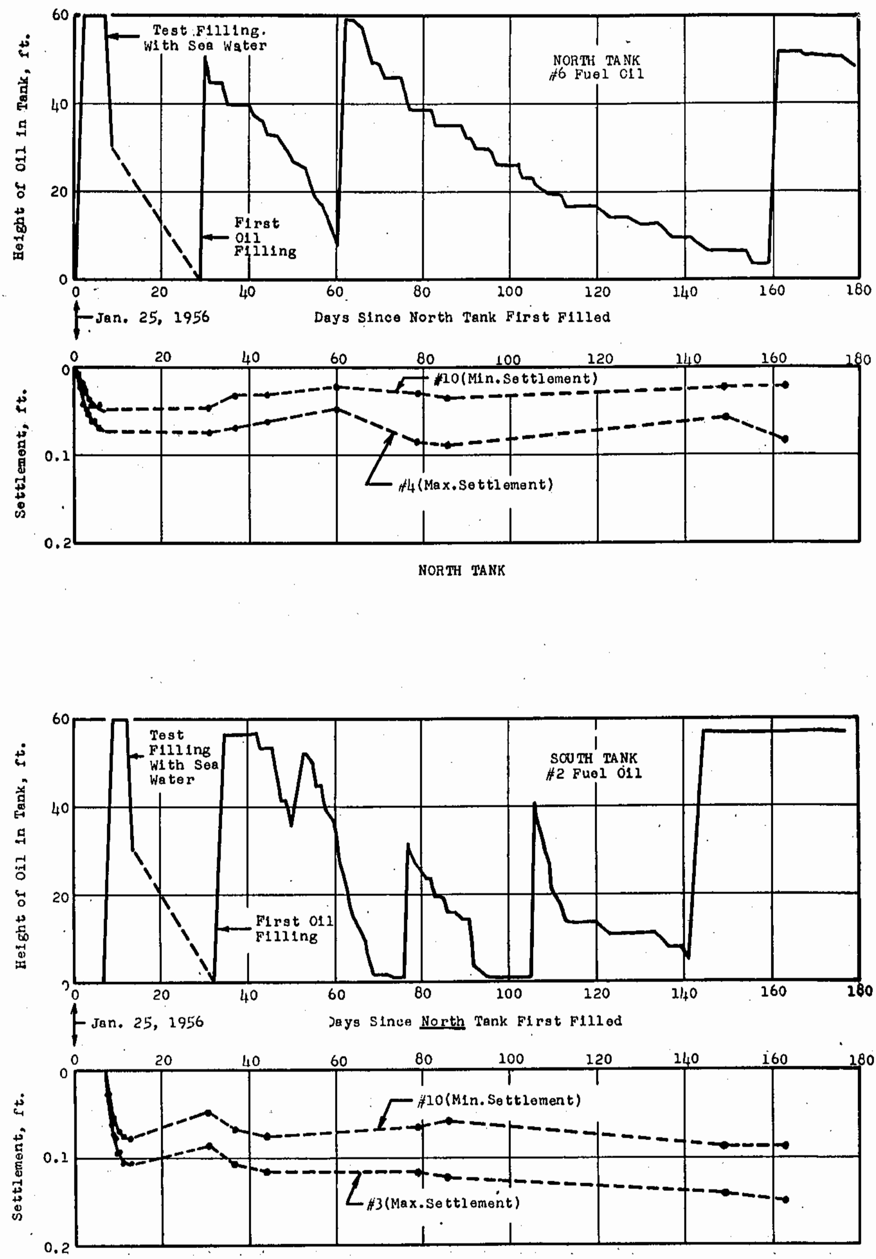


FIG. 7.—MAXIMUM AND MINIMUM OBSERVATIONS OF SETTLEMENT AT NORTH AND SOUTH TANKS.

Again as expected, the maximum observed settlement is occurring along the east edge, at the 4 o'clock and 3 o'clock positions at the North and South tanks, respectively. The minimum observed settlement occurs on the opposite side indicating that the tanks are tilting slightly to the east. As of July 7, 1956, the maximum observed settlement of the North tank was 16 per cent of the maximum preload settlement. At the South tank it was 45 per cent which reflects the effect of the organic silt found at this site.

The July 7th settlement observations for all points have been plotted in Figure 8. It can be shown that if the settlement of each observation point is plotted as shown, a uniform tilt without warping would be represented by points falling on a sine curve. I have

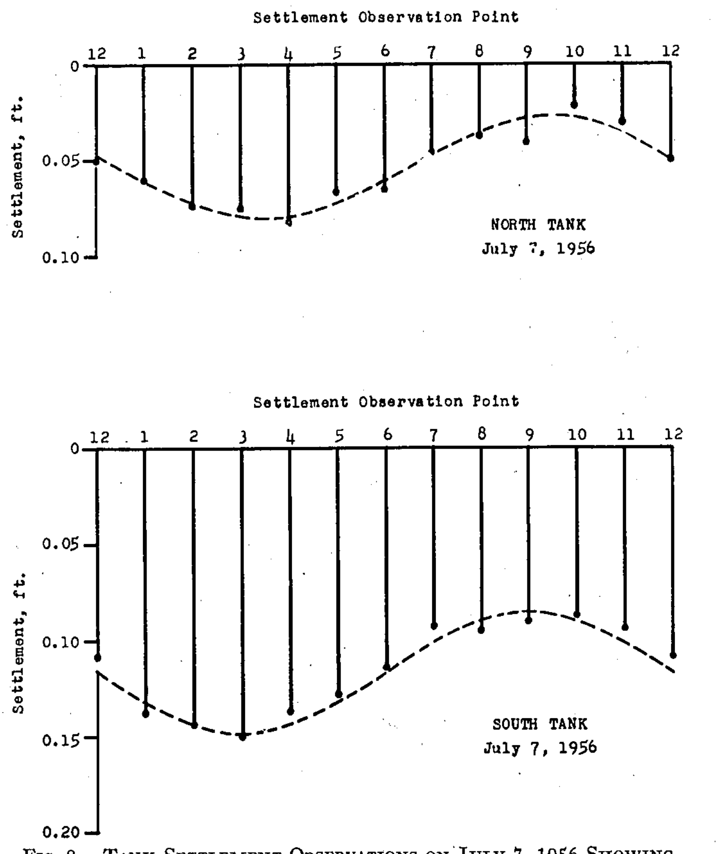
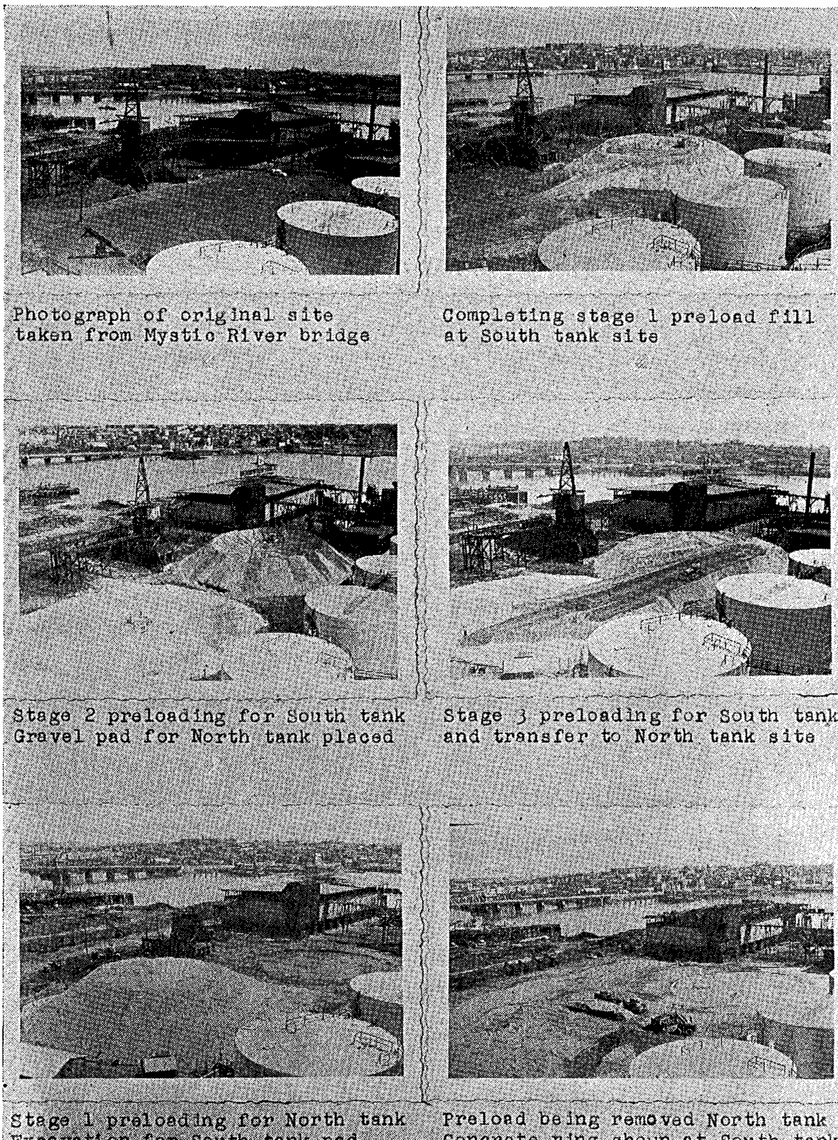


