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Volume 44 APRIL, 1957 Number 2

THE PROPRIETORS OF THE LOCKS AND CANALS ON MERRIMACK RIVER

Presidential Address by JOHN G. W. THOMAS*

Boston Society of Civil Engineers, March 20, 1957.

THE By-Laws of the Boston Society of Civil Engineers state that at the annual meeting the president shall deliver an address. No bounds are set on the subject matter. This leaves me in a very favorable position, with you, a captive audience, completely at my mercy. I can only hope that you will find something of interest and value in my remarks.

For many years the Proprietors of the Locks and Canals on Merrimack River was closely connected with the Boston Society of Civil Engineers, with hydraulic engineering, and with New England industry. Many of you probably have never heard of the company and may only think of Lowell as a city that used to have a textile industry, but at its zenith it was one of the marvels of the day. It was shown with pride as the epitome of what a industrial city should be, the Venice of America, with its "mile of mills" along the river, its boarding houses for the operatives, and its churches and Public buildings. The Locks and Canals supplied the water for all the power used and was the reason for the development. Almost every engineer of the Locks and Canals has been intimately identified with the Boston Society of Civil Engineers as founder, president, or active member of its Board of Government and Committees. Changing times have broken or changed these relationships and it seemed to me to be fitting to record here some of the history of the Locks and Canals and note the effects wrought by our dynamic economy.

*Project Engineer, Metcalf & Eddy, Engineers, Boston.

The charter of the Proprietors of the Locks and Canals on Merrimack River was granted by the Massachusetts Legislature in 1792, which makes it one of the older corporations that is still in operation. The time was before the start of the industrial revolution in New England and the company's name indicated its original purpose, which was to convey lumber and other goods around the rapids in the Merrimack River at Lowell. Much of the lumber for the thriving ship building industry in Newburyport was obtained from New Hampshire and transported in the only possible way at that time, by water down the Merrimack River, and it was Newburyport capital that financed this first canal.

The original works consisted of a low dam across the Merrimack River above the head of the rapids called Pawtucket Falls and a canal, 1¾ miles long, excavated from above the dam, across country to the Concord River just above its confluence with the Merrimack. The difference of elevation at that time was a little over 30 feet and there were four sets of locks to pass the boats and rafts from one level to the other. The legislature set the rates of toll and it is interesting to see all the products listed from nails and shingles, through staves and hoops for barrels and hogsheads, to boat loads of manure.

The transportation canal was a moderately successful venture, and made money for its backers, for a limited time only. As the communities further up the river grew and traffic increased, a larger proportion of the business and the travelers wished to make Boston the terminus of their trip. The Middlesex Canal, running from Charlestown to the Merrimack River just above Lowell, was conceived in 1793 to satisfy this demand and eliminate the long trip from Newburyport to Boston around Cape Ann. The Middlesex Canal was completed and started operation in 1804. Over the next few years the traffic through the Pawtucket Canal of the Proprietors of the Locks and Canals decreased and the owners probably began to wish that they had chosen some other form of investment.

In 1813 a cotton mill was built in Waltham. It is interesting to note that this mill was probably the first one in the world that combined all the operations necessary for converting the raw cotton into finished cloth. The earlier mills in this country, Slater's for instance, in Rhode Island, were spinning mills only, and in England, though the power loom had been introduced, it was used in separate mills by people that bought their yarn from the spinners, as had always been done by the hand weavers. Mr. Francis C. Lowell and his brother-in-law, Patrick Tracy Jackson, were instrumental in establishing the mill in Waltham and as it became successful and expanded to use all the limited capabilities for water power on the Charles River at Waltham, Jackson began looking around for some locality where additional water power was available.

He eventually heard of the falls on the Merrimack River and learned of the rights belonging to the Proprietors of the Locks and Canals. The stock in the Pawtucket Canal was obtained with comparative ease since the Middlesex Canal was offering such strong competition. So the canal company was purchased together with the farms and land in Chelmsford which later became the major portion of the city of Lowell. On February 5, 1882 the Merrimack Manufacturing Company was incorporated and the shares of the Locks and Canals were conveyed to the directors, in trust.

The enlargement of the original Pawtucket Canal, probably only 30 ft. wide and 4 ft. deep, was the first and most important step to be accomplished. It was decided to make it 60 ft. wide and 8 ft. deep which, it was estimated, would furnish 50 mill powers. The work began in the spring of 1882, but could not be completed until the summer of the following year. An additional canal was also dug to the Merrimack River at the site of the mills of the Merrimack Manufacturing Co. where the full 30 ft. fall could be utilized, because Mr. Moody said he had a fancy for large wheels. As all the original installations in the mills used breast wheels, this would mean 60 ft. diameter wheels at this location. The first wheel of the Merrimack Company was set in motion on September 1, 1823 to begin the many changes that took place in subsequent years.

In 1825 the Proprietors of the Locks and Canals was reorganized by selling them all the land and water power not required by the Merrimack Manufacturing Co. The Locks and Canals proceeded to construct the necessary new canals to develop the water power and sell the land to various new manufacturing companies which were formed to take advantage of the power available. The development was made on two levels with the fall between the upper and lower levels being indentured at 13 ft. and that between the lower level and the river at 17 ft. Between 1822 and 1848 the length of canals was increased from 1¾ miles to 5 miles and the use changed from transportation to power.

The water was sold to the mills by the Locks and Canals by the mill power. According to Nathan Appleton this standard came from Waltham. He says in his "Introduction of the Power Loom and Origin of Lowell," "The second mill built at Waltham contained 3,584 spindles, spinning No. 14 yarn, with all the apparatus necessary to convert cotton into cloth. This was taken as the standard for what was called a mill power, or the right to draw 25 cubic feet per second, on a fall of thirty feet, equal, according to Mr. Francis, to about 60 horse powers." This form of measurement is still used by the Locks and Canals, the only change being a division into Permanent and Surplus mill powers in 1853. The mills own the water wheels and convert the water bought from the Locks into power. Their investment in generating equipment may have contributed to the status quo in later years, as any redevelopment would have made such equipment obsolete. Such a division also sets the Locks apart from most power companies as it has never generated any power itself. According to Mr. Safford they have served merely as a "water boy for the mills."

The first engineer of the Locks and Canals was Major George W. Whistler. There is very little information about when he started in Lowell, but we know that he resigned to go to Russia and build railroads in 1837, and died over there. His son, the artist, is, of course, more famous for the well-known portrait of his mother. An old memoir of Patrick Tracy Jackson by John A. Lowell states that in 1830 Mr. Jackson began to think about constructing a Boston & Lowell Railroad, which was finally opened in 1835. There is no direct mention of Major Whistler, but reading between the lines, it seems probable that he was brought to Lowell in connection with the building of the railroad since his experience had been in that field rather than in connection with the water power.

Major Whistler had an assistant, from 1834 to 1837, a man who later became one of the best known early hydraulic engineers, James B. Francis. At the age of 22, Francis was appointed chief engineer of the Proprietors of the Locks and Canals, after the resignation of Major Whistler. He held this position for 48 years. One of his great works was the Northern Canal, which provided additional capacity for supplying water and power for the mills. He not only designed and built the canal, with its great granite river wall, but he designed the machinery for lifting the ten headgates and the water wheel which was used to operate them. The water wheel is probably the only surviving example of an early Francis Type turbine in existence. It has been used for operating the head gates until about two years ago when electric motors were installed for remote control.

Francis also built the guard gate at the head of the original Pawtucket Canal. At the time, it was believed by most people to be entirely unnecessary and was nicknamed Francis' Folly. But in April 1852, only two years after it was hung, a freshet occurred which required the gate to be dropped to prevent the Merrimack River from flowing through the city of Lowell. The gate was dropped again to save the city in 1936. Even in this great flood, the water overtopped Francis' gate by only about a foot, and sandbags were able to increase the effective height by this small amount.

An important part of Farncis' duty at Lowell was to distribute the water among the various mills in accordance with their respective rights. In order to be certain that the division was equitable, he performed many original experiments on hydraulic motors and the flow of water over weirs and in short canals. These and later experiments, which included diverging tubes, were published as "Lowell Hydraulic Experiments" and formed one of the earliest works on hydraulics.

James B. Francis was one of the founders of the Boston Society of Civil Engineers and its president in 1874. He was later elected an honorary member.

Uriah A. Boyden was Francis' assistant in Lowell. His name is usually associated with the outward flow turbine which bears his name. While the turbine itself was not new, his diffuser which applied the principles of the expanding tube to slow down the velocity of the water leaving the turbine and thus increase the power produced, was a very important development. It was the forerunner of the modern draft tube which is an essential part of all reaction turbine installations. Boyden wheels gradually replaced the original breast wheels in the Lowell mills.

Uriah A. Boyden was also a founder and member of the Boston Society of Civil Engineers.

The mills in Lowell were now growing rapidly. More buildings were being built, more machinery installed and more power consumed. Mr. Francis found that many, if not all, of the mills were using more water than they were entitled to under their water power rights. In 1853, after detailed study by Mr. Francis, all the agreements with the mills were renegotiated. It was at this time that the water was divided into Permanent and Surplus Mill Powers. The Permanent Mill Powers, in the same number as the original mill powers, were redeeded to the mills for $300 per mill power per year "forever." Any available water in the river above what was needed to satisfy the Permanent Mill Power use, was to belong to the Locks and Canals to be used by them for their needs, or distributed equitably between the mills for a fair price. The annual payment for the Permanent water was fixed at a point which would cover the operating expenses of the Locks and Canals, and amounts received from the sale of Surplus Water were to be used for extension and improvement of the facilities for utilizing the water power.

In 1885 when Mr. Francis retired, Hiram F. Mills, a member of the Boston Society of Civil Engineers, who was chief engineer of the Essex Company in Lawrence, was appointed engineer of the Locks and Canals also. During his tenure a great deal of work was done to improve the hydraulics of the canals in Lowell to increase their capacities. Before his retirement in 1916, a start was made toward electrifying the Lowell mills.

Arthur T. Safford was appointed chief engineer of the Locks and Canals in 1916 and held that position until 1940. He was also a member of the Boston Society of Civil Engineers and president in 1934-35. He was elected an honorary member in 1944.

During Mr. Safford's time at the Locks and Canals, the remaining mills were electrified and some of them installed new vertical water wheel units after World War I. The first part of this period was still one of expansion and growth, although probably at a slower rate than in the early years, because by this time the Merrimack River at Lowell was pretty fully developed. But by 1927 the cotton mills were beginning to feel the effects of southern competition and soon began to go out of business. The depression of the 1930's was very serious to Lowell and many more mills closed their doors or moved south.

The Locks and Canals, in spite of fixed permanent power rentals, had operated very satisfactorily with 1853 rates until the depression. While the "Mill Day" had originally been twelve hours, later reduced to ten and then eight hours, it was not until the mid-thirties that more than one shift was operated. To the Locks and

Canals, this meant two and three shifts of operating personnel, because the method of operating the canals had not changed since they were first used for power.

Sales of "surplus water" had been the means, for a good many years, of financing the Locks and Canals operations and it was only the low wage rates through the thirtys that offset the declining sales of surplus as mills closed down or curtailed production. When World War II started, a great impetus was given to manufacturing. The remaining textile mills, now including wool and synthetics, received large government orders and many new small industries started operations in the vacant mill buildings.

It was in 1940 that S. Stanley Kent, a member and vice-president of the Boston Society of Civil Engineers, was made chief engineer. He had been assistant engineer under Mr. Safford for 21 years and had contributed much in those years without receiving any credit. He was a member of the 1927 and 1936 flood committees of the Boston Society and practically the entire credit for the concept of the unit hydrograph is due to his work. Unfortunately, Mr. Kent died in 1943 before having an opportunity to receive full acknowledgment of his abilities.

The writer was appointed chief engineer in 1943, after serving two years as Mr. Kent's assistant, and continued in that position until 1956.

During the remaining war years, and post-war years when industry had a large pent-up demand, the Locks and Canals continued to be able, on the average, to make both ends meet. But it was evident, as the spiral of increasing wages and material costs set in, that it would be impossible to continue operating without taking steps to counteract the fact that a large proportion of income was based on a dollar that was nearly a hundred years old. Rates for "surplus water" could be raised, but by this time other forms of power were available and the local public utility furnished very effective competition to prevent unduly high costs for any portion of power used by the mills. It was this situation which resulted in our study and application of remote control of gates and automatic control of canal level by syphon spillways, which was first described in my paper "New Controls for Old Canals" published in the July 1950 Journal. Additional remote control installations allowed the Locks and Canals to reduce their operating personnel for the canal system from fifteen in 1946 to five in 1956.

The past three or four years, however, have seen textiles in New England experiencing poor business again and three of the remaining mills in Lowell moved or liquidated. To a certain extent new enterprises have taken the place of the cotton business and supplied many new jobs in place of the old ones. From the point of view of the Locks and Canals, however, the new industries which have moved into the city are of little value because they are light manufacturing and do not raise the load high enough to require the use of "surplus water."

As an example, one mill which used to have a load of about 4,500 kw. runs a maximum of about 1,500 kw., even though its space is filled and there are very likely more employees working in the buildings. Another which ran as high as 3,000 kw. during the war, had a 400 kw. load last year, with only part of the buildings being used.

Over a period of a good many years, studies have been made for the redevelopment of the water power of the Locks and Canals in a single modern station. Nothing has ever come of these plans for a number of reasons. First, the cost would be very large with too little savings to make it appear worthwhile. Second, the canals are used to supply process and condenser water to the mills. Even if a central steam plant were built, process water would still be needed, and additional cost would be involved to lay pipes for that service. Third, waste water from processes and toilets is discharged into some of the canals, so that provision would also have to be made to conduct these wastes to the river. Fourth, the existing water wheels, boilers, steam turbines and generators belonging to the mills would have to be scrapped. Lastly, to be effective, all waterpower users would have to agree to the necessary changes.