FIG. 8. TANK SETTLEMENT OBSERVATIONS ON JULY 7, 1956 SHOWING UNIFORMITY OF TILTING.



Photograph of original site
taken from Mystic River bridge

Completing stage 1 preload fill
at South tank site

Stage 2 preloading for South tank
Gravel pad for North tank placed

Stage 3 preloading for South tank
and transfer to North tank site

Stage 1 preloading for North tank
Excavation for South tank pad

Preload being removed North tank
Concrete ring shown at South tank

FIG. 9.

drawn the best sine curve through the plotted points. Only one point, No. 9 at the North tank, deviates more than 0.01 ft. or about one-eighth inch from the sine curve. This performance is indeed very satisfactory.

Settlement below the center of the tanks is undoubtedly greater than that observed below the ring. However, a tank can withstand considerable differential settlement in this direction. Future settlement is expected to be well within tolerable limits.

A final check on the performance of the South tank was made from mercury manometer readings for Piezometer P-2 when the tank was filled with sea water. These readings converted to piezometric levels indicated a rise of 14 ft. compared to a maximum of 6.5 ft. during preloading. Nevertheless, this still represents only 23 per cent of the theoretical maximum even though the tank was filled in 48 hours.

The total cost of site preloading and foundation preparation for the two tanks including piezometers, settlement platforms and concrete rings was about $93,000.

Summary

Site preloading is occasionally a practical and economical method of controlling foundation settlements. This is especially true under the following conditions:

1. When earth fill is needed at the site for other reasons such as grading for parking areas, etc., or when earth is readily available at or near the site.
2. When the compressible strata are inorganic and where the soil type and strata thickness are such that compression will occur rapidly.
3. When sufficient time is available for a thorough soil engineering study and for the preloading operation itself.

In the project which has been described, site preloading appears to have saved the owner more than $100,000 over the cost of a pile foundation.

Acknowledgments

I wish to acknowledge the splendid cooperation from Mr. Everett Hunt and the Metropolitan Coal Co. during the entire investigation. Many people assisted the author and their important contributions have been noted throughout the paper.

ANALOG AND DIGITAL COMPUTERS IN CIVIL ENGINEERING

By Saul Namyet*

(Presented at a joint meeting of the Hydraulics Section, B.S.C.E. and Massachusetts Section, A.S.C.E., held on November 19, 1957.)

Introduction

During the past 10 years there has been a rapid growth in the development of electronic computers, both digital and analog. The initial impetus to this growth was undoubtedly provided by the requirements of military research and development, and for that matter the continued rapid growth is currently being sustained by the same primary factor. Parallel with the developments in computer technology there has been an equally phenomenal growth in the use of electronic computers in research, engineering, business practices, process control, and data processing. The obvious reason for this is that we have been waiting for computers to come along and help us with our work. All of us, at one time or another, have faced problems that we would have liked to investigate, and solve if possible, which we have set aside because the methods available would require an impractical expenditure of time and money. Many of these problems can now be undertaken with the aid of computers.

The purpose of this discussion is to introduce you to analog and digital computers and to indicate some of the ways in which practicing civil engineers can make effective use of them. A few of your colleagues in each branch of Civil Engineering have begun to use computers; however, their use in Civil Engineering should be much more general.

Computers are now available in a variety of sizes, speeds, costs, and types. The two principal categories are called digital and analog (Figure 1). The digital computer deals with numbers and the analog computer is concerned with continuous variables. The output of a digital computer is generally a series of digits forming a number in some kind of arithmetic. In the analog computer the output is usu-

*Professor, Massachusetts Institute of Technology Computer Laboratory, Cambridge, Mass.

ally a curve representing the variation of a physical quantity to some scale. The conventional speedometer produces analogic information, but the odometer which records the distance travelled has a digital output.

All other considerations aside, the accuracy of a digital computer solution is limited only by the approximations in the mathematical method of analysis and the amount of machine-time that is devoted to the solution. In the analog computer the accuracy is only limited by the agreement between the analog and the actual problem.

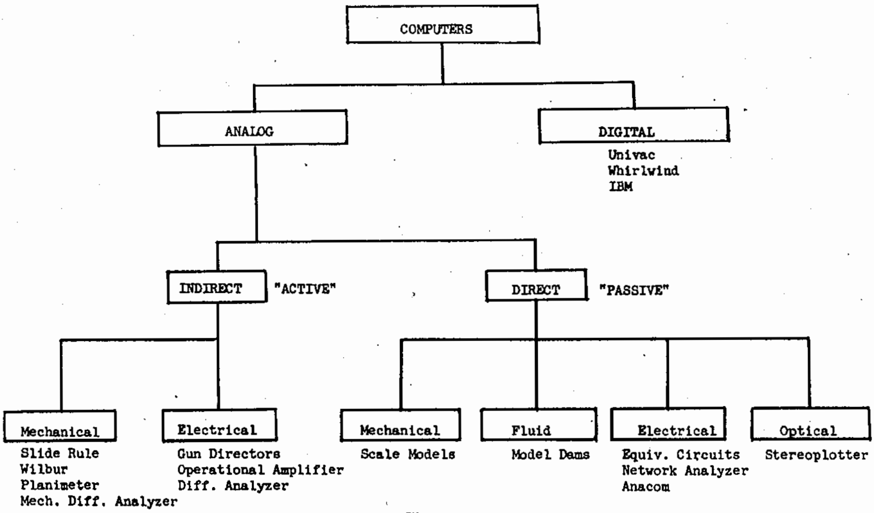


FIG. 1.

The precision of computers is a slightly different story. In a digital computer, increased precision is obtainable merely by increasing the number of digits that can be handled in one number. In general this results in a costlier machine. Precision in analog computation, on the other hand, is difficult to increase beyond about .01%. In general the cost increases rapidly for a small increase in precision.