The question has been asked, "Why wasn't the rate for Permament Powers raised?" It might have been done before the depression, when all water users were also stockholders in the Locks and Canals, if the legal problems could have been solved. Apparently no one fully appreciated, not only how much the dollar had already depreciated since 1853, but more particularly, how much more it was going to depreciate in the future. During the depression and after World War II, many of the waterpower rights became separated from the stock in the Locks and Canals. This would make the renegotiation of the leases very much more difficult. A mill having only waterpower rights would naturally want to get its power as cheaply as possible and might not be willing to voluntarily increase their costs. On the other hand, those who are only stockholders would undoubtedly like to see income increased so that dividends might be paid. The complexities of standby power and the small amount of firm power available, make the possibility of increased total income slight and to date, no move has been made to increase Permanent Power rentals.

What is the future of the Proprietors of the Locks and Canals? We have seen them fulfill a demand for power and spark the founding and growth of the textile city of Lowell, which was proudly displayed to visiting celebrities of this country and Europe. We have seen them survive the depression and two wars. But now the newer industries do not want to own and operate their own power plants to supplement the water. The "Permanent Mill Powers" are available to the lessees for only 15 hours per day, and yet, we all want electricity available 24 hours a day at the flick of a switch. Today, in contrast to the early 1800's, the amount of power available from the Merrimack is very small in relation the loads which have developed. If additional capacity is needed in a Public Utility system, a single large unit of perhaps 140,000 or 150,000 kw. is installed. Compare this with the firm power available from the Merrimack of less than 4,000 kw. and the maximum development, which at one time reached nearly 20,000 kw.

So it would appear that the contribution of the Proprietors of the Locks and Canals on Merrimack River and its Engineers, while most important in the past, has seen its peak. The company may go on supplying what power it can to the waterpower owners for some years to come, but an era is past, and it seems as though a short history of the accomplishments and the men of those days should be in the records of this Society with which they were so closely associated.

REHABILITATION OF WHARVES AND PIERS ARMY BASE, SOUTH BOSTON, MASS. — GENERAL

By Robert J. Basso*

(Presented at a joint meeting of the Boston Society of Civil Engineers and the Structural Section, B.S.C.E., held on January 23, 1957.)

In order to present a clear picture and understanding of the work now under construction of the Boston Army Base, it is advisable to give some of the leading facts of the matter and the background leading up to the present construction.

Original Construction

Construction of the Army Base was completed during the latter part of 1918, on what was originally tidal flats. The construction, which cost in the vicinity of $22,000,000 at that time, included a 2-story steel frame Wharf Shed 135 feet wide by 1,638 feet long, 2-3-story reinforced concrete frame pier sheds 100 feet wide by 924 feet long, an 8-story reinforced concrete storehouse 126 feet wide by 1,638 feet long, an Administration building, Boiler House, Substation and several other smaller buildings, and included wharves and piers for docking ships in 10 berths.

Prior to the 1917-1918 construction, a portion of the Army Base property was filled representing approximately one half the present area. During the 1917-1918 construction, a finger of solid fill extending out into the harbor and retained by a concrete sheet pile bulkhead was built. This latter area is that on which the pier sheds now stand.

The 8-story Storehouse, the Administration Building, Boiler House, Substation and the several other smaller buildings were founded on concrete caissons or concrete piles. The Wharf Shed, the easterly end of the Pier Sheds, and the wharf and pier aprons which form the entire perimeter of the Army Base are supported by untreated timber piles driven into soft and medium blue clay, with the ground line at the piles varying from low water to about 30 feet below low

*Project Engineer, Corps of Engineers, U. S. Army, New England Division.

water. The range of tide at the Army Base is about 9.5 feet. The elevation of the wharf and pier aprons is 18 feet above low water.

Wharf and Pier Structures

The wharf and pier structures have offered the greatest problems over the period of years. The structural components of these structures consist of very lightly reinforced pile cap walls extending transversely across the structures and supported on rows of the timber piles. On these walls are carried the heavily reinforced longitudinal crane and railroad track beams and the deck slab which spans from wall to wall. The purpose of the pile cap walls was to permit cutting off the timber piles at an elevation below high tide to keep them permanently wet, with the walls acting as a continuation to support the deck structures.

Damages to Wood Piles

At the time the wood piles were driven, marine borer activity in Boston Harbor had been at a very low level for a number of years. However, in 1925, the crustacean borer, Limnoria, again became active in considerable numbers. By the early 1930's, the wood piles had been damaged to a great extent. The severity of the attack increased steadily and a detailed inspection in 1933 revealed that the average cross-sectional area at the low water line was about 66% of the original area of the piles, and it became necessary to make repairs and prevent further damage to the piles by the marine borers.

Repairing Damaged Piles and New Protection Installed

As a result of a study made at that time, it was determined to surround the entire perimeter of the structures supported on wood piles by a steel sheet pile bulkhead and to fill the inclosed area with sand.

This work was accomplished as a PWA project in 1935 under supervision of the Constructing Quartermaster at a cost of $976,000. A total of about 750 wood piles were replaced by driving through holes cut in the concrete deck, and a great many more were repaired by encasing the damaged portion with concrete. The area was, at this same time, inclosed by a steel sheet pile bulkhead extending over 5,600 feet around the perimeter of the wharves and piers, and filled with about 290,000 cubic feet of sand, hydraulically placed. The top of the steel bulkhead was anchored to the pile cap walls of the wharf aprons and sheds.

In order to reduce the water pressure behind the bulkhead, drainage slots 1 inch in width by 24 inches high were cut in the webs of the piling sections. The bottom of the slots was at an elevation of 2 feet above mean low water, and each slot was backfilled in its immediate vicinity with sand and gravel intended to serve as a filter.

Within a short time following placement of the sand fill behind the bulkhead, small outward movements of the wharf and wharfsheds were detected in certain sections. In 1937, to prevent further movement of the structures, a system of anchors was installed. Each anchor consisted of a mass of concrete buried in Terminal Street with a tie rod imbedded in the mass concrete and attached to the concrete pile cap walls.

Damage to Wood Piles Re-Occurs

By 1944, the sand fill behind portions of the bulkhead had been washed out through the drainage slots to a depth of 4 to 5 feet, and the wood piles were again severely damaged by new limnoria attacks and in 1945-1946 a drainage system was installed behind the bulkhead; the drainage slots previously cut were closed with welded steel plates, the most severely damaged piles were repaired by incasement in concrete and the lost sand fill was replaced.

First Discovery of Damage to Steel Sheet Pile Bulkhead

In March of 1953, it was discovered that holes in the steel sheet piling were occurring at and just below the low water line and that the sand fill was being washed out leaving the wood piles once more open to attack by the marine borers. In many cases where holes had not yet occurred, the thickness of the metal had been reduced to the extent that a sharp blow with a boat hook penetrated the piling. Further, the loss of cross-sectional area of the sheet piling was placing the structure in an unsafe condition, and necessity for major reconstruction was becoming evident.

Possible Causes of Loss of Metal

The problem of the corrosion of steel members in salt water has been and still is the subject of considerable study, with widely different theoretical explanations being given for what happens.

The first thought after discovery of the corrosion taking place was the possibility of loss of metal through electrolysis caused by stray currents from sources of direct current electric lines in the vicinity.

Therefore, two sets of recording chart readings were made for a period of 24 hours. However, both tests showed the salt water to be positive to the steel piling in the order of about 1 millivolt which indicated that deterioration of the steel piling could not be from stray currents.

Dr. A. P. Richards of Clapp Laboratories submitted a statement based on investigations and conferences with other scientists interested in the causes of and protection against corrosion in sea water. In part, Dr. Richards' statement is as follows: Quote—

"A number of theories have been advanced to explain the accelerated corrosion at the low water level, which is the problem in this case. One states that this is due to a marked reduction in aeration cell. The area affected by oxygen being the cathode or potential zone. It is stated that during a falling tide, the steel in the air becomes passivated. When the tide turns and the steel is again wet, the newly wet sections become active. The lowest area, of course, has less total exposure to oxygen, resulting in accelerated corrosion."

Investigations and Results

Faced with this latest development of corrosion, the New England Division office of the Corps of Engineers was directed to make a thorough investigation of the condition of the waterfront structures and to report on the findings of these investigations together with recommendations for repairs and the estimated costs thereof.

The firm of Fay, Spofford & Thorndike was engaged to make the investigations and prepare the report and to present various schemes from which one would be selected.

In December of 1953, the investigation had been completed, and a report was presented which indicated that corrosion in the bulkhead had progressed rapidly since the first discovery earlier in the year and that about 10 percent of the sheet piles had holes in them at low water line indicating that, because of the rapid rate of the corrosion, if corrective measures were not taken a failure of the bulkhead must occur, and that such a failure was likely to occur suddenly with disastrous effects on the wharf structures and possibly to the Wharf Shed. There appeared to be no practical stop-gap method of arresting the corrosion and providing a satisfactory safety factor to the structure.

Abandonment of the Waterfront Structures

Since the close of the last war, the waterfront structures were being leased to private interests and were not considered necessary for immediate Government use. Some thought was being given to complete abandonment of the structure. However, in view of the failure which would occur, with the possibility of portions of structures falling out into the adjacent waterway, it was determined that if the facilities were abandoned, some means of protection against collapse, or removal must be provided. It was estimated that temporary protection, which would provide satisfactory safety measures for about 10 years, would cost approximately $1,400,000. To remove the structures, with the exception of the major portion of the pier Sheds, would cost approximately $4,300,000. In both cases, all possibilities of shipping would be eliminated.

Move to Save the Army Base

Faced with the loss of Boston's greatest shipping facilities, the Port of Boston Commission, together with other local interests, held a meeting in Washington with Government officials in an effort to prevent this great loss. Considering the cost involved to abandon the structures, which the Government would have to expend, together with the inability to use the Army Base as a shipping port in the event of a national emergency, Congress passed an act authorizing the Department of the Army to rehabilitate the wharf and pier structure at a cost not to exceed $11,000,000 and to lease those facilities to the Commonwealth of Massachusetts. However, as a condition to the execution and delivery of such lease, the Commonwealth is to pay 10 percent of the total cost of rehabilitation. Further, the lease provides that during any national emergency, or declaration of war, if the Department of the Army determines that the leased property is necessary for military purposes, the United States shall have the right to re-enter such property for such period of time determined to be necessary in the interests of national security.

Basis of Method of Repairs

The report of December 1953 by Fay, Spofford & Thorndike based on field investigations contained four recommended schemes and estimated cost of each for rehabilitation of the wharf and pier structure. After evaluation of each scheme by various departments of the Army, a final one was selected. In July 1954, the New England Division office negotiated with Fay, Spofford & Thorndike for designs, plans and specifications which were completed in April 1955, issued for bids and finally awarded for construction in June 1955.

REHABILITATION OF WHARVES AND PIERS ARMY BASE, SOUTH BOSTON, MASS. — DESIGN

By F. L. Lincoln,* *Member*

(Presented at a joint meeting of the Boston Society of Civil Engineers and Structural Section, B.S.C.E., held on January 23, 1957.)

When the problem of devising a scheme for rehabilitation of the wharf structures at the Army Base, as described by Mr. Basso, was undertaken in the summer of 1953, there were many conditions which influenced the thinking. It will be helpful if a few are kept in mind during this discussion.

1. The steel sheet pile bulkhead was so badly corroded in the low tide area that there were many large holes through it and in many places the metal was paper thin.

2. The commercial and military shipping interests desired berth depths of 35 feet at M.L.W., but this depth was available in only two of the ten berths and deeper dredging of the others had been avoided for many years to safeguard the structures.

3. The subsurface material in which the new structure would be built varied from a deep layer of soft clay extending down to 60 feet below M.L.W. at the Summer Street end of the property to ledge rock higher than the desired 35-foot depth of berth at the outboard end.

4. The reconstruction scheme must permit the continuous operation of the base and most of the ships' berths during the construction period.

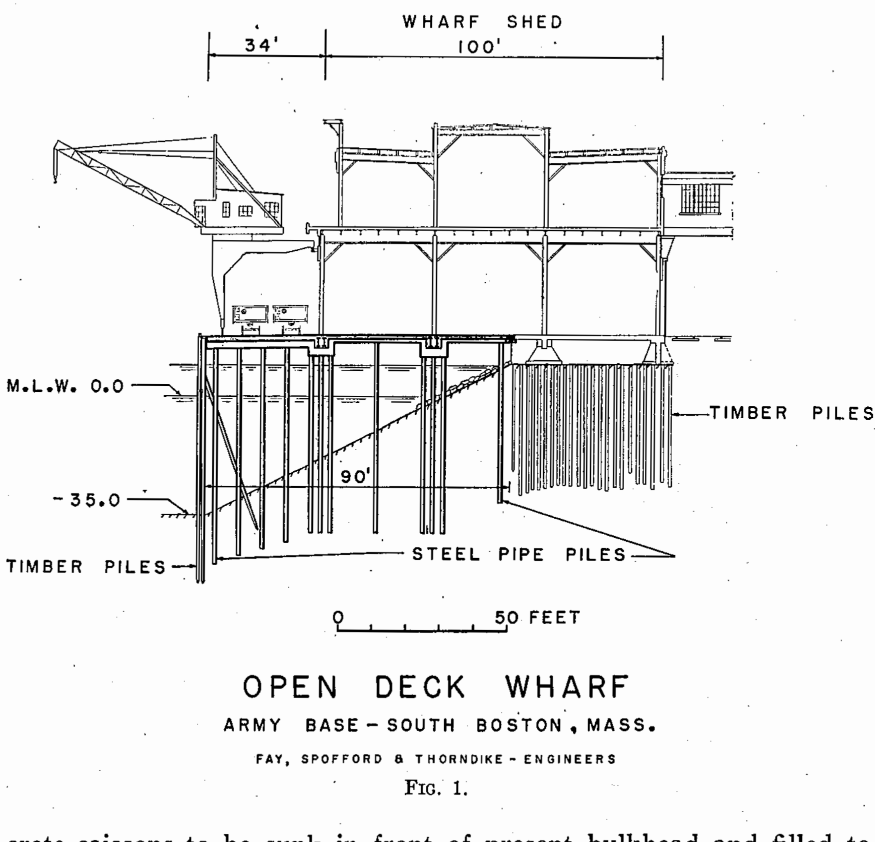
5. No type of structure was excluded from consideration, but little enthusiasm was indicated for a second steel sheet pile structure.

Rehabilitation Schemes Considered

A great variety of rehabilitation schemes were considered, many of which were found to be impractical and were discarded at an early date. Those which appeared more practical were developed in sufficient detail to permit study of the methods of con-

*Partner, Fay, Spofford & Thorndike, Boston, Mass.

struction, the advantages and disadvantages of the finished structure, and the approximate construction cost in each case. These schemes included some structures which would make use of the present steel sheet piling by adding buckstays or other strengthening members, new steel and concrete sheet pile structures to be driven in front of the present bulkhead, floated-in-place con-



OPEN DECK WHARF

ARMY BASE – SOUTH BOSTON, MASS.

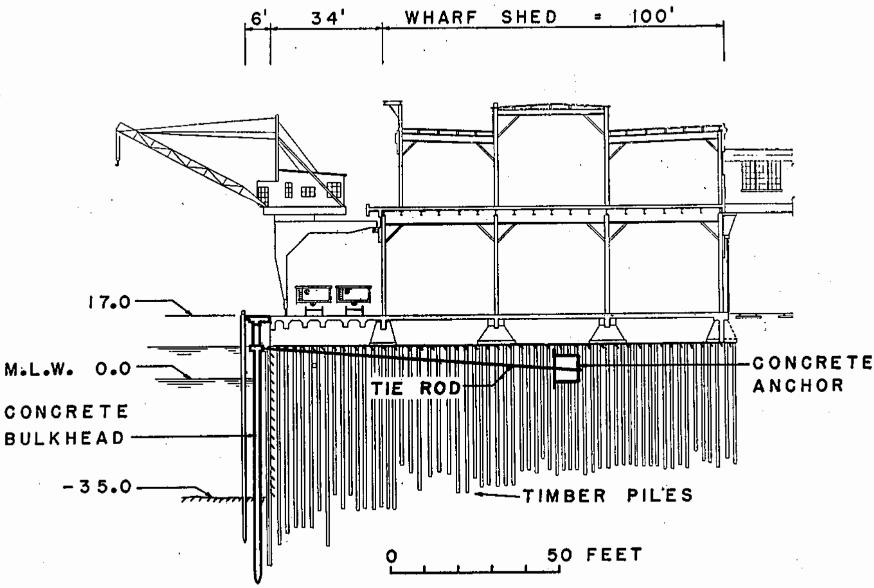
FAY, SPOFFORD & THORNDIKE – ENGINEERS

FIG. 1.

crete caissons to be sunk in front of present bulkhead and filled to make a gravity wall, relieving platform type structures in front of present bulkhead, cast-in-place gravity walls of concrete, gravity walls of precast concrete blocks, and open pile and deck structures with earth slopes.

From the various schemes studied in this preliminary manner, four were chosen to be developed more fully as to detail of design and construction, and as to utility and maintenance. This selection was made with the assistance of representatives of the Office of Chief of Engineers, the First Army, the Boston Army Base, and the New England Division at conferences held in Boston and Washington. The four schemes chosen were:

1. Open wharf as shown on Figure 1. Installation of a new



CONCRETE PILE BULKHEAD

ARMY BASE - SOUTH BOSTON, MASS.

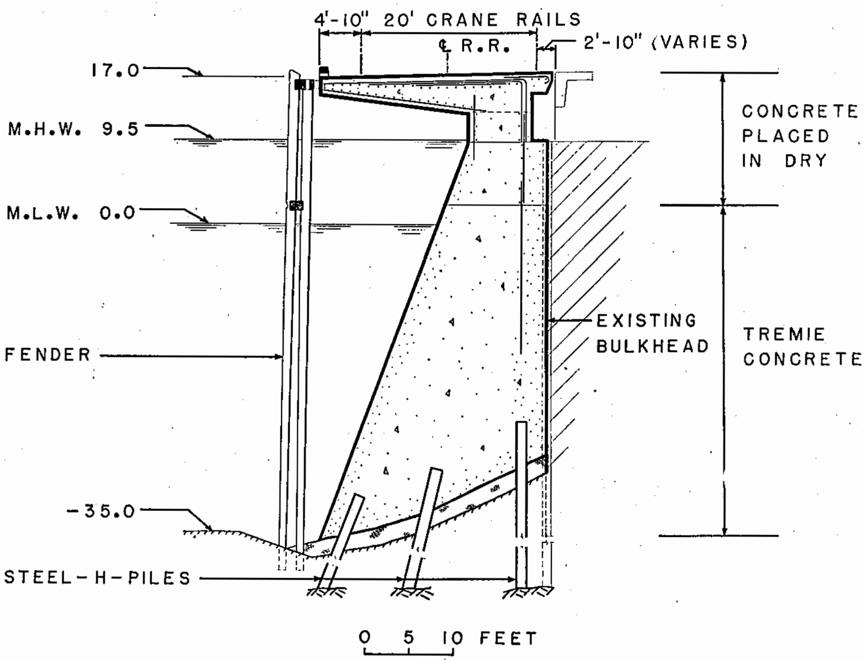
FAY, SPOFFORD & THORNDIKE - ENGINEERS

Fig. 2.

open deck wharf, removing the existing steel sheet pile bulkhead and sand fill, and using a new concrete deck supported on creosoted timber and concrete-filled steel pipe bearing piles, with the new structure located, as far as possible, within the present wharf area.

2. Concrete bulkhead as shown on Figure 2. Installation of a new concrete sheet pile bulkhead located outside of the present wharf face at the West Open Wharf and Wharf Shed areas where the rock is low and located inside the present wharf face at the East Open

Wharf and outer end of the Pier Shed areas where the rock is high, the installation of a new narrow open wharf structure in front of the new and existing concrete bulkheads at the East Open Wharf and Pier Shed areas, and the restoring to service as a bulkhead the existing concrete sheet pile structure at the Pier Sheds.



CAST - IN - PLACE GRAVITY WALL

ARMY BASE - SOUTH BOSTON, MASS.

FAY, SPOFFORD & THORNDIKE - ENGINEERS

FIG. 3.

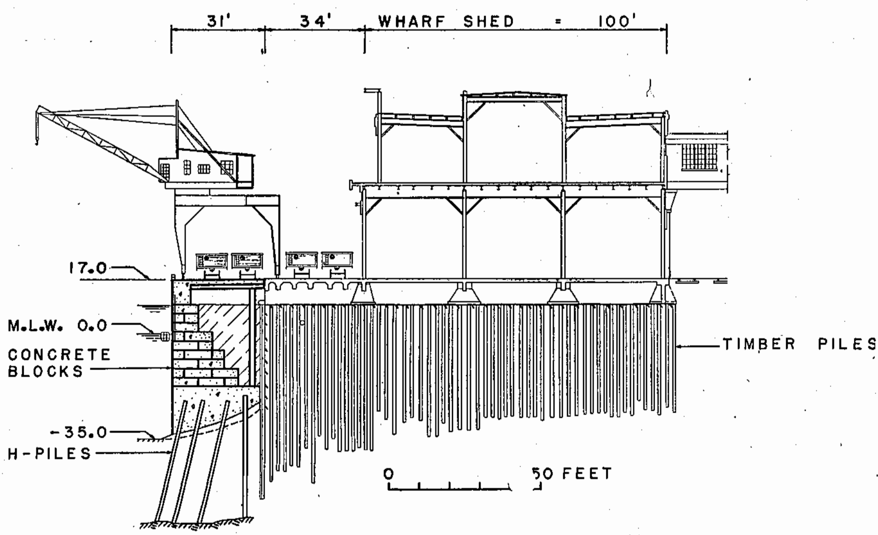
3. Cast-In-Place wall as shown on Figure 3. Installation of a cast-in-place concrete gravity wall outside of the present sheet pile bulkhead, using the sheet pile bulkhead as the rear form and cantilevering the deck forward from the wall.

4. Precast Block wall as shown on Figure 4. Installation of a gravity wall made up of large precast concrete blocks outside of the present sheet pile bulkhead and filling the space between the present bulkhead and the new wall.

Before proceeding with the comparison of these four schemes which were developed in detail, it may be of interest to discuss the reasons for discarding some of the others.

Strengthening Present Bulkhead Impractical

There are several reasons why the schemes involving continued use of the present steel sheet pile bulkhead were not considered practical.



BLOCK GRAVITY WALL
ARMY BASE - SOUTH BOSTON, MASS.
FAY, SPOFFORD & THORNDIKE - ENGINEERS
Fig. 4.

The badly corroded area of the sheet piling extends below mean low water, making it difficult to provide proper connections between the submerged undamaged portion of the sheeting and any new members which might be installed above.

Removal of some of the sand fill, as required to relieve the bulkhead, would necessitate protection of many of the timber piles by other means and introduce the hazard of damage to the piles during the sand removal process.

The depth of berth would be limited by the present bulkhead to 30 feet in Pier Shed and East Open Wharf Areas.

The anticipated life of the repaired structure would be comparatively short.

These schemes were all more or less makeshift in character, did not furnish any permanent solution, and did not possess the factor of safety of the other schemes.

Concrete Caissons Expensive. The use of caissons in the shape of reinforced concrete boxes, floated into position, sunk into place, and filled with rock, gravel or other material to form a gravity wall was not recommended because the estimated construction cost would be greater than that of a gravity wall of mass concrete while the length of life was believed to be less. The hazards accompanying such construction are great, particularly in this case where a tall but narrow caisson would be required, and an accident would be expensive in both construction cost and construction time.

Relieving Platform Life Uncertain

While the cost of a relieving platform type of structure was estimated to be slightly less than that of a gravity wall, this type of structure was not recommended because it would require a great deal of underwater work on the platform, the lower portions of the present steel sheet piles (from a few feet below mean low water to the lower ends) must be continued in service, and the life of the reinforced concrete platform which would have to be placed under water would be uncertain at best and surely shorter than that of a mass concrete wall.

Open Wharf in Front of Present Structure Too Wide

The use of an open wharf located entirely in front of present structure with an earth slope from the bottom of the dredged berth up to the top of the present fill would be the least hazardous of any scheme considered as far as construction difficulty is concerned, and its construction cost was estimated to be practically the same as that of the gravity wall of mass concrete. However, it would increase the width of the wharf about 60 feet and would require widening both the Wharf Shed and Pier Shed. Its maintenance over the years would be more than that of the gravity wall.

New Steel Sheet Pile Bulkhead Unattractive

Estimates of cost made for a new steel sheet pile bulkhead in front of the present one, using ordinary carbon steel shapes, were found to be nearly as much as for a concrete sheet pile bulkhead. Considering the experience with the previous structure, a second sheet pile structure would not be attractive unless longer life could be assured. The use of cathodic protection, combined with surface coatings, is probably the best means developed to date of preventing the type of corrosion which has occurred in the present structure, but this protection is still believed to be uncertain as to cost and effectiveness. The possibility of using nickel alloy steel was investigated. It was found that low alloy steels which cost considerably more than carbon steel are no more resistant to sea water than carbon steel. Monel metal has shown greater resistance to corrosion in sea water when tested in small samples. However, it is feared by some of the corrosion specialists that Monel may suffer the same destructive corrosion by galvanic action as the carbon steel has if it is immersed as a long member extending from below the mud line to the high tide line. In any case, Monel metal was not available in the shapes and quantities required for such a structure and has never been used for such purposes. Its cost is high, the base cost being about ten times that of carbon steel.

Comparison of Schemes

Each of the four schemes which were developed in full was believed to be a satisfactory solution to the problem and each had its advantages and disadvantages, the comparative value of each depending much upon the point of view of the evaluator. In the following tabulation we have rated each scheme in the order of desirability in which we believe it stands for any particular qualification:

	Open Deck Wharf	Concrete Sheet Pile Bulkhead	Gravity Wall Cast-in-Place	Gravity Wall of Blocks
Length of Life	3	4	2	1
Economy of Maintenance	3	4	2	1
Construction Disturbance	3	4	1	2
Construction Hazard	4	3	2	1
Utility in Operation	1	2	4	3
Factor of Safety	1	4	3	2
Fire Danger	4	3	2	1
Resistance to Explosion	3	4	1	2
Construction Cost	3	1	2	4

Length of Life. The life of the precast block gravity wall with granite facing of the upper portion of its vertical face should be very long with little maintenance. The cast-in-place gravity wall with timber facing on the upper portion of its sloped face and cantilever deck would require replacement of timber facing in time and the cantilever deck would have a shorter life than the simply supported deck located behind the vertical face wall. The concrete sheet pile structure would have the shortest life, the limiting features being the spalling of the concrete and the corrosion of the steel in the piles themselves and the corrosion of the tie rods. In the schemes in which an open deck is used, it is expected that creosoted timber piles would have a longer life than the concrete sheet piles, barring disastrous fire, and without any maintenance work. Concrete-filled pipe piles would be expected to have a life as long or longer than the creosoted piles, but with considerable maintenance work, such as jacketing in the severe corrosion range.

Maintenance. With a gravity wall the maintenance would be reduced to a minimum on the wall itself and, although replacements would be required to the structure behind the wall in time, it is believed that the work would be small for a number of years. With the new open deck schemes, the deck which is replaced would be free of maintenance for a long time but the exposed piles would need maintenance or replacement sooner than the gravity wall. Concrete sheet piles and their tiebacks would probably require the earliest and most extensive maintenance of any of the parts and would be least able to stand the deferring of maintenance, should economic or national emergency conditions require it.