WHAT IS AN ANALOG COMPUTER?

Analog computers may be divided into two broad classifications, the direct and indirect analog. Both classifications may be subdivided further: direct analogs into mechanical and electronic; and indirect analogs into four categories, mechanical, electronic, hydraulic, and optical.

The direct analogy is characterized by those cases where problem variables and parameters are represented directly by corresponding units on the machine. The mechanical direct analog computers are generally scale models such as are used in wind tunnels or in structural analysis. The electrical direct analogs are instruments such as the network analyzers and equivalent circuits exemplified by the Anacom computer which has found considerable use for static and dynamic applications in the aircraft industry. The civil engineer is well acquainted with the fluid analog in the form of model dams, harbors, and stream beds which are found in hydraulic laboratories. The last category, optical analogs, are probably familiar to only a small group of highway engineers and photogrammetrists in the form of the stereoplotter which is capable of producing an optical 3-dimensional model of any object that has been photographed by stereoscopic methods. Of these different types of analog the electrical analogs are the only general purpose computers in that they may be used to solve a variety of problems by turning knobs or by varying the interconnection of the components.

The indirect analog computers are capable of solving algebraic or differential equations. The most common example of a mechanical indirect analog computer is the slide rule. At the other end of the spectrum in size and complexity is the mechanical differential analyzer. The electronic indirect analog computer is probably the most popular analog computer today and one that the Civil Engineer should be acquainted with if he is to take maximum advantage of modern aids to computation.

Electronic Indirect Analog Computers

There are two categories of electronic indirect analog computers, real-time and suppressed-time. The difference between the two exists in the end results. The real-time computer produces solutions repetitively as fast as 1/3000th of real time. Thus it may handle problems which are being continuously modified to study the effect of a particular parameter.

The best known commercial example of a real-time computer is the REAC manufactured by the Reeves Instrument Company. Some representative repetitive computers are the BOEING by the Boeing Airplane Co., the GEDA by Goodyear Aircraft Co., the

GAP/R by Geo. Philbrick Researches, Inc. and the GPS computer by GPS Instrument Co., the latter two being Boston firms.

In the electronic indirect analog computer the voltage is considered to be the analog of the dependent variable with time as the independent variable. If we have a problem in which we wish to know the vibration of a mass on a spring, a curve of voltage vs. time would be obtained in which the voltage could be interpreted as displacement. In other types of problems, for which, for example, distance may be the independent variable, the time axis of the resulting voltage-time curve would represent distance.

The basic components of the computer which are of primary interest and utility are the Adder, the Coefficient Unit, and the Integrator. The Adder sums voltages algebraically (in some devices it is possible to introduce a finite gain to each input voltage before adding). The coefficient unit is capable of multiplying by an adjustable constant, usually between 0 and 1. The integrator is capable of integrating voltage with respect to time. The behavior of these units is illustrated in Figure 2. In its physical form each component is unidirectional, that is, information flows only from input to output; in addition, any component may instruct any number of others without correction. These three basic units are all that are required to simulate a linear system. However, there are available many types of non-linear components, and the computer manufacturers are always developing new special-purpose components. Some of the more common non-linear GAP/R components, for example, are known as bounding, backlash, inert zone, square root, squaring, and absolute value units. The behavior of each of these units may be derived from Figure 3 which shows the different output of each unit for a common input.

It is often necessary to represent functional relationships between two variables which exceed the capabilities of the basic components. For this purpose there are function fitters which can fit a curve by a linear segmented polygon with adjustable lengths and angles.

The most important non-linear component is the electronic multiplier which can multiply two varying voltage signals. A complete high speed analog computer includes auxiliary equipment such as a power supply, an initial condition signal generator, a timing signal to indicate the time scale of the output, a calibrating device to deter-

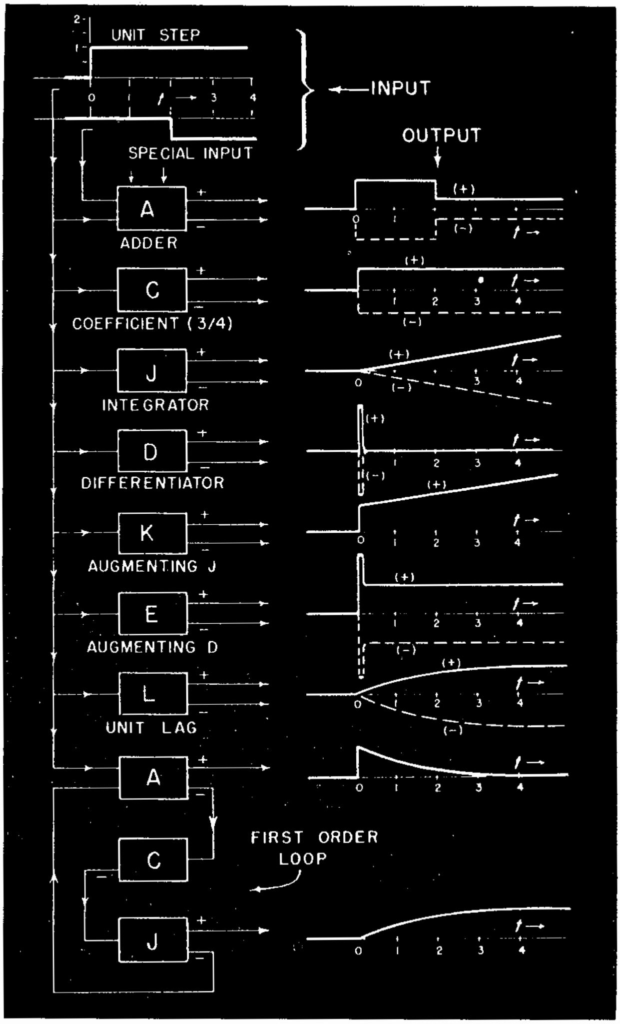


FIG. 2.—CHARACTERISTICS OF LINEAR ANALOG COMPUTER COMPONENTS.
(Courtesy of G. A. Philbrick Researches, Inc.)

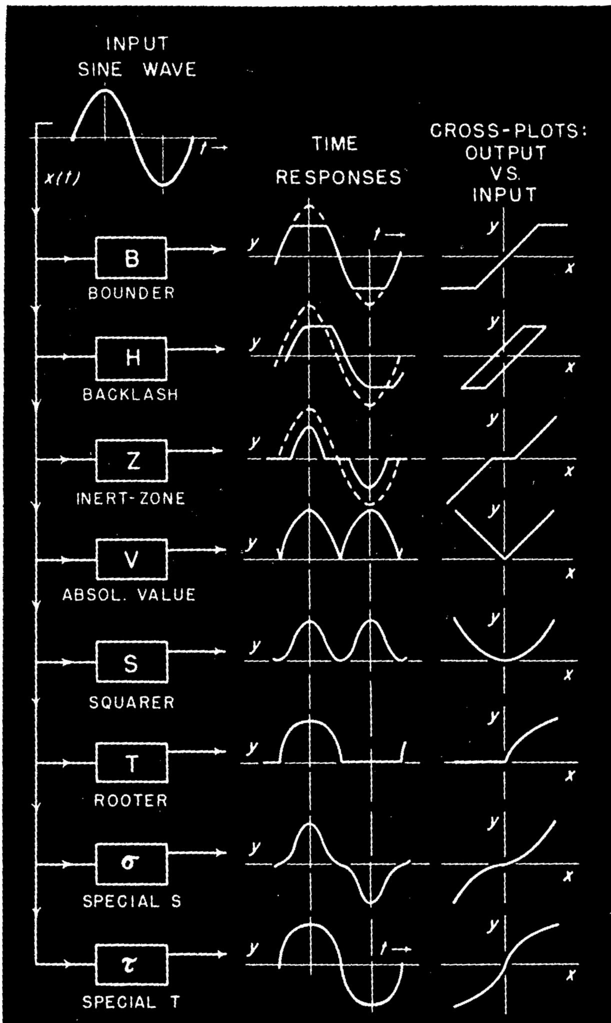


FIG. 3.—CHARACTERISTICS OF TYPICAL NON-LINEAR ANALOG COMPUTER COMPONENTS.
(Courtesy of G. A. Philbrick Researches, Inc.)

mine the output voltage scale, and finally, a display oscilloscope with the usual controls.