Construction Disturbance and Hazard. It would be necessary to interrupt somewhat the operation of the terminal to make any of the installations. The gravity wall schemes constructed outside of the present steel sheet piling would cause the least interference to the operation of tracks, sheds and open areas and would tie up a berth for a shorter period than the scheme of removing the sand fill and installing new piles.

The hazard of an accident during construction which might lead to collapse of part of the wharf and Wharf Shed was believed to be the least during the installation of the gravity wall, the hazard in this case being that caused by pile driving in front of the present structure and preparation of the bottom for placing the concrete base.

When the base is placed, the present structure should be relieved considerably.

Utility in Operation. Constructions included in the gravity wall schemes would extend about 31 feet beyond the original face of the wharf and hence require relocation of crane rails and railroad tracks. Under these schemes, the width of apron is more than would ordinarily be provided, and while it was believed that the cargo handling operation would be entirely workable, changes to the sheds would be needed to make operation as satisfactory as before or as provided by the concrete sheet pile and open deck schemes.

Factor of Safety. In general, it was felt that any scheme in which the horizontal earth loads are taken by side slopes and the loads to be resisted by structures are principally vertical is the safest. In this particular case, there would be more construction hazard in bringing about this open wharf scheme, but after the construction, it was felt that the factor of safety against normal conditions would be the greatest.

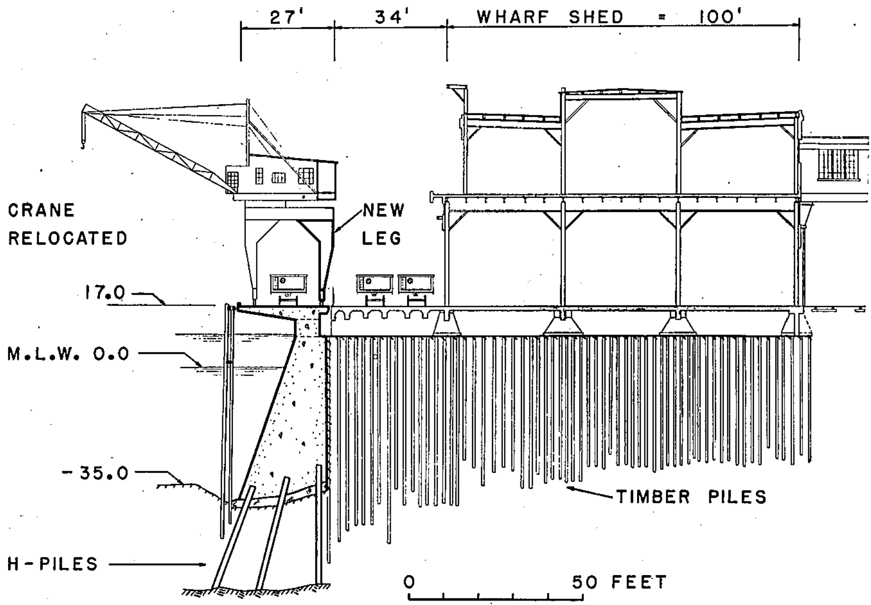
Against such hazards as explosions in a ship, bombs or earthquake, the gravity wall structures would probably have the greatest resistance and the concrete sheet pile the least.

Against such a hazard as an oil fire, the straight face gravity wall should be the most resistant and the open deck the most seriously affected.

Construction Cost. Considering first cost only, the least expensive scheme would be that of the concrete sheet pile bulkhead and the most expensive the gravity wall of block construction. However, when the cost per year of life is considered, the order changes and the cast-in-place gravity wall becomes the most economical, the block wall second, the open deck wharf third and the sheet pile bulkhead the most expensive. This condition holds whether an interest rate on the money invested is included or not. The determination of the cost per year is made by assuming a maintenance free service life of the gravity wall of precast blocks to be 75 years, of the gravity wall of cast-in-place concrete to be 60 years, of the new open deck structure to be 40 years, and of the concrete sheet pile bulkhead to be 25 years.

Scheme Adopted

The scheme adopted for construction is the cast-in-place concrete gravity type sea wall (Figure 5) located immediately outside the present steel sheet pile bulkhead. The wall has a sloping front face and cantilever deck to make most efficient use of the mass of material and to utilize the present bulkhead as a form for the rear side of the wall.



CAST - IN - PLACE GRAVITY WALL

ARMY BASE - SOUTH BOSTON, MASS.

FAY, SPOFFORD & THORNDIKE - ENGINEERS

FIG. 5.

The wall is supported on 14-inch, 89-pound steel H-piles driven to rock, except in the few areas where the rock is so high that the piles are omitted. The piles project up into the concrete which is placed under water by the tremie method. The tremie concrete extends from the ground line at Elev. —35 up to a point just above the low water line. The remaining concrete is placed in the dry. The concrete is placed directly against the present sheet pile bulkhead from the ground line to the top of the sheeting.

The present steel sheet pile bulkhead and the structures behind it are retained in place without change, except for some necessary repairs to the deck in front of the Pier Shed, changes to the gantry cranes to make them full-portal rather than semi-portal in style, and minor rehabilitation of the Wharf and Pier Sheds. By the installation of the new wall, the wharf is widened about 27 feet with the fender system and the gantry crane rail at the face of the wharf moved seaward the same distance. One additional railroad track is installed on the new portion of the deck, making three tracks in the Wharf Shed area and two tracks in the Pier Shed areas.

With the new construction, the width of the apron becomes 60 feet from the side of the Wharf Shed to the face of the wharf and becomes 54 feet from the side of the Pier Shed to the face of the wharf. These widths are greater than would be recommended in a new installation for efficient handling of freight, particularly that handled between the second floor and the ship by use of ship's tackle, but they are entirely workable. The wide aprons are advantageous for trailer truck operation.

Fresh water service to ships is furnished through 2-inch hose connections in pits at the face of the wharf at each berth. The 6-inch service piping is not protected from frost but is sloped to drain back to the valves located underground back of the sea walls.

Electric service to ships is furnished at 120/208 volts through dock receptacles also located in pits at the face of the wharf at each berth.

Provisions are made for telephone connection to ships at each berth through outlets at the wharf face, but the conduits only are being installed at this time.

The entire deck area is paved with a bituminous pavement 2 inches thick.

In the original construction the gantry crane rails were installed only at the Wharf Shed and East Open Wharf which cover four berths. In the present reconstruction, crane rails are being installed in nine of the ten berths, the berth omitted being No. 3 which is across the outer end.

Basis of Design

The wall is designed as a gravity section to withstand full earth pressure and an unbalanced head of water of 7 feet. The earth fill extends up to Elev. 110, which is about a foot above mean high water. In the design no advantage is taken of the strength of the present sheet pile bulkhead which is left in place, but it is assumed to provide a permanent cutoff at the bottom of the masonry wall. The drainage system which was in place behind the sheet pile bulkhead at the West Open Wharf and at the Wharf Shed is continued in operation and a system of weep holes installed in the Pier Shed area. The drainage system discharges through new outlets extended through the wall.

The 14-inch, 89-pound steel piles supporting the wall as designed for a maximum load of 77 tons with full vertical live and dead loads and the lateral earth and water pressures. The pile load for the normal conditions of dead load only and full lateral earth and water pressure is 65 tons per pile. At the few locations where the rock is high, the wall is placed directly upon it, a small amount of underwater rock excavation being required to make the 35-foot depth. At the transition points adjacent to high rock, where piles less than 10 feet long are required, the use of a few heavy 193-pound piles cored into the rock to provide adequate resistance to the wall against sliding is anticipated. To date, the use of such anchor piles has not been necessary.

The deck is designed for the standard A.A.S.H.O. H20-S16-44 truck loading, plus a 30 per cent impact factor. The railroad track supports are designed for Cooper's E-40 locomotive loading and 100-ton total load railroad cars, plus 25 per cent impact factors. The crane rail supports are designed for trucks of four 16-ton wheels, plus a 25 per cent impact factor. The cantilever deck of the cast-in-place wall is designed for the most severe combinations of these loads.

The wall is designed for a depth of berth of 35 feet at mean low water. If desired in the future, the depth of berth can be increased to 40 feet at a point 10 feet in front of the wall by maintaining a sloped surface between the wall and the 10-foot line.

Before designing the concrete mixes, the New England Division of the Corps of Engineers made an exhaustive study of aggregates available in this vicinity. Alternate mixes were designed for concrete using crushed stone and gravel and for the use of subaqueous concrete placed by the tremie method and that placed by the Prepakt method. The mixes using crushed stone aggregate contained less cement and fly ash per cubic yard than those using gravel aggregate. The most favorable price received was for crushed stone concrete placed by the tremie method and the contract was awarded on that basis.

The concrete was designed to produce a minimum compressive strength of 3000 psi at 90 days, and consisted of crushed stone coarse aggregate with a maximum nominal size of 2 inches, sand, type I Portland Cement and fly ash. Approximately one-half bag more cement per cubic yard is used in the tremie concrete than is required to obtain the specified strength, to compensate for losses or uncertainties for concrete placed under water.

The mix established for the tremie concrete has the following composition per yard:

Cement	423 lbs.
Fly Ash	141 lbs.
Sand	1,225 lbs.
2-inch Crushed Stone	1,206 lbs.
¾-inch Crushed Stone	805 lbs.
Water	260 lbs.
Darex	10 oz.
W/C Ratio	5.2

It will be noted that the mix has a total of six 94-pound bags of cementing materials, 1½ bags of which are fly ash.

The average compressive strength of the concrete cylinders made in the first several months of operation is

at 7 days	1,973 psi.
at 28 days	3,290 psi.
at 90 days	4,173 psi.

Shores and forms supporting concrete are kept in place until the concrete develops a minimum compressive strength of 2,000 psi, which required the supports be retained an average of approximately seven days through the warm months.

The studies conducted by the Corps of Engineers indicated that the use of fly ash, replacing 25 per cent of the cement, would have the following benefits:

1. Increase the durability of the concrete, especially where exposed to salt water and freezing temperatures.

2. Permit the use of Type I cement without losing the resistance to salt water ordinarily obtained with Type II cement. Adequate supplies of Type II cement have not been available for a project of this size.

3. Increase the workability of the concrete.

4. Minimize cracking due to contraction.
5. Produce a saving, because the cost of fly ash is less than the cost of cement. In this case, there is a saving of approximately 62 cents per cubic yard of concrete. The additional costs of handling, storing and weighing of the fly ash, as observed on this job, raises some doubt as to whether the difference in price is entirely justified.

To protect the concrete within the tidal range from damage by the salt water and by temperature changes, a permanent facing of creosoted timber plank is provided. The creosoted plank is used as a liner inside the tremie concrete form from a point about 2 feet below low tide up to the top of the tremie concrete, and as the form itself for the wall which is poured in the dry. The forms for the underside of the deck are also left in place, but they are of untreated lumber.

The fender system consists of Greenheart fender piles backed up by rectangular rubber blocks, 7 inches wide by 10 inches deep by 12 inches long with a 3-inch diameter bore, which are compressible from a thickness of 7 inches to about 3 inches. The force required to compress each rubber block this amount is about 50 tons. The piles are blocked in place with creosoted pine chocks and wales.

The railroad track is of girder rails embedded in the concrete deck, the rails weighing 128 pounds per yard and being set on steel plates. The new crane rail is of heavy tee section also set on steel plates.

Expected Life

The service life of this structure is expected to be long as compared to most of our wharf structures, and with a minimum of maintenance expense. The steel H-piles which are completely buried in the harbor bottom and in concrete are believed to be permanently protected from corrosion unless strong stray electric currents should develop or destructive chemicals become present in the soil. Inspection of the present steel sheet piling shows practically no loss of metal at the mud line after being in place 20 years. Actually, in the bearing piles the stress is low, so that a considerable area of the steel could be lost without danger to the wall. The gross area of metal of the pile is required not to satisfy the stress requirements of the pile itself but to provide a satisfactory connection between the pile and the concrete and between the pile and the rock, points which are believed to have the greatest protection against corrosion.

Mass concrete has been used in many dry dock, bridge pier, and sea wall constructions, and below the tidal range it has generally proven to have long life with little deterioration. In the cases where repairs have been necessary below the tidal range, it has generally been at locations where there were strong currents. Until recent years, walls of this type were made without the use of air-entraining cements or fly ash, and it appears to be conclusively established that the resistance of the concrete is increased by the use of such materials.

Experiences on several structures in place twenty years or more indicate that the creosoted timber sheathing used will protect the concrete in the vulnerable tidal range as long as the sheathing itself remains intact; that properly creosoted plank should have a life of at least twenty-five years, and that it can be replaced without too much difficulty as replacement is necessary.

ACKNOWLEDGMENTS

I want to express on behalf of my office our appreciation of the helpful and cooperative way in which the many members of this combined team have worked together on this not too simple undertaking. There are so many individuals from the various Army Engineer Offices, Contractors' and Subcontractors' offices and my own office and field forces who have contributed to this project and, incidentally, the preparation of this paper, that I must refrain from thanking individuals and express one big "Thank You" to them all.

REHABILITATION OF WHARVES AND PIERS ARMY BASE, SOUTH BOSTON, MASS. — CONSTRUCTION

By Joseph Peraino*

(Presented at a joint meetinng of the Boston Society of Civil Engineers and the Structural Section, B.S.C.E., held on January 23, 1957.)

As a representative of the contractor, Merritt-Chapman & Scott Corporation, it is now my privilege to explain how the plans and specifications of the owners and engineers have been translated into a physical structure.

As you probably already are aware from published reports in technical publications two basic problems confronted the contractor in this major project:

(1) Construction work required close coordination with ship movements to keep disruption of pier operations to a minimum.

(2) A massive subaqueous gravity wall and cantilever deck more than a mile in over-all length had to be constructed without visible anchorage.

The contractor's phase of the project started May 13, 1955, when Merritt-Chapman & Scott was the successful low bidder, with an estimate of $7,995,582.

In addition to the wharf and pier rehabilitation, which increased the width of the entire open pier by 27 feet, the project entailed rehabilitation of the Wharf and Pier Sheds. For the purposes of tonight's discussion, we will confine our review to the gravity wall and cantilever slab structures. In the interest of clarity, we will discuss this phase of the work in chronological order, starting with preparation of the bottom and design of the special forms for the work. I think it should be pointed out in passing that to our knowledge this job represents a unique construction "first" in terms of underwater concreting.

The construction of conventional bridge piers, drydocks, etc., by

*Chief Engineer, Merritt-Chapman & Scott.

tremie concreting is comparatively routine when compared with the problems of building a geometrical molded form underwater, as required in the case of the Boston Army Base. Add to this the fact that no visible anchorages were provided in the specifications, and you begin to have an appreciation of the trail blazing construction techniques that were required.

As already indicated, the physical proportions of the gravity wall, and the volume of materials required, were in themselves impressive. Running around the perimeter of the pier for a distance of approximately 5,500 feet, the wall rises 52 feet from a minimum elevation of —35 ft. below mean low water. It is 27 feet wide at the base and narrows to eight feet in width at mean high water line or elevation +9.5. Construction of the wall and its 27-foot-wide cantilevered deck will require a total of approximately 180,000 cubic yards of concrete and 4,100,000 lbs. of reinforcing steel

Design of the special gravity wall forms obviously represented the major consideration in all planning for this job. Upon award of the contract, Merritt-Chapman & Scott proceeded to finalize and detail the preliminary drawings that had been developed during the estimating period.

The gravity wall forms were uniformly 49′ 6″ high, with 5′ 3″ extensions that could be installed at the toe wherever depth required. In essence, these forms were extremely simple. Each set was 94′ 6″ long which, for ease of handling, was designed in three sections—two of them 34′-2″ in length, and the other 26′-2″.

The face of the forms was fashioned from light steel plate, stiffened at approximately two foot centers with horizontal steel joists. This panel construction was backed by vertical triangular shaped trusses made up of wide flange beams and "K" web bracing. The trusses were spaced approximately 10′ 6″ on center and tied longitudinally with horizontal bracing. Yokes attached to the extremities of the outboard chord member of each truss provided guides for spud piles with which the forms were permanently positioned for concreting.

A simple interlocking arrangement along the edges of the face of the form permitted each section to be joined to the other by a single length of flat web sheet piling.

This special design greatly expedited alignment of the forms. Alignment of the outboard chord of the truss in a vertical position automatically set the surface of the form at a slope of 4½ on 12, which was the specified batter for the gravity wall.

Once aligned, the forms were pinned in place by driving the spud piles through the yokes to a minimum penetration of 10 feet below the prepared bottom. The steel sheet pile face of the existing pier served as the inboard form. The 26′-2″ length comprised the center section of each set of forms. The outer ends of each of the two 34′-2″ sections were fitted with sheet pile interlocks to receive transverse bulkheads.

The inboard end of the closure bulkheads was secured in place by two vertical triangular-shaped trusses. The vertical truss was joined to the bulkhead by a sheet pile interlock arrangement. Once the bulkhead was positioned, the truss was pinned in place by driving two vertical piles through yokes on its outboard legs.

At bottom the reaction was taken by penetration of the piles into the river bed. At the top, the trusses were tied together by a longitudinal tie rod extending across the top of the form.

Throughout the job it was standard practice to set 94′ 6″ lengths of wall in alternate positions. When filling the gaps, between these alternate sections the previous pours served as end bulkheads. All told Merritt-Chapman & Scott used three sets of forms for the gravity wall, two with end bulkheads and one without.

Concurrently with finalization of the form design, construction activity was being pushed in the field. The Army Base Pier at South Boston is divided into ten berths. Under the terms of the contract governing construction schedules, the contractor could occupy only two berths at a time for construction. Under this schedule, the contractor was required to consider each berth virtually a separate contract since he had to schedule his work entirely in accordance with the availability of idle berths.

Normally, it would be a contractor's practice on a project such as the Boston Army Base Pier to schedule each phase of the work in one continuous operation. In other words, preparation of the bottom would have continued progressively along the length of the pier with construction of the gravity wall following immediately behind. Construction of the cantilever slab similarly would have followed close on the heels of each section of gravity wall.

Since certain berths had to be kept free until released to the contractor, this progressive schedule could not be followed at South Boston.

As a first construction step, pile loading tests were made to establish the length of foundation piling required to support the gravity wall. Each test pile was loaded to 150 tons.

Except for a small section resting directly on rock, the wall is supported throughout its length by three rows of H piles. Along part of Berth 3 the wall rests on rock.

Preparatory to placement of the piles, the bottom was dredged approximately to elevation —37.0. A two foot blanket of gravel was then placed as a bedding for the tremie concrete wall. The inboard row of 14″ bearing piles 89#, spaced 10′-6″ on centers, were driven vertically into the bottom to an average penetration of 25 feet, and imbedded in wall approximately 8′ above the gravel bed. The two outboard rows were driven on a batter toeing outboard and 3′-6″ centers.

Upon completion of the design and details by Merritt-Chapman & Scott construction of the forms was entrusted to Blaw-Knox Company of Pittsburgh, Pa. While the forms had been scheduled for completion in two and a half months, delivery was delayed six months by an acute shortage of wide flange beams. Eventually, warehouse steel had to be used for fabrication, and I need hardly emphasize to you gentlemen the cost entailed.

For convenience in shipment, forms were fabricated sectionally for assembly at the project site. Upon arrival, the component parts were placed aboard barges and welded into complete units.

Accuracy in placement of the forms was insured by a template arrangement extending eight feet beyond the face of the dock's existing sheet piling. Placement was by floating derricks, which could easily handle the approximate 36 tons average weight of each section. Once the top of the form was flush against the template, the outboard chord of the truss was plumb, the form was temporarily secured. Adjoining sections were successively tied together by means of an interlocking sheet pile, and closure bulkheads added. After the entire 94′-6″ section was aligned, the outboard spud piles were driven. This secured the toe of the form. The top of the form was secured by a tie rod anchorage system running beneath the existing deck of the pier. The outboard end of the tie rods, spaced between the master piles, were connected to a strongback, set outboard of the piles.

The inboard end of the tie rod anchorage system posed special problems for the contractor because of limitations imposed upon him by the specifications. Fearing that the pier was in too weakened condition to withstand any lateral forces, the U. S. Army Corps of Engineers had ruled that no anchorage could be secured to the existing structure.

Following a series of proposals submitted by the contractor to the consulting engineers, the following arrangement was arrived at. A row of vertical anchorage piles was driven through holes in the pier deck 80 feet inboard of the existing sheet pile bulkhead, or 40 feet inside the pier shed. The first floor of the shed was only 32 feet high which, together with seven feet leeway below the deck, effectively limited pile lengths to 30 feet.

After being driven, the pile tops were chocked against the existing pier deck, distributing the load between the deck and the fill beneath the pier. 2½″ diameter rods coupled into 116 foot lengths were used to tie gravity wall forms to these anchorage piles.

As a further precaution, the consulting engineers initially directed the contractor to provide an additional anchorage by driving a second row of piles inboard of the first row and connecting them to the first row with another set of tie rods running above the deck. Strain tests taken on the tie rods during the first placement of concrete indicated, however, that this secondary anchorage would be unnecessary.

With the forms securely anchored, concreting got underway. Each 94′ 6″ section of gravity wall entailed the placement of approximately 2,100 cubic yards of tremie concrete to elevation +2.0.

Since only 2 sets of forms had the end closure panels, the third form was set and ready to receive the closure panel as soon as it was stripped from previously placed and set monolith.

The extensive fleet of floating equipment placed on the job by Merritt-Chapman & Scott included the following:

One 75 Ton Floating Whirley Derrick.
Two 50 Ton Floating Whirley Derricks.
One Floating Concrete Plant.
One Floating Pile Driver.
One Stiff Leg Floating "A" Frame Derrick.
Two Truck Land Cranes.
One Cats Driven Crane.

And numerous smaller pieces of equipment.

Concrete was batched aboard a floating concrete plant with 110 c. y. capacity per hour. Concrete for the gravity wall was placed through four tremie pipes at an average of 50 yards per hour. Stone, sand and cement was delivered to the job site aboard barges. Fly ash was received on freight cars in L. C. L. containers.

Once the tremie concrete had been placed in the three 94′ 6″ forms, a seven-foot lift was added in the dry, carrying the wall to elevation +9.3. After allowing the concrete to set for 3 days, forms were stripped preparatory to installing two of them for the two intermediate monoliths. While forms were being stripped, the anchorage system for the first two sections was removed and repositioned to handle the intermediate pours.

It might be noted at this point that the creosoted timber lagging that sheaths the outboard face of the gravity wall from elevation —2 to +9.3 were built into the forms during their construction, and left affixed to the concrete when forms were stripped.

It was general practice to build five adjoining monoliths at a time since their total length approximated one berth. As berths were successively turned over to the contractor, he repeated this process of dredging, sanding, pile driving, placement of forms and concreting. He consequently was frequently moving from one end of the pier to the other.

Construction of the 27-foot deck, slab, with 17′ 2″ cantilever, was started following completion of the gravity wall sections. The cantilever slab tapered from 4′ to 1′ 6″ in depth. The inboard end of the new deck slab abutted the deck of the existing pier.

An underwater footing for forms used in construction of the cantilever slab was provided during erection of the gravity wall. This was achieved by boxing out the gravity wall form at elevation —2 so as to create pockets to receive the struts supporting the cantilever forms.

Frames for the cantilever forms were fashioned from light channels and angles in approximately the shape of a number 7, with a supporting diagonal strut. These frames were individually set at 3′ centers, with the lower end of the struts resting in the prepared pockets previously cast in the outer face of the gravity wall. The top section of the frame was anchored by tie rods embedded in the previously poured concrete of the gravity wall.

The top members of the frame were sheathed with two-inch timber lagging, which, after serving as the bottom form for placement of concrete, was left in place to protect the undersurface of the slab.

Creosoted sheathing served as the outboard form for the vertical portion of the gravity wall, and was left in place when forms were stripped. Spikes nailed to the inboard side of the sheathing acted as anchorage for the lagging.

The cantilever slab was cast in 31′ 6″ sections. All told, approximately 130 frames were purchased and used in assembling 13 sections of forms. The 31′ 6″ sections were cast in alternate monoliths.

Before concreting, reinforcing steel was placed and secured. Cantilever slab forms were boxed out for utility manholes and bollards. As a final step, provision also was made to box out depressions in the cantilever slab for a set of gantry rails and a set of standard gauge railroad track. This was done by suspending steel forms from a strongback placed transversely over the form. Anchor bolts for the gantry and railroad rail were secured to forms and cast in concrete slab. These forms were securely chocked against the strongback to prevent uplift during placement of concrete. Forms were stripped after compressive strength of concrete reached 2,000 lbs. per sq. inch.

After construction of the cantilever slab, a fender system comprised of a double row of timber piling was placed. Piles were driven on 10′ 6″ centers, their tops resting against a rubber block secured to the inboard face of the inner pile. The lower chocks are set at Elevation +2.5.

After placement of the gantry and standard gauge railroad tracks, the deck of the cantilever slab was covered with a 2″ surface of bituminous paving.

A comparatively minor, but most interesting phase of the contractor's work on the Boston Army Project involved reconstruction of its two gantry cranes. Prior to rehabilitation of the pier, their two outboard legs rode on a rail atop the existing deck. The inboard edge of the crane rode on a rail framed into the side of the pier shed above the deck.

Transfer of the gantries to their new location atop or new deck required construction of a new set of inboard legs. As you can readily gather from the foregoing, there were comparatively few unique construction aspects to rehabilitation of the Boston Army Base Pier, beyond the basic conception and design of the gravity wall and cantilever slab forms. However, the contractor's problems were complicated by the requirement to schedule the rehabilitation berth by berth, as they became available. The project could have been expedited considerably had the contractor been able to carry out each operation in a continuous, uninterrupted cycle. This would have required occupation of more than the two berths permitted at one time. The expeditious manner in which this project has been executed in the face of these difficulties is a tribute to the cooperation of all four parties involved: the owner, the operators of the pier, the consulting engineer, and the contractor. On behalf of Merritt-Chapman & Scott, we want to take this opportunity to express the contractor's thanks for the cooperation shown throughout by the U. S. Army Corps of Engineers, as the owners, Tidewater Terminal as the operators, and by Fay, Spofford and Thorndike, as the consulting engineer.

THE NEW ACI CODE — ITS IMPLICATIONS AND RAMIFICATIONS

By Howard Simpson,* *Member*

(Presented at a meeting of the Structural Section, B.S.C.E., held on December 12, 1956.)

Introduction

In 1956 the American Concrete Institute revised its Building Code requirements for Reinforced Concrete.[1] Some of the changes are of considerable importance. It is the purpose of this paper to discuss the more significant of these, with particular attention given to an evaluation of the magnitude of their effects and to precautions which should be observed in their application.

Beams

Ultimate Strength Design

Undoubtedly the most significant of the revisions is the addition of the single sentence in Section 601(b):

"The ultimate strength method of design may be used for the design of reinforced concrete members."

The latitude which this provision gives the designer can be more fully appreciated when it is realized that the Code does not specify what factors of safety or load factors must be employed when using the ultimate strength method. The Code Committee does append to the Code an abstract of the Report of the ACI-ASCE Joint Committee on Ultimate Strength Design,[2] thus tacitly recommending, but not requiring, its use.