The description of the indirect electronic analog computer is best completed by a description of the method employed in setting up a simple problem involving a differential equation.

Consider the differential equation of motion for the system in Figure 4. The mass is subjected to a time-varying force f(t). The spring force varies directly with displacement so that the dynamic equations of motion may be expressed by

$$f(t) - kx = M\ddot{x}$$

The usual routine is to rewrite the equation so that the highest derivative is alone on the left so that

$$\ddot{x} = \frac{f(t)}{M} - \frac{kx}{M}$$

Then a block diagram is constructed by assuming that the voltage entering integrator A is $\ddot{x}$. The output must be $\pm\dot{x}$. If $+\dot{x}$ is introduced as input to another integrator B its output voltage will be $\pm x$. Putting the $-x$ voltage through the coefficient unit $\frac{k}{M}$ gives $\frac{-kx}{M}$.

In the other circuit, a constant voltage is introduced as input to a function fitter which produces as output the function f(t). This is multiplied by the coefficient $\frac{1}{M}$ to give $\frac{f(t)}{M}$. The voltages $\frac{f(t)}{M}$ and $\frac{-kx}{M}$ are combined in adder C to give $\ddot{x} = \frac{f(t)}{M} - \frac{kx}{M}$ which is what is assumed as input to integrator A so the loop can be closed. To operate, the input voltage is introduced at the function fitter. Each of the functions can then be read on an oscilloscope. This completes the set-up of the computer except for scaling the voltage and time to the variables of the equation, a subject which is properly left for future detailed study.

EXAMPLES OF APPLICATIONS FOR ELECTRONIC ANALOG COMPUTERS

Without going into any details, it seems desirable to run through a few examples of the type of problem that can be handled with the analog computer. Back in 1947 the M.I.T. Hydrodynamics Laboratory with the aid of an analog computer initiated an investigation

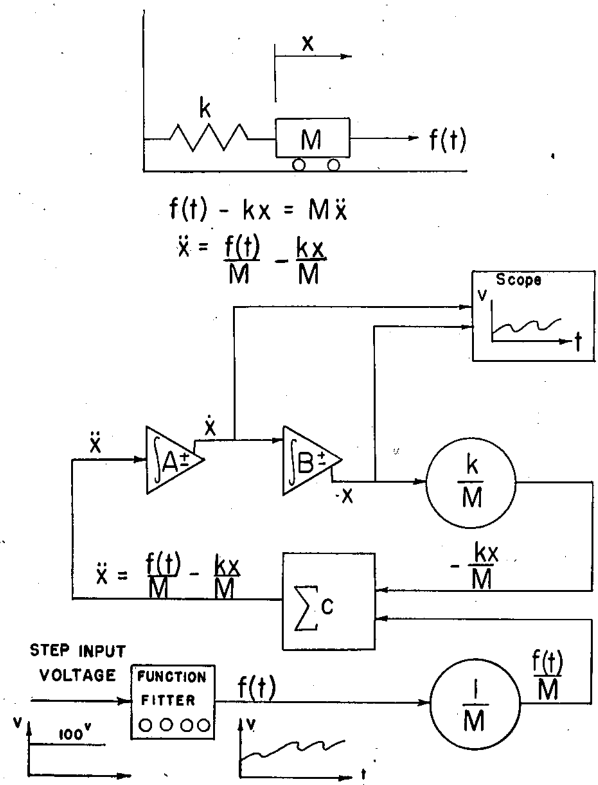


FIG. 4.

into the transient performance of power system prime movers. This led to a study of hydraulic surge and water hammer which is reported in two papers by Dr. H. M. Paynter (1, 2). A practical application by the hydraulic engineer involves the study of the dynamic routing of water flow in drainage basins. With an analog computer one can simulate a river system on a real-time or suppressed-time basis for purposes of flood prediction and water control. By varying the water storage conditions the most desirable operation can be devised. Dr. H. M. Paynter has reported on an investigation of this sort (2).

The sanitary engineer whose problem is designing complex sewage disposal systems should find that a similar approach to that used in water control studies will assist the design process immensely.

The transportation engineer trying to untangle the snarled traffic patterns of our cities should see many uses for analog computers and devices in design and control of traffic systems.

In the field of structures the analog computer finds a place in a long range investigation of bridge vibrations under the Joint Highway Research Program sponsored by M.I.T. and the Massachusetts Department of Public Works (3). The purpose of this investigation is to develop a method of predicting the magnitude and character of highway bridge vibration due to the passage of heavy vehicles. Field and model tests are run. The results of these tests are compared with a series of analog computer solutions. The problems performed on the analog computer are idealized and simplified mathematical representations of the complex system of moving load and bridge based partly on theory and partly on judgment. By varying the computer problem until the analog results agree with the test results a satisfactory mathematical relationship is obtained.

The soil mechanics engineer has used analog computers in problems involving consolidation, seepage, and thermal behavior (4). Professors Aldrich and Paynter of M.I.T. have recently devised an electronic computer incorporating commercially available components that is used to represent the behavior of a soil cross-section during freezing and thawing cycles. This computer is now in use at the Arctic Construction and Frost Effects Laboratory of the New England Division, Corps of Engineers, in Boston.

What Is a Digital Computer?

The modern automatic computer consists of four main elements and several possible subsidiary units depending on the size and complexity of the installation (Figure 5). The basic elements are the memory unit, control unit, arithmetic unit and input-output devices. These are the same as the basic elements of the conventional computation system utilizing desk calculators. The memory of the digital computer is analogous to the notebook of the numerical analyst, the arithmetic unit to his desk calculator. The control unit simulates the actions of the human as he manipulates the keys of the calculator and transfers data in and out of the calculator.

A memory unit consists of a large number of memory cells each identified by an address. Each of these in turn is subdivided into

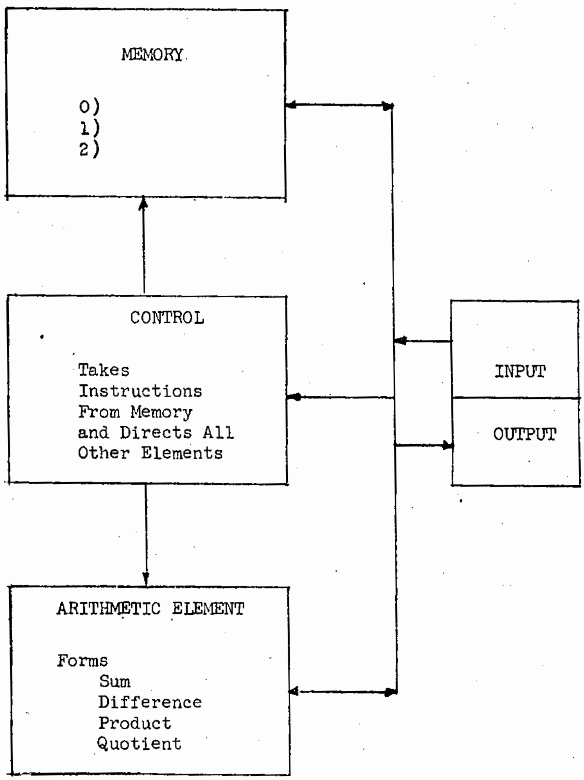


Figure 5 Automatic Digital Computation

memory elements which are the smallest memory subdivision. In most machines information is stored in these elements in the form of binary digits, zero or one. The most popular memory devices are magnetic cores, drums, or tapes, with cores or drums for the primary or high-speed memory and drums or tapes for the secondary or slow-speed memory.