A. *Load Factors*

Section A604 of the Appendix states in effect that the ultimate strength of members not subject to wind or earthquake loading should be the larger of the values obtained by the following formulas:

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$$U = 1.2B + 2.4L \quad \text{(1)}$$

$$U = K\ (B+L) \quad \text{(2)}$$

where U = ultimate strength of section
B = effect of basic load, consisting of dead load plus any volume changes which may affect the strength of the member
L = effect of live load plus impact
K = 1.8 for beams subjected to bending only.

Similar equations are given for members subject to wind or earthquake loadings.

These equations enable the designer to take into account the fact that one can usually predict far more accurately the magnitude and distribution of dead loads than of live loads, and that there is a reduced probability that the maximum wind or earthquake loads would occur simultaneously with the maximum gravity loading.

If the live load equals or is less than the dead load, Equation (2) governs; that is, the computed ultimate strength must be at least 1.8 times the total load. If the live load is greater than the dead load, Equation (1) governs. Thus the required overall factor of safety of members without wind or earthquake loads will theoretically vary between 1.8 and 2.4, depending upon the ratio of live to dead load. If this ratio is 3, a value which is rarely exceeded in practice, the required ultimate strength is 2.1 times the total load.

B. *Ultimate Strength Equations*

Section A605(b) of the Appendix gives for the ultimate resisting moment of a singly reinforced concrete beam

$$M_u = bd^2 f'_c\, q\ (1\text{-}0.59\ q) \quad \text{(3)}$$

where $q = pf_y/f'_c$
b = width
d = effective depth
f'_c = yield point of reinforcement, but no greater than 60,000 psi. (f_y = 40,000 psi for intermediate grade bars and 50,000 psi for hard grade bars.)
p = steel ratio. $p_{max} = 0.40\ f'_c/f_y$ for concrete strengths equal to or less than 5000 psi.

The above value of p_{max} is about 12 percent less than that necessary to develop the maximum resisting moment of the concrete thus providing some protection against downward variations in concrete strength.

The effect of the ultimate strength method on the dimensioning of beams subject only to dead and live gravity loading will now be considered.

C. *Comparison Between Elastic and Ultimate Strength Design*

One of the most significant effects of the employment of ultimate strength design is the possible use of much smaller concrete sections, without the necessity for providing compression steel. Assuming a concrete strength of 3000 psi, the ACI elastic theory requires that the maximum resisting moment of the concrete at working loads shall not exceed

$$M = 236 \text{ bd}^2 \qquad (4)$$

The corresponding equations for the ultimate strength theory are, for example:

$$\text{for } \frac{L}{B} = 1 : M = 509 \text{ bd}^2 \qquad (5)$$

$$\text{for } \frac{L}{B} = 3 : M = 436 \text{ bd}^2 \qquad (6)$$

Note that Equations (5) and (6) give maximum working load moments, and hence can be compared directly with Equation (4). They were derived by substituting in Equation (3) $f'_c = 3000$ and the maximum permissible value of p, then dividing by the factors of safety required for the respective load ratios assumed.

The use of Equations (5) and (6) is equivalent to increasing the allowable concrete stress at working load from .45 f'_c to .77f'_c and .69f'_c, respectively. (Maximum steel stress assumed retained at 20,000 psi.)

It should be noted that the steel ratio, p, required to develop the concrete resisting moments given by Equations (5) and (6) is .030 for intermediate grade steel and .024 for hard grade steel. These "balanced design" steel ratios are considerably larger than the .0136 required to develop the smaller maximum resisting moments permitted by the elastic theory. It is thus apparent that to avoid difficulties with steel placement, beams developing ultimate strength balanced design must be proportioned relatively wide and shallow. Shallow beams require larger steel areas, and are not usually economical; also, they may have excessive deflection. Furthermore, smaller concrete areas require more diagonal tension reinforcement. Therefore, except where it is necessary or desirable to severely limit beam depth, such as where headroom is restricted or in slab band construction, it will not ordinarily be practical or economical to take full advantage of the reduction in section dimensions permitted by ultimate strength design. If for the sake of argument, however, these problems are temporarily ignored, the ultimate strength method would permit, in the instance of a given beam designed for elastic balanced design, a 54 percent reduction in width or a 32 percent reduction in effective depth if L/B is equal to or less than unity, and a 46 percent reduction in width or a 26 percent reduction in depth if L/B equals 3. This is illustrated in Figure 1, together with the relative steel quantities required.

Since various considerations will usually dictate the use of sections which are under-reinforced according to the ultimate strength theory, the comparison of the two theories will now be extended to this type of section.

Defining M as the maximum allowable moment at working loads, the two curved lines in Figure 2 give the relation between M/bd^2 and p as obtained by dividing the expression for M_u in Equation (3) by the factors of safety 1.8 and 2.1, respectively (corresponding to $L/B = 1$ and $L/B = 3$). Intermediate grade steel and 3000 psi concrete are assumed. For comparison, the straight lines plot the elastic theory steel design equation

$$A_s = \frac{M}{f_s jd} \quad \text{.........................} \quad (7)$$

or, assuming j is constant at .87,

$$p = \frac{M}{bd^2} \times \frac{1}{.87 f_s} \quad \text{....................} \quad (7a)$$

These elastic theory curves are extended beyond the limits corresponding to elastic balanced design, and hence into regions where this theory requires compression steel. They are somewhat approximate in that they do not take into account the variation in j. Nevertheless, they serve as a means for comparing the tensile steel areas required by the two theories for a given moment and concrete section. For example, at the point where an ultimate strength curve crosses the $f_s = 20{,}000$ psi line, the area of tensile steel required by the ulti-

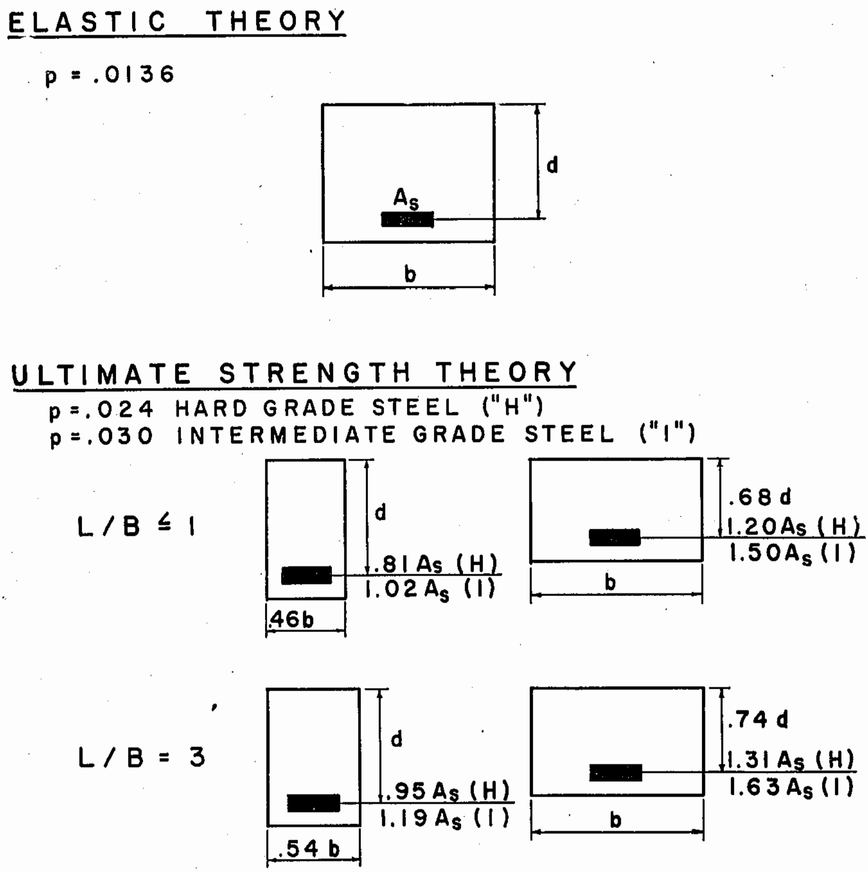

FIG. 1.—COMPARISON OF "BALANCED DESIGN" SECTIONS. ALL SECTIONS SUSTAIN THE SAME WORKING LOAD MOMENT.

mate strength theory equals that required by the elastic theory, if b and d are the same. Similarly, an intersection with the 22,000 psi line indicates a point at which the ultimate theory requires 20/22 as much tensile steel as the elastic theory. It should be kept in mind that the elastic theory also requires compression steel whenever p is greater than .0136.

Figure 2 shows that for intermediate grade steel and the values of M/bd^2 usually employed in elastic theory designs (equal to or less than 236 for 3000 psi concrete), ultimate strength design in effect permits a steel working stress of between 20,000 and 24,000 psi. For

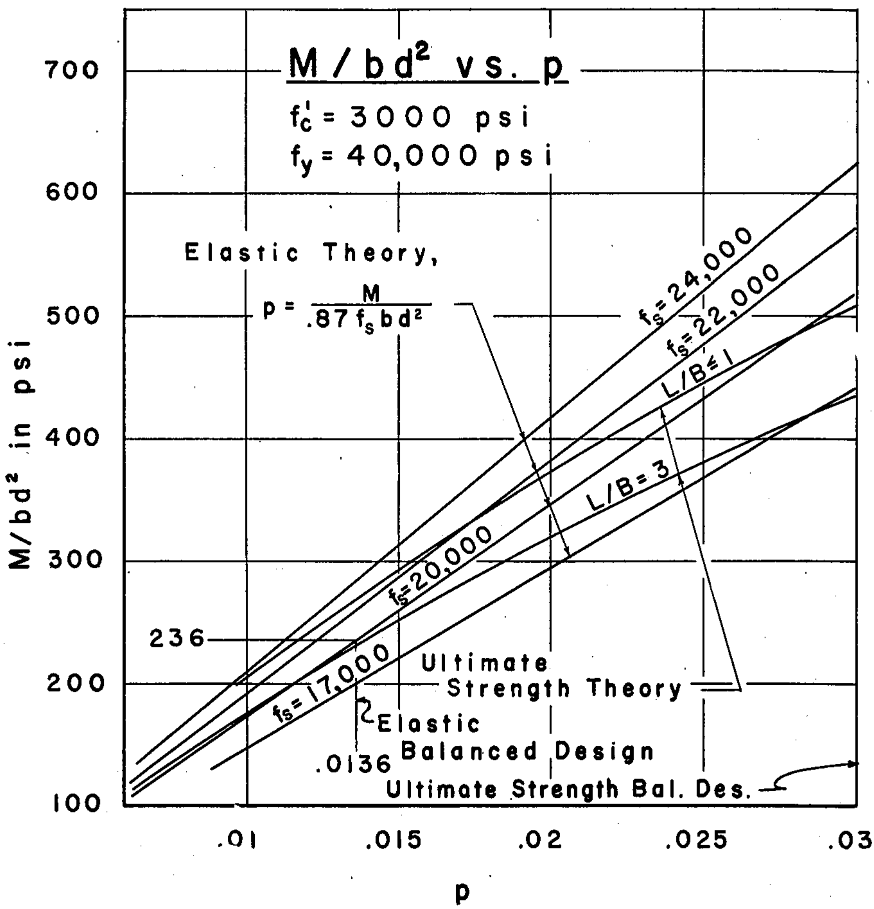

FIG. 2.

the smaller ratios of L/B some increase over the currently used 20,000 psi working stress is allowed even when M/bd^2 is greater than 236.

Figure 3 gives the corresponding curves for hard grade steel. It is at once apparent that in contrast to the ACI elastic theory, ultimate strength design permits full advantage to be taken of the greater yield strength of hard grade steel. The ultimate strength curves lie everywhere above the $f_s = 20{,}000$ psi line, and for the usual values of M/bd^2, the effective steel working stress is between 23,000 and 29,000 psi.

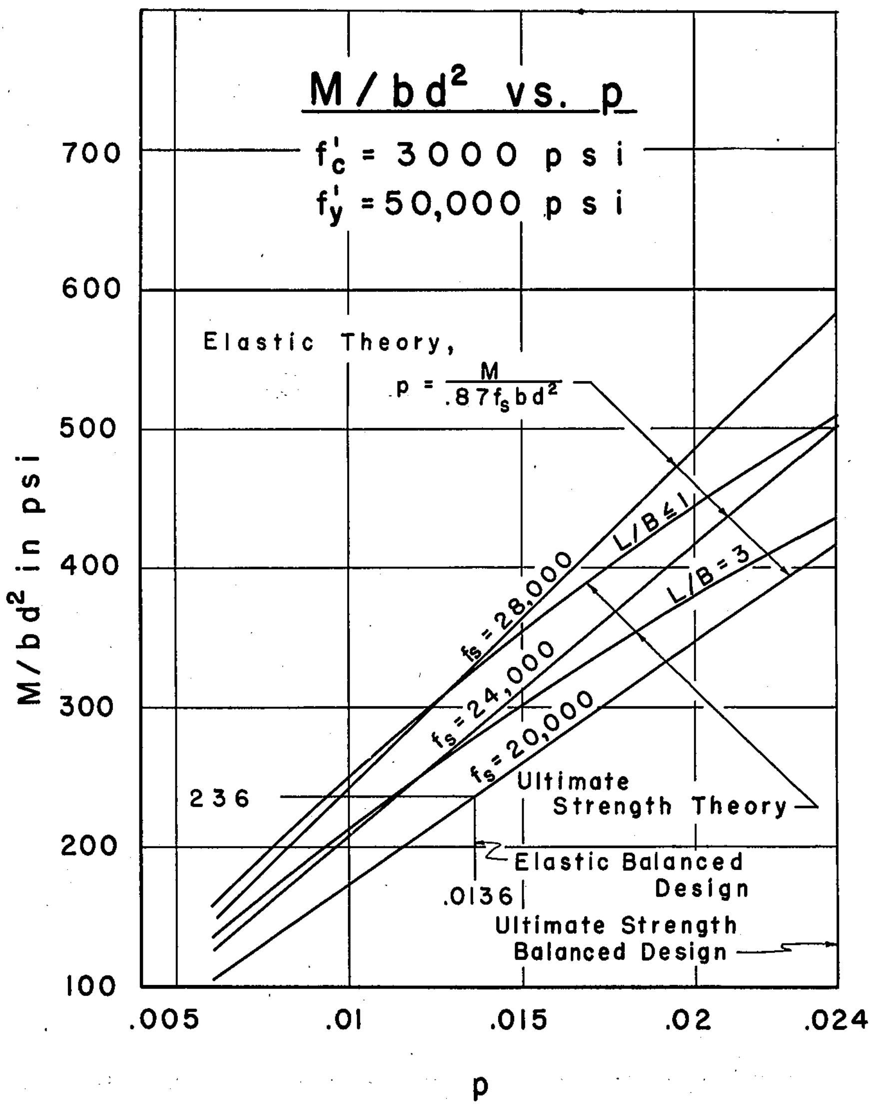

FIG. 3.