The only important difference between the arithmetic unit of a digital computer and a conventional desk calculator is the extremely high speed of computation of the former. Otherwise a digital computer arithmetic unit operates on two numbers, adding, subtracting, multiplying, and dividing, just as in the desk calculator.

One of the more important aspects of large modern computers is that the control information is stored in the memory with the data that is being processed. The information for the control unit is stored in memory cells as instructions, consisting of two basic parts—one part indicating the operation desired, the other part indicating one or more addresses or memory locations involved in the operation. The control unit picks up the instructions according to a routine that is either automatic or programmed and interprets them by activating specific circuits, each instruction being a series of binary digits that determine the circuit to be selected.

The input-output system is comprised of devices which place numbers and instructions in the memory unit by punched tape, punched cards, or magnetic tape and obtains results from the memory for reproduction by automatic typewriter, punched tape, punched cards, or pictures of an oscilloscope screen. These devices convert from conventional language to machine language for input and back again from machine language to conventional forms for output.

It is almost universal practice to base the operation of digital computers on the binary system (base 2). This results from the fact that many electronic devices operate best when required to distinguish between the fewest possible number of different conditions, namely two. In the binary system each digit is either a zero or a one. Digits are coefficients of powers of 2 rather than powers of 10 as in the decimal system. Instead of a decimal point we speak of a binary point.

In decimal notation: 378.5 equals $3 \times 10^2 + 7 \times 10^1 + 8 \times 10^0 + 5 \times 10^{-1}$. The binary number 101001 equals 41.0 decimal. This can be shown by the expansion: $1 \times 2^5 + 0 \times 2^4 + 1 \times 2^3 + 0 \times 2^2 + 0 \times 2^1 + 1 \times 2^0 = 32 + 0 + 8 + 0 + 0 + 1 = 41.0$. Note that 6 binary places are approximately equal to only two decimal places.

The length of the binary numbers varies from machine to machine, anywhere from 16 to 50 *b*inary dig*its* (bits) being used in contemporary machines. The position of the binary point may be "floating" or "fixed". In "floating-point" arithmetic the binary number consists of two parts, one indicating the position of the binary point and the other containing the significant digits. This arrangement permits storage of both very large and very small numbers. In "fixed-point" arithmetic the binary point is considered to be fixed between certain bits which results in a much smaller range of numbers that the machine can accept. Some computers can handle both "fixed-point" and "floating-point" arithmetic. To overcome the limitations on the size of numbers in a fixed point machine it is possible to introduce scale factors which adjust the numbers to fit within the machine limits.

When a human operator solves a problem with a desk calculator he must start out with a set of instructions which specify how the computations are to be performed. In like manner the digital computer needs a program. Preparation of a program consists of two steps:

1. Planning the sequence of elementary steps.
2. Coding the sequence of steps.

Planning a solution generally may be accomplished without special knowledge of the particular computer that will be used, although a given problem may be solved more efficiently if planned for a specific computer. On the other hand, coding of a program must be for a specific computer because ordinarily each machine has its own code and understands no other.

To simplify our introduction to digital computers we might consider a small fictitious computer based in part on M.I.T.'s Whirlwind. Provision is made for an accumulator register (AC) which is a special storage place for intermediate results in a sequence of arithmetic operations or a place where a number is held preliminary to further operations. For our purposes this machine can handle any number, large or small. The coded program may contain decimal numbers as well as the coded instructions, both of which are converted to the binary equivalents by the machine after "read-in". This is a single address computer which means that each instruction contains only one address, which is generally the location of the number to be operated on. The control system considers the instructions in sequence unless control is transferred to another address. When the instructions are "read-in" they are stored in sequence in the memory.

Some of the basic instructions for such a computer which help indicate the type of operations that can be performed are tabulated below. To simplify the explanation of the instructions use is made of the notation c(x) to represent the word contained in register x, reading it as the "contents of x" where x is the address of a memory cell. An understanding of the usefulness of these few instructions can only be obtained by some illustrative problems for which there is no time in this presentation, or by the reader attempting to solve some simple problems of his own invention. The large machines have many more instructions which can only be considered meaningfully in terms of the particular machine.

BASIC INSTRUCTIONS FOR A DIGITAL COMPUTER

Instruction	*Explanation*
CA x	*C*LEAR AC and *A*DD c(x) to AC
CS x	*C*LEAR AC and *S*UBRACT c(x) from AC
AU x	*A*DD c(x) to c(AC) and store sum in AC
SU x	*S*UBTRACT c(x) from c(AC) and store difference in AC
AO x	*A*DD *O*NE to c(x) and store in x and AC
TS x	*T*RANSFER c(AC) to STORAGE register x
TD x	*T*RANSFER address *D*IGITS of c(AC) to address portion of register x
SP x	*S*KI*P* control to register x. This is an *unconditional* transfer of control.
CP x	*C*ONDITIONAL *P*ROGRAM control. If c(AC) is negative take next instruction from register x; if c(AC) is positive go on in sequence.
MR x	*M*ULIPLY c(AC) by c(x) and *R*OUND-OFF the product in AC to fit one memory cell.
DV x	*D*IVIDE c(AC) by c(x) storing quotient in AC

We have made considerable use of digital computers in the Civil Engineering Department at M.I.T. in recent years and have been very pleased with the results especially since many of our problems could not have been undertaken without the aid of computers. However, this has not been without the usual—and some unusual, difficulties. Computers have not reached the stage at which they do any thinking, no matter what the newspapers are saying. In fact, today's computer as yet can only do what you tell it to do. Indeed you must be extremely careful, for example, that you punctuate your information exactly as specified for the particular machine. The human operator of a calculator has a good chance of detecting a misplaced decimal point but a machine cannot.

One of the fundamental characteristics of the digital computer, its ability to modify the instructions as the computation proceeds, is a major source of difficulty in obtaining completely satisfactory programs. The ability to change instructions results oftentimes in changing the wrong instructions. However, there is considerable effort being devoted to the development of procedures for detecting and preventing mistakes as well as for automatic programming, all of which should help the user of digital computers.

It should be emphasized that the digital computer is only another aid, although a very powerful one, for solving engineering problems. In this respect the engineer should list the digital computer with the slide rule and the desk calculator. In fact, for many problems in engineering the other two devices are better suited than the digital computer for obtaining solutions. The primary utility of the digital computer lies in its ability to repeat a series of simple operations at fantastic speed. Thus digital computers are best suited for problems that require numerous repetitive solutions of the same equations or relationships. Some problems can be solved once by the use of desk calculator (some many more times) for a given set of conditions much more rapidly than they can be programmed to the point where answers can be obtained. An obvious advantage of the digital computer is that once a program for a problem has been completed satisfactorily, in general it requires no more calendar days to obtain 100 answers than one answer. In production computations, that is computations using an established program, an engineering office can save considerable time or, what is oftentimes more desirable, make a more thorough analysis than is customary because of the time required to obtain a solution by conventional desk calculators. If a problem is solved by conventional procedures and a change in basic data is introduced near the end of the process, a revised set of computations will ordinarily require the expenditure again of a large percentage of the man hours and calendar days required for the original calculations. However, with the digital computer, although the original programming might require considerable time, new sets of answers can be obtained overnight.

Examples of Applications for Digital Computers

What are some of the ways in which a Civil Engineer can use a digital computer? In general, the applications should have a repetitive nature, either considerable repetition of the same type of problem through the years, or repetition of a typical calculation within a particular problem. If a problem does not contain one or both elements of repetition it is probably not desirable to apply high speed digital computer techniques.

Two examples which embody both types of repetition come readily to mind from the newly adopted practice in a few state highway departments, notably California where all traverse and earthwork calculations are performed by digital computers.

The structural designer who handles rigid frames by approximate methods such as moment distribution, can use digital computers to obtain "exact" solutions by solving the simultaneous equations that result from writing the slope-deflection relationships. This is very simple to do; but more can be done by giving the machine a procedure for selecting member sizes. To further complicate things, the machine may be given a procedure for varying the properties of the elements of the frame so as to permit it to search for the most economical design.

It should be evident that any procedure that an engineering group can systematize for its own use is amenable to digital computer methods. In this regard it seems reasonable to expect that someone will soon have a program for determining all of the data that is required to prepare the construction drawings of a highway bridge. This of course would require a separate program for each type of bridge. On a steel stringer bridge, for example, some of the factors that the program would account for are: span, skew angle, width of roadway, width of walks, profile of road over, clearance requirements, profile of road under, spacing of strangers, load specifications, etc.

Highway structural engineers are designing rigid frame piers ad infinitum using routine tabulated procedures which the digital computer can follow. The machine procedure would have to be more complex than the office routine because the designer uses his judgment as a basis for neglecting certain design load conditions. Although some of these judgment factors can be provided in the program, one would expect the machine to produce a more complete design pob than the design engineer would if he had to make all the computations himself.

The traffic engineer needs the help of a digital computer to handle the vast amount of statistical information that he collects in highway use studies and accident records, to mention a few instances.

There is a whole class of problems which are best described as transportation problems. These problems are handled most efficiently by a mathematical procedure called linear programming. The practical transportation problem will generally benefit by the use of digital computers. An example of such a problem that might interest a traffic engineer would be to determine how to obtain the maximum flow of traffic through a given complex network of one-way and two-way streets.

In general, any of the problems which are amenable to analog computer solution can be handled by the digital computer. The essential difference is that the suppressed-time electronic analog permits rapid survey of the various parameters involved in a problem. On the other hand, the digital, like the real-time analog can only yield one answer at a time for a given set of data. In many instances it is good practice to use both analog and digital devices.

In the highway bridge vibration study (3) mentioned earlier we have an example of such effective utilization. The analog computer with its ability to quickly survey an extended range of the various parameters but with its relatively moderate accuracy was used to define the significant limits of the critical parameters. The digital computer was then used to determine the maximum bridge deflection for various combinations of the parameters within the critical range.

Before closing it is appropriate to consider briefly a current research activity which is tied directly to digital computers in the Photogrammetry Laboratory of M.I.T.

Location and Design of Highways by Machines

a. *General*

Digital computers have recently been introduced into the list of engineering devices and aids that find use in the highway engineer's domain. In more than a dozen state highway departments, the earthwork calculation problem has been attacked in various ways on several different computers. In most instances, excepting a few, however, the computers have been solving conventional problems using conventional techniques, with the sole difference that the high speed computer replaces either the planimeter or the desk calculator.

The outstanding exception is a procedure recently announced by the Ohio Department of Highways in which it is proposed that topographic data be collected from a Kelsh plotter for use in a digital computer.

It is our feeling that the full potential of computers will not be achieved except by approaches that depart from conventional procedures for data procurement and data utilization in favor of all-inclusive procedures which eliminate as many conventional steps in the engineering process as possible and obtain all the useful design data in one continuous automatic or semi-automatic operation. With this as a guiding principle the Photogrammetry Laboratory at M.I.T. is developing a system for locating and designing highways by the use of stereoplotters linked with electronic computers.

If we consider the over-all process of location and design of highways to consist of three steps, namely:

(1) Preliminary location using 5′ contour maps,

(2) Design location using 2′ contour maps, and

(3) Construction drawing preparation, the proposed system is expected to make step 2 a machine operation insofar as possible.

This step begins with aerial photography of ground strips approximately 2,000 feet in width along the tentative route. From these aerial photographs the usual photogrammetric maps may be prepared in stereoplotters. At this point however, the automatic system of obtaining data is introduced so that full advantage may be taken of the stereoplotter. From this point on the design and location calculations are expected to be performed entirely by a digital computer.

b. *Description of the Basic System*

The proposed system may be divided into two principal parts: first, the method of data procurement and second, the method of data processing. The first part is basically concerned with the photogrammetric plotter and the second part is essentially a digital computer programming problem.

The basic idea of the system which determines almost the entire process involves the establishment of a rectangular grid system relative to which the coordinate axes of the photogrammetric model can be located. From the photogrammetric model, at the nodes of the grid system, in a predetermined sequence the elevations of the existing terrain are automatically recorded onto a tape or punched card to form a digitalized model of the terrain. The tape may be punched paper or magnetic.

Having the digitalized model of the terrain, the second part of the system comes into play. A computer program is necessary that will perform the following tasks:

(1) Accept and respond to any legitimately defined highway alignment equation.
(2) Accept and respond to any definition of profile grade, or compute the profile grade according to specifications and terrain.
(3) Select the appropriate cross-section templet at any station from a review of the model and profile data, and the highway design specifications.
(4) Compute the limits of slopes that are required.
(5) Compute the amount of cut and fill between stations and cumulative cut and fill.

c. *Unconventionalities of the System*

It should be emphasized at the outset that the highway centerline is expected to be skewed to the coordinate axes of the model grid system and as a consequence of this it is expected that preliminary data will be obtained for skew cross-sections. This means that slope stake data obtained directly from the computer would be on skew sections.

Another important change in conventional practice that results from adhering rigidly to the basic concept of a rectangular grid system is the general use of plus stations instead of full stations. This is necessary if the centerline of the highway is skewed to the grid and only one set-up of the photogrammetric model is to be used.

These two deviations from conventional design data presentation practices, it is expected, would meet with great objections from the constructor, although plus stations are necessary to some degree in current practice and "skewed slope stakes" once installed should not look much different to the construction engineer or equipment operator than "square slope stakes", except that the spacing would not be at 100 feet. However, these unconventional procedures can be avoided by making the procedure slightly less automatic and producing "square slope stake" data from "skew slope stake" data by graphical methods. The final profile station and elevation data can be obtained by the computer in a separate run for full stations once the final line has been selected.