It can therefore be concluded that for a beam of a given depth, the ultimate strength theory usually will require less steel area and beam width than will the elastic theory, particularly if hard grade steel is used. Of course, it may not always be possible or economical to take maximum advantage of the permitted reduction in beam width, because of possible difficulties with steel placement and diagonal tension, etc., and because the steel area required by the ultimate strength theory increases as the beam width is decreased.

D. *Recommended Precautions*

The use of steel and concrete working stresses considerably larger than those with which we have built up a considerable backlog of experience requires increased care in design and construction. Some of the factors requiring particular attention are discussed below.

1. *Concrete Strength*

The effect of a downward deviation in concrete strength is potentially considerably more serious with the increased concrete working stresses which are, in effect, permitted by the ultimate strength method. This fact is recognized by the comparatively severe control requirements of paragraph A602(f) in the Appendix to the Code:

"Controlled concrete should be used and shall meet the following requirements. The quality of concrete shall be such that not more than one test in ten shall have an average strength less than the strength assumed in the design, and the average of any three consecutive tests shall not be less than the assumed design strength. Each test shall consist of not less than three standard cylinders."

2. *Buckling of Compressive Steel*

Steel located in the compressive face of a beam, even though not necessary for or considered in the design, can buckle at moments less than the calculated ultimate, thus disrupting and weakening the member. Such steel should be adequately restrained by stirrups or ties.

3. *Diagonal Tension and Bond*

No recommendations are given for the ultimate strength theory of diagonal tension and bond; it is assumed that for the present the customary elastic theory will be employed on an interim basis. But the computed ultimate bending moment usually cannot be developed without considerable yielding and plastic flow, during which the diagonal tension and bond strength must be enough to maintain the integrity of the section. Because of this, and because of the relative paucity of test data supporting the use of the customary allowable diagonal tension and bond stresses in beams with large steel percentages and working stresses, particular care must be exercised. Whitney and Cohen[3] recommend the following precautions in addition to the more severe diagonal tension and bond provisions of the new Code:

"Web reinforcement shall be provided from the support to a point beyond the extreme position of the point of inflection a distance equal to either 1/16 of the clear span or the depth of the member whichever is greater even though the shearing stress does not exceed v_c. . . . Where required by this paragraph, the amount of web reinforcement at each section shall be [at least]:

1. Sufficient to carry 2/3 of the total shear where the unit stress exceeds v_c.

2. Sufficient to carry 2/3 of the total shear existing at the point of inflection, that is, the ratio of web reinforcement required at the point of inflection will be maintained back to the support . . ."

"This addition is not intended to apply to small T-beams forming part of a joist floor construction. The use of bent-up bars for diagonal tension reinforcement is desirable and should be used where practical."

In order to resist longitudinal tension due to volume changes Whitney and Cohen also recommend that at least half of the positive reinforcement lap with at least one-third of the negative reinforcement on the opposite face for a distance not less than the depth of the beam.

If high tensile steel and bond stresses exist simultaneously at working loads, there is a possibility that bond creep may eventually cause an objectionable opening up of transverse cracks in the concrete adjoining the tensile steel. Until a sufficient backlog of experience is built up, it would be wise to avoid a combination of high tensile and bond stresses at working loads.

4. *Deflections*

The use of smaller sections means more instances where deflection must be checked. Furthermore, the usual assumptions made for deflection calculations may not be sufficiently accurate. Elastic deflections are usually calculated on the basis of the gross moment of inertia of the section; because of the increased number of concrete cracks which exist at higher steel stresses (even though they may be almost invisible to the naked eye) the use of the gross moment of

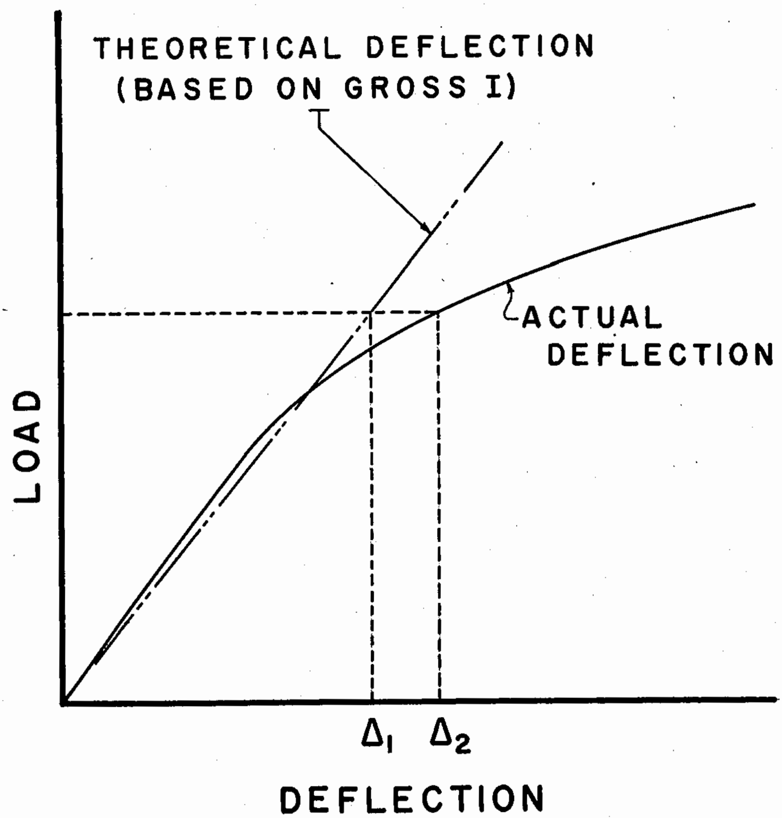

FIG. 4.

inertia may be considerably in error. For example, in Figure 4, the actual deflection $\triangle_2$ is much larger than the calculated deflection, $\triangle_1$. Also, the usual allowances[4] for creep may be too small for sections with high concrete working stresses.

5. *Elastic and Plastic Flexibility*

Most structures are subject to some differential settlement. This settlement, while not usually considered in design, is sometimes of sufficient magnitude to develop considerable plastic deformation (yield hinges). According to the principles of limit design, members which can sustain this plastic deformation without failure suffer no reduction in their ability to sustain gravity loads. But smaller, heavily reinforced beams, while more flexible elastically, cannot sustain as much plastic rotation before failure.[5] Therefore high percentages of reinforcement should be used only in structures on good foundations, or where a careful evaluation is made of the possible magnitudes of the differential settlements and their effects. In the latter case, the section rotation angle at ultimate moment must be calculated[5,6,7] and compared with the requirements of the particular application.

Another undesirable characteristic of beams with high percentages of reinforcement is the possibility that they may give insufficient warning prior to bending failure. Large deflections under overloads are desirable; heavily reinforced beams in general have smaller section rotation angles at ultimate moment and therefore less deflection prior to failure.

6. *Fatigue*

Although this subject is not discussed in the Code, it is important that suitable limits be placed on the stresses at working loads in members subject to fatigue loading, such as elevator machinery supports.

E. *T-Beams*

True T-beams will occur only very rarely if the ultimate strength design method is used, since there ordinarily will not be enough room for sufficient steel to bring the neutral axis at failure into the stem. Formulas for T-beams are presented in Section A607 of the Appendix.

Limit Design

The Code does not as yet permit the use of limit design, which is the taking into account of the redistribution of bending moments and forces in a structure as failure approaches. Section A601(b) states:

"It is assumed that external moments and forces acting in a structure will be determined by the theory of elastic frames."

Shear (Diagonal Tension) and Bond

There have been a number of failures recently in certain rigid frame structures[8,9] which were apparently designed in accordance with the ACI Code. These failures occurred near the point of inflection under shears less than that permitted by the concrete alone. There is still some doubt concerning the exact cause of these failures, although they were apparently due to tensile stress caused by volume changes in restrained members. The Portland Cement Association has undertaken a test program which promises to throw additional light on this matter. Meanwhile, however, certain additions and changes have been effected in the shear and bond provisions which, it is believed, will eliminate this type of failure in the future.

Section 801(d) requires that when the shearing stress exceeds that permitted for the concrete alone, web reinforcement shall be provided for a distance equal to the depth, d, of the member beyond the point theoretically required.

Section 801(e) states:

"Where continuous or restrained beams or frames do not have a slab so cast as to provide T-beam action, the following provisions shall apply. Web reinforcement shall be provided from the support to a point beyond the extreme position of the point of inflection a distance equal to either 1/16 of the clear span or the depth of the member, whichever is greater, even though the shearing unit stress does not exceed v_c. Such reinforcement shall be designed to carry at least two-thirds of the total shear at the section. Web reinforcement shall be provided sufficient to carry at least two-thirds of the total shear at a section in which there is negative reinforcement."

Section 807 requires that when web reinforcement is necessary, the amount shall be not less than 0.15 percent of the spacing multiplied by the beam width. This is equivalent to saying that when the shearing stress exceeds the amount that can be taken by the concrete, the assumed excess shear must not be less than 30 psi.

The following statement has been added to Section 902(a):

"At least one-third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance sufficient to develop by bond one-half the allowable stress in such bars, not less than 1/16 of the clear span length, or not less than the depth of the member, whichever is greater."

Flat Slabs

Chapter 10 on Flat Slabs has been completely rewritten. Some of the more significant changes are discussed herewith.

Design by Elastic Analysis

A. *Calculation of Design Moments*

The 1951 Code permitted the design negative moment to be taken at a distance .073 L + .57 A from the column centerline, where A was usually one-half the column capital diameter. This provision sometimes resulted in too low a design moment for flat plate floors. The new Code states that the critical negative moment should be computed at a distance A from the support centerline, where A has been redefined so as to be usually equivalent to one-half the column capital diameter plus the depth of the drop panel (if any) plus one-half the slab thickness. The new provision is approximately equivalent to the old when the capital diameter is .225 L, the drop panel thickness L/80, and the slab thickness L/40, but gives more conservative results for shallower or no drop panels, or when smaller or no column capitals are used.

In the 1951 Code, a formula was given for the required minimum sum, M_0, of the maximum positive and average maximum negative bending moments. This formula gave larger values than the formula specified for the empirical method of design. Because of the more conservative assumption for the location of the critical sections, there is no longer a minimum requirement for M_0 when the elastic analysis method is used.

B. *Apportionment of Moments*

A new table is provided which gives the percentage of total moment at any section to be distributed to the column and mid strips. No distinction is drawn between slabs with and without drop panels.

Design by Empirical Method

A. *Limitations*

The maximum variation in successive span lengths has been changed from 20 percent of the shorter span to 20 percent of the longer span. Also, columns can now be offset as much as 10 percent of the span in the direction of the offset.

B. *Minimum Slab Thickness*

In Section 1004(d) two rather involved equations are now given for the minimum thickness of slabs with and without drop panels. These equations were each derived from a strength calculation for a slab of certain assumed minimum proportions. M_0 was calculated from the empirical equation given in Section 1004(f) (discussed below). The design coefficients were taken as .50 for the slab without drops (which is the tabulated coefficient at the first interior column) and .38 at the edge of the drop for the slab with drop panels. The latter coefficient is an average value obtained from studies, based on the tabulated coefficient of .56 for the moment at the centerline of the first interior column.

C. *Calculation of M_0*

The revised equation for M_0,

$$M_0 = 0.09\ WLF \left[1 - \frac{2c}{3L} \right]^2$$

differs from the old one in the inclusion of the factor F, which is equal to

$$1.15 - c/L,$$

where c is usually equal to the column capital diameter, or to the column diameter if capitals are not used. The purpose of this provision is to provide additional protection when the column capitals are small or absent.

D. *Empirical Coefficients*

The table of coefficients for empirical design has been considerably changed and enlarged for exterior panels and panels continuous across beams or walls.

E. *Reinforcement*

Tables and diagrams are now provided, giving details of steel lengths, bends and cut-offs. This information was calculated from elastic studies, and makes a very useful addition to this chapter.

F. *Columns for Flat Slabs*

An assumption implicit in the empirical equation for M_0 is that the columns can be depended upon for a certain resistance to rotation. Unless sufficient column stiffness and strength are provided, use of the empirical coefficients may prove unsafe, particularly for large ratios of live to dead load. The new Code formally acknowledges this in the following provisions, which are based on extensive studies:

"1. The minimum dimension of any column shall be 10 in. For columns or other supports of a flat slab, the required minimum average moment of inertia, I_c, of the gross concrete section of the columns above and below the slab shall be determined from the following formula, and shall be not less than 1000 in.[4] If there is no column above the slab, the I_c of the column below shall be twice that given by the formula with a minimum of 1000 in.[4]

$$I_c = \frac{t^3H}{0.5 + \frac{W_D}{W_L}}\text{"}$$

(W_D = total dead load on panel; W_L = total live load on panel.)