It is also conventional design practice to consider the three cross-sections at plus stations where the existing ground line intersects the proposed subgrade elevation at the outside of the base and the centerline. In the proposed system, to consider these sections would greatly complicate the procedure. The possible adverse effect of omitting these conventional cross-sections can be minimized by use of a smaller grid length than the conventional spacing of cross-sections.

d. *Advantages of This System*

There are more or less advantages inherent in such a system relative to conventional methods of location and design of highways, depending on whether photogrammetry is currently being used in any form. Assuming the best current practice, however, the proposed system is expected to have the following characteristics to recommend it:

Rapid Multiple Trial Alignment and Profile:

With the complete grid system model available on punched cards or tape the location engineer may try various alignments and profiles in sequence by changing the short alignment profile tape or the small deck of alignment profile punched cards. A group of trials may be planned in advance and handled continuously or alternatively; successive trials may be based on a study of the results of the previous runs. In the latter case the problem would probably be removed from the computer pending a decision on the next trial alignment profile in which case there would be a certain amount of lost time in getting "on" and "off" the computer. However multiple trial alignments are obtained at a minimum of personnel effort and time because the entire model is available at the press of a button and the relatively slow and expensive stereoplotter set-up need not be repeated.

Flexibility:

In the location stage it is reasonable and customary to base decisions on cross-section data obtained at 200 to 500 feet rather than 50 or 100 feet as needed for design purposes. It is a simple operation to vary the size of the rectangles of the grid system that are to be used in the computations. To have a simple operation the grid system will always be an integral number of basic grid units.

Thus, in the location problem a large grid may be used and when designing from the same model, the smaller grid may be used.

There are other reasons for varying the grid size so that the decision will probably be left to the engineer in each particular case.

It is only necessary that the basic grid on which the digitalized model is recorded be at the minimum size that may be used in the design phase.

e. *Concluding Remarks*

The preceding description of some of the basic elements of a semi-automatic procedure for performing the data collection and processing of a highway location and design system only touches the surface of the potential system that should include consideration of all the operations involved in the design and construction of highways, including land-taking. The purpose of discussing this research program in its very preliminary stages is to call your attention to the potential developments in this area of interest and perhaps lead you to propose innovations in the methods of obtaining, processing, and presenting engineering information for all kinds of projects.

One hesitates to conjecture about the general acceptance and use of computers by Civil Engineers. However, a short word on the subject is probably necessary to round out this presentation. There are many problems which cannot be solved properly without computers. These computer applications, we can be certain, will take place. But what about the type of problem that is being handled well enough by conventional methods? Why change accepted techniques for new and untried techniques? There are probably many engineering offices and many problems for which computers hold little promise. However, there is no question that for many other engineering groups, computers promise the ability to provide faster engineering service, more thorough engineering, more economical engineering, more accurate engineering, as well as a better environment for engineers who will then have an opportunity to THINK a little more about some of the problems they face instead of spending so much tedious time grinding out computations.

References and Bibliography

1. Paynter, H. M., Surge and Water Hammer Problems, A.S.C.E. Transactions Paper No. 2569, vol. 118, 1953.
2. Paynter, H. M., Methods and Results from M.I.T. Studies in Unsteady Flow, B.S.C.E. Journal, vol. 39, no. 2, April 1952.

3. Biggs, J. M., Suer, H. S., and J. M. Louw, Bridge Vibration—A Theoretical and Experimental Investigation of the Vibration of Simple-Span Highway Bridges, Progress Report No. 2, Joint Highway Research Project-Research Report No. 19, Massachusetts Institute of Technology Department of Civil and Sanitary Engineering and Commonwealth of Massachusetts Department of Public Works, Cambridge, Mass., July 1956.
4. Aldrich, H. P., Jr., and H. M. Paynter, Analytic Studies of Freezing and Thawing of Soils, First Interim Report for Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, Boston, Mass., June 1953. (Excerpts in A Palimpsest on the Electronic Analog Art—see no. 5.)
5. Paynter, H. M. ed., A Palimpsest on the Electronic Analog Art, George A. Philbrick Researches, Inc., Boston, Massachusetts, 1955.
6. Korn and Korn, Electronic Analog Computers, McGraw-Hill, 1952.
7. Soroka, W. W., Analog Methods in Computation and Simulation, McGraw-Hill, 1954.
8. Engineering Research Associates, Inc., High Speed Computing Devices, McGraw-Hill Co., New York, 1950 (Digital and Analog).
9. Hartree, Douglas R., Calculating Instruments and Machines, The University of Illinois Press, Urbana, Illinois, 1949.
10. Wilkes, Wheeler, and Gill, The Preparation of Programs for an Electronic Digital Computer, Addison-Wesley Press, Inc., Cambridge, Mass., 1951.
11. Journal of the Association for Computing Machinery.
12. Computers and Automation.
13. Denke, P. H., and I. V. Boldt, A General Digital Computer Program for Static Stress Analysis, Proceeding of Western Joint Computer Conference, p. 72, March 1955.
14. Livesley, R. K., The Application of An Electronic Digital Computer to Some Problems of Structural Analysis, The Structural Engineer, January 1956.
15. Livesley, R. K., Analysis of Rigid Frames by an Electronic Digital Computer, Engineering Vol. 187, p. 236, August 1953.
16. Institute of Transportation and Traffic Engineering, University of California, Berkeley, California, Analysis and Simulation of Vehicular Traffic Flow, Report No. 20.
17. Highway Research Board, Washington, D. C., Proceedings of Annual Meeting, 1956 (to be published).

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF THE MEETING

Boston Society of Civil Engineers

DECEMBER 19, 1956.—A Joint Meeting of the Boston Society of Civil Engineers and the Sanitary Section, BSCE was held this date at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by President John G. W. Thomas, at 7:00 P.M.

President Thomas stated that the minutes of the previous meeting held on November 19, 1956 would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes would be waived unless there was objection.

President Thomas announced the death of the following member:—

Luzerne S. Cowles, who was elected a member March 16, 1904 and who died December 3, 1956.

The Secretary announced the names of applicants for Membership in the BSCE, and that the following had been elected to membership on December 12, 1956.

Grade of Member—Addison P. Munroe, Santo S. Nicolosi, Anthony Sakakeeny, Clement D. Zawodniak.

President Thomas called upon Darrell A. Root, Chairman of the Sanitary Section to conduct any necessary business for that Section at this time.

President Thomas then introduced the speaker of the evening, Harold A. Thomas, Jr., Gordon McKay Professor of Civil and Sanitary Engineering, Harvard University, who gave a most interesting illustrated talk on "Civil and Sanitary Engineering trends in Europe and Latin America".

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

Forty members and guests attended the meeting.

The meeting adjourned at 9:05 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

DECEMBER 19, 1956.—A joint meeting was held with the parent Society at the United Community Services Building, 14 Somerset Street, Boston, Mass., at 7:00 P.M.

The meeting was called to order by President John G. W. Thomas. Following the business meeting of the BSCE, Chairman Root was asked to conduct any business of the Sanitary Section. Mr. Root made an announcement concerning the Waste Water Seminar and then called for the nomination of a Committee to recommend a slate of officers for the coming year at the annual meeting in March. John S. Bethel, Jr., E. W. Moore, and F. M. Cahaly were duly elected Nominating Committee.

Mr. Root turned the meeting over to President Thomas who introduced Professor Harold A. Thomas, Jr., as speaker of the evening. Professor Thomas gave a very interesting illustrated talk covering his visit to many places of interest to Sanitary Engineers in Europe and Brazil during 1956.

There were 40 members and guests in attendance. The meeting was adjourned at 8:25 P.M.

CLAIR N. SAWYER, *Clerk*

STRUCTURAL SECTION

DECEMBER 5, 1956.—A meeting of the Structural Section was held in the Society Rooms on this date. The meeting was called to order at 7:00 P.M., by Chairman A. L. Delaney who introduced the speaker, Mr. Howard Simpson, Association Professor of Structural Engineering, Massachusetts Institute of Technology.

Mr. Simpson's talk, "The New A.C.I. Code—Its Implications and Ramifications", covered the 1956 Building Code of the American Concrete Institute with respect to new provisions and changes from previous codes. The most important change appeared to be the inclusion of the ultimate strength method of design in the code. The effect of this method in reducing the size of members was outlined. The talk was illustrated by slides showing important specification requirements and curves indicating relative strengths obtained by ultimate strength and elastic design methods. At the conclusion of the talk, there was a general discussion of the code and the methods of design to be used.

Eighty-eight members and guests attended this meeting.

The meeting adjourned at 8:45 P.M.

Respectfully submitted,

RICHARD W. ALBRECHT, *Clerk*

JANUARY 9, 1957.—A meeting of the Structural Section was held in the Adams Room of the United Community Services Building on this date. The meeting was called to order at 7:30 P.M., by Chairman A. L. Delaney who introduced the speaker, Dr. Karl Terzaghi, Professor Emeritus of Civil Engineering, Harvard University.

Dr. Terzaghi spoke on the problems encountered in the design and construction of the Vermillion Dam across Mono Creek in California. This earth fill dam was built in a very rugged area where many different glacial deposits had built up a heterogeneous and inconsistent soil. As clay deposits suitable for an impermeable blanket were not available, the impermeable portion of the dam was constructed from material stripped from the surface which contained sufficient fine material to be usable. Leakage was originally estimated as approximately 15 c.f.s. but was held down to only 4.3 c.f.s.

The talk was illustrated by slides showing the soil conditions encountered and by color slides showing the site and the actual construction.

Ninety-seven members and guests attended this meeting.

The meeting adjourned at 9:00 P.M.

RICHARD W. ALBRECHT, *Clerk*

ADDITIONS

Members

Henry Adelman, 12 Sunhill Lane, Newton 59, Mass.

William E. Brooks, 9 Lantern Lane, Hingham, Mass.

Robert A. Carleo, 35 Fiske Street, Waltham, Mass.

David E. Coffe, 64 Hacker Rd., Lynn, Mass.

Frank J. Heger, 108 Thornton Road, Chestnut Hill 67, Mass.

Thomas J. Lambie, 84 Westgate, Cambridge 39, Mass.

Addison P. Munroe, 54 Cross Street, Belmont 78, Mass.

Santo S. Nicolosi, 212 Saratoga Street, Lawrence, Mass.

Howard F. Peckworth, 114 N. Washington Ave., Batavie, Illinois
Anthony Sakakeeny, 552 Massachusetts Ave., Cambridge 39, Mass.
Stiles F. Stevens, 51 Park Drive, Boston 15, Mass.
Joseph B. Taylor, H. E. Fletcher Co. P.O. Box 112, Lowell, Mass.
Roger H. Wood, 21 Winchester Place, Winchester, Mass.
Clement D. Zawdniak, 51 Washington Avenue, Torrington, Conn.

Juniors

Richard Gleeson, 5 Fuller Street, Everett, Mass.
John D. Goodrich, 386 Commonwealth Avenue, Boston 35, Mass.
Robert J. Smith, 376 Riverway, Boston 15, Mass.

Student

Alfred W. Hoadley, 25 DeWolf Street, Cambridge, Mass.

PROFESSIONAL SERVICES
AND
ADVERTISEMENTS

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

Volume 44 JANUARY, 1957 Number 1

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROFESSIONAL SERVICES

INDEX TO ADVERTISERS

Please mention the Journal when writing to Advertisers

COMMITTEES

1956-1957

NOMINATING COMMITTEE

Past Presidents (Members of the Committee)

Thomas R. Camp — John B. Wilbub — Emil A. Gramstorff

Allen J. Burdoin
Fozi M. Cahaly
Elliot F. Childs
(Term expires March, 1957)

Ariel A. Thomas
Alexander J. Bone
H. Lowell Crocker
(Term expires March, 1958)

SPECIAL COMMITTEES

PROGRAM

John G. W. Thomas, *Chairman, ex-officio*

William L. Hyland
Arthur Casagrande
Howard M. Turner
Charles H. Norris
George R. Rich

James Adams, Jr.
Fozi M. Cahaly
H. Lowell Crocker
Robert W. Moir
Darrell A. Root

Augustine L.Delaney
John Clarkeson
Joseph C. Lawler
Llewellyn T. Schofield

Elliot F. Childs, Ruth D. Terzaghi, Byron O. McCoy

PUBLICATION

Charles E. Knox, *Chairman*

Dean F. Coburn
Robert L. Meserve
Stephen E. Dore, Jr.

Clifford S. Mansfield
Darrell A. Root
Augustine L. Delaney

John Clarkeson
Joseph C. Lawler
Llewellyn T. Schofield

LIBRARY

George M. Reece, *Chairman*

Roger C. Collette
John J. Cusack
James L. Dallas

Donald P. Rand
Paul A. Dunkerley

James R. Cass, Jr.
Henry R. Wheeler

HOSPITALITY

David H. Hamilton, *Chairman*

Frank T. Smith, Jr. — Llewellyn L. Cross, Jr. — Logan T. Murdock

JOHN R. FREEMAN FUND COMMITTEE

Howard M. Turner, *Chairman*

William F. Uhl — Harvey B. Kinnison — Leslie J.Hooper

DESMOND FITZGERALD AWARD

Carroll A. Farwell, *Chairman*

Thomas R. Camp — John B.Babcock, 3rd

SANITARY SECTION AWARD

Thomas R. Camp, *Chairman*

Kenneth F. Knowlton — Frank L. Flood

STRUCTURAL SECTION AWARD

Carroll A. Farwell, *Chairman*

Kentaro Tsutsumi — Emil A. Gramstorff

HYDRAULIC SECTION AWARD

Thomas R. Camp, *Chairman*

Donald R. F. Harleman — Harold A. Thomas, Jr.

SURVEYING & MAPPING SECTION AWARD

John B. Babcock, 3rd

Lorrin M. Pittendreigh — Hyman B. Ullian

TRANSPORTATION SECTION AWARD

John B. Babcock, 3rd, *Chairman*

John D. M. Luttman-Johnson — George W. Hankinson

CONSTRUCTION SECTION AWARD

Carroll A. Farwell, *Chairman*

Albert G. H. Dietz — Lewis E. Moore

INVESTMENT COMMITTEE

Charles O. Baird, Jr., *Chairman*

John G. W. Thomas — Arthur Casagrande

MEMBERSHIP CENTRAL COMMITTEE

Joseph C. Lawler, *Chairman*

James L. Dallas
Frank L. Lincoln
James F. Brittain
Chester A. Moore

Stephen Haseltine, Jr.
Edward W. Moore
George W. Hankinson
Percival S. Rice

John B. Biggs
Elliot F. Childs
Robert W. Moir

AUDITING COMMITTEE

Thomas A. Berrigan — George G. Bogren

ADVERTISING COMMITTEE

Ralph M. Soule, *Chairman*

Charles E. Knox
Ferdinand A.Trautner

C. Hugo Bergman
Herbert E. Fletcher, Jr.

Ralph S. Archibald
Robert F. Kelsey

JOINT LEGISLATIVE COMMITTEE

Edward Wright, *Chairman*

James F. Brittain — Thomas A. Berrigan

PUBLIC RELATIONS COMMITTEE

Thomas C. Coleman, *Chairman*

John H. Hession
Joseph W. Lavin

Frank L. Heaney
Samuel J. Tomasello

Thomas J. Rouner
George G. Hyland