"2. Columns supporting flat slabs designed by the empirical method shall be proportioned for the bending moments developed by unequally loaded panels, or uneven spacing of columns. Such bending moment shall be the maximum value derived from

$$(WL_1 - W_DL_2)\frac{1}{f}$$

L_1 and L_2 being lengths of the adjacent spans ($L_2 = 0$ when considing an exterior column) and f is 30 for exterior and 40 for interior columns. (W = total dead and live load on panel.)

"This moment shall be divided between the columns immediately above and below the floor or roof line under consideration in direct proportion to their stiffness and shall be applied without further reduction to the critical sections of the columns."

Columns

Elastic Theory

A. *Uncracked Sections*

ACI 318-51 allowed columns for which the eccentricity ratio e/t is equal to or less than unity to be designed on the basis of the uncracked section. This sometimes resulted in a dangerously low factor of safety in cases where e/t was nearly one. The revised Code has reduced this limiting value of e/t to 2/3.

The formulas for the design of eccentrically loaded uncracked sections have been replaced by an interaction equation similar in form to that used for the design of steel columns:

$$f_a/F_a + f_b/F_b = 1$$

where f_a = nominal axial unit stress
F_a = nominal allowable unit stress
f_b = bending unit stress (bending moment divided by transformed section modulus)
F_b = allowable stress for pure bending

This equation gives results identical to those obtained from the old equations. In order to illustrate the proper application of the new

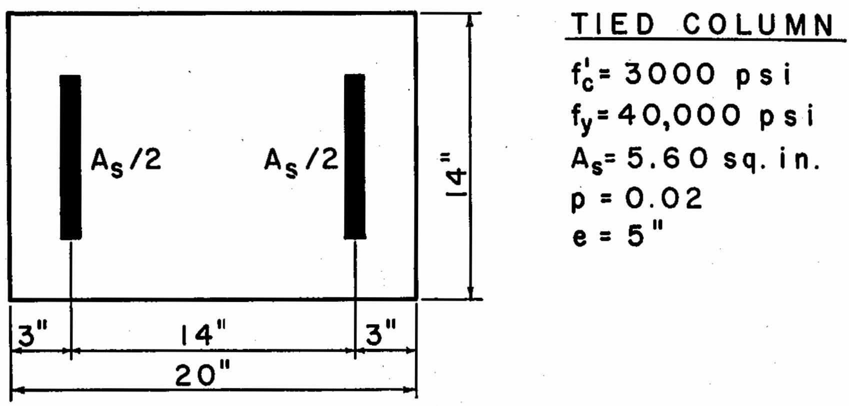

FIG. 5.

formula, the allowable capacity of the section shown in Figure 5 is calculated below.

$$f_a = N/A_g = N/20 \times 14 = \mathit{N/280}$$
$$F_a = .8A_g\ (.225\ f'_c + 16{,}000\ p)/A_g$$
$$= 18\ f'_c + 12{,}800\ p = \mathit{796\ psi}$$
$$I = bt^3/12 + (n\text{–}1)\ A_s\ (14/2)^2 = \mathit{11{,}800\ in.^4}$$
$$S = 2I/t = 11{,}800/10 = \mathit{1180\ in.^3}$$
$$f_b = M/S = \mathit{M/1180}$$
$$F_b = .45\ f'_c = \mathit{1350\ psi}$$
$$f_a/F_a + f_b/F_b = N/280 \times 796 + M/1180 \times 1350$$
$$N/223 + 5N/1592 = 1 \quad (\text{N in kips})$$
$$\mathit{N = 131\ kips}$$

B. *Cracked Sections*

For the purpose of taking into account the effect of plastic flow as ultimate load is approached (as is done in the elastic design of beams with compression steel), the new Code permits the modular ratio for the compressive reinforcement to be assumed at double the value given in Section 601. It is important to note that this value of modular ratio is to be used when computing the depth of the compressive stress block, as well as when taking the statical summation of the forces acting on the cross-section; otherwise, serious errors may result.

The allowable compressive stress in the concrete is given as .45 f'_c, instead of as a function of eccentricity. This simplifies calculations, without encroaching unduly on the factor of safety.

Ultimate Strength Theory

The ultimate strength theory offers a far more rational basis for column design than does the elastic theory. This has been reflected, in part, by the axially loaded column equation which has been in the Code for many years. The equations and load factors given in the Appendix now make it possible to design all columns with suitable factors of safety based on their ultimate strengths. The use of proper combinations of load factors, as provided in Section A604 of the Appendix, is especially important for columns, for which the critical loading condition is not always that in which all loads are at their maximum values.

The ultimate strength equations, while quite complex in appearance, are readily solved with the aid of published graphs.[3] In consideration of the unavoidable accidental eccentricities always present in columns, axially loaded columns are designed as eccentrically loaded columns with eccentricities of .05 times the depth for spirally reinforced columns, and .10 times the depth for tied columns.

Figure 6 compares, for a typical column, the ultimate load with the allowable loads by the elastic theory and by the ultimate strength theory. The elastic theory appears to provide an excessive factor of safety at low values of moment, and a dangerously low value as pure bending is approached. (The low value is somewhat fictitious, however, since for this particular section, the ultimate strength equation gives overly conservative results at low values of P.)

The discontinuity in the elastic theory curve in Figure 6 is due to the change in assumptions at $e/t = 2/3$. In this case, the permissible increase in load due to the doubling of the compression steel modular ratio exceeds the decrease due to the cracked section assumption. This sudden change in capacity as e/t if increased may be either negative or positive, depending upon the section dimensions. In the case of columns with high steel ratios, the change is frequently an increase surprisingly large in magnitude.

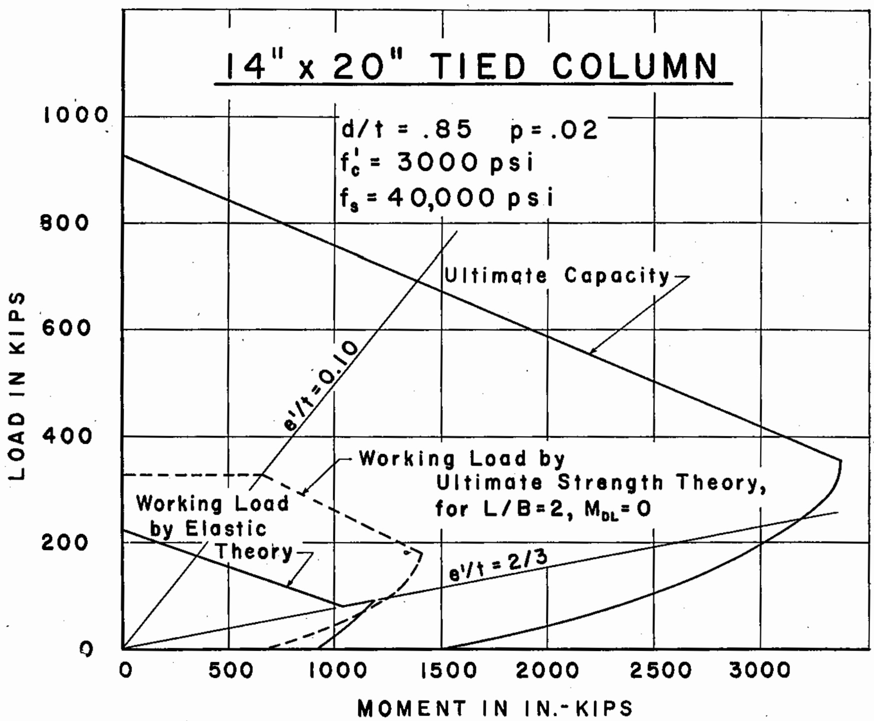

FIG. 6.

PIPE COLUMNS

The formula given in the 1951 Code for columns consisting of steel pipe filled with concrete sometimes gave a lower allowable load than that obtained by omitting the concrete and using the column formulas specified by the American Institute of Steel Construction. This formula has been replaced by one based on an investigation sponsored by the Housing and Home Finance Agency, the report of which appears in Reference 10.

Conclusion

The 1956 Code contains a number of important changes, and represents a major step forward in concrete design specifications. Nevertheless, there is still work to be done, particularly in connection with diagonal tension and shear.

The most significant of the new revisions is the provision permitting ultimate strength design. The use of this method will give designs which have more rational and consistent factors of safety. Considerable changes in section proportioning can result. There will be an increased demand for hard grade steel for beams.

Engineers employing ultimate strength design must proceed with caution. Here experience and good judgment will be more important than ever. Increased attention must be given to concrete quality, deflections at working loads, diagonal tension and bond, the possibility of fatigue loading or stress reversals, and foundation conditions. Greater consideration will have to be given to the economics of section proportioning, since the balanced design section will rarely be economical.

References

1. "Building Code Requirements for Reinforced Concrete (ACI 318-56)."
2. "Report of the ASCE-ACI Joint Committee on Ultimate Strength Design," Proceedings ASCE, Vol. 81, Paper 809, October 1955.
3. "Guide for Ultimate Strength Design of Reinforced Concrete," by C. S. Whitney and Edward Cohen, Journal of the American Concrete Institute, Vol. 28, No. 5, November 1956.
4. "Deflection of Reinforced Concrete Members," Publication No. ST70, Portland Cement Association, Chicago, 1947.
5. "The Behavior of Under-Reinforced Concrete Beams Under Long-Term Loads," by Herbert A. Sawyer, Jr., Publication No. 12, The University of Connecticut Engineering Experiment Station, September 1956.
6. "The Ultimate-Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames," by A. L. L. Baker, Concrete Publications Limited, London, 1956.
7. "Effect of Far-Advanced Compressive Strains of Concrete in Reinforced Concrete Beams Submitted to Bending Moments," (in Swedish) by Sven Sahlin, Bulletin No. 17, The Division of Building Statics and Structural Engineering, The Royal Institute of Technology, Stockholm, Sweden, 1955.
8. Engineering News-Record, August 25, 1955, p. 23.
9. Engineering News-Record, November 10, 1955, p. 21.
10. "Structural Properties of Light-Gage Tubular Columns," Housing Research Paper No. 21, Housing and Home Finance Agency, Washington, D. C., October 1953.

GENERAL RICHARD KING HALE

1880-1956

General Hale was born in Boston on March 17, 1880, the son of Joseph Little and Annie Skinner Hale. He graduated from Harvard College in 1902 and continued his professional training at Massachusetts Institute of Technology, graduating in 1904. During the next twelve years he engaged in private engineering practice in Boston with Metcalf & Eddy and other firms and later in partnership with Mr. Edward B. Richardson.

General Hale married Mary Dean Pierce. They had two daughters, Mary D. and Joanna M. Hale. They lived in Brookline, with a summer home at Dennis.

General Hale's military career started when he entered the Massachusetts National Guard in Artillery Battery A, where he was commissioned a Second Lieutenant in 1908, Captain in 1913, Major and Lieutenant Colonel in 1916, when he entered active service with the U. S. Army on the Mexican Border. In the First World War he served as Colonel with the 101st Field Artillery in France on the staff of the Second Army Corps and returned as Chief of Staff of the 26th Division. Colonel Hale was decorated with the distinguished service medal. After the war his military interests continued in the National Guard, in which he served as Brigadier General from 1922 to 1929.

On his return from France, he began his long and distinguished career for the Commonwealth, when he was appointed by the Governor as one of two Associate Commissioners in the Waterways Division of the Massachusetts Department of Public Works. He continued in that office until a reorganization of the Department in 1927 when he was appointed one of the Associate Commissioners, with jurisdiction over both Waterways and Highways. Later, in 1938, upon the reestablishment of the Division of Waterways in the Department, he was appointed Director of that Division, which office he held up to his retirement in 1955.

For some years General Hale was also a member of the State Reclamation Board, engaged in its endeavors to control and reduce mosquito breeding grounds. He was for many years also a member of the Beach Erosion Board. This Board was appointed by the Chief of Engineers in the Corps of Engineers, and consisted of seven members, four of which were Engineer Officers and three Engineers of State Agencies, for devising effective means of preventing erosion of the shores of coastal and lake waters by waves and currents. Under certain conditions it made investigations and recommendations, which involved local contributions to cost of projects.

It is in his thirty years as Director of the Waterways Division that General Hale will be best remembered. This was an administrative position, dealing with engineering matters. The Division's jurisdiction over tidal lands of the Commonwealth included the development of harbors, the Port of Boston until 1945, State Piers of Gloucester, New Bedford, Fall River, Plymouth and Bourne and all shore protective works and dredging channels. Activities of the Division also included improvement of rivers and streams for the purpose of protection against floods, for stream clearance, channel improvement or any form of flood control or prevention work, and licensing of dams, roads or bridges or other structures or other work in streams where public funds had been spent for flood control or prevention work were included under his jurisdiction.

Starting when the Commonwealth undertook the building of the great Logan Airport at East Boston, General Hale was active in the design and construction of that project. His main desire was that sound judgment should determine the fundamental layout or pattern for the airport and the conception of the essential features and their magnitudes and general methods of construction, including the initial reclamation of tidal flats of the Commonwealth by the use of local material for the basic fill, and that the runway construction and building layout should be properly designed for a great airport.

General Hale's great knowledge and experience in all matters of hydraulics of rivers and harbors, beach and shore made a great contribution to the solution of many problems that came to the Division. His work included not only the general engineering work but many dealings with engineers, local, state and Federal authorities, legislators, committees, and the public.

Those who came in contact with General Hale must have been impressed by his personality, his open frankness and fair dealing, his integrity and also a certain serenity, which while it did not prevent a strong stand and effort for what he thought was right, did not become ruffled at opposition. His was a great contribution to the service of Massachusetts.

General Hale's interest in the activities of the Engineering Societies has included membership in the American Society of Civil Engineers and the New England Water Works Association for which he served as Editor of its Journal for eight years. He became a member of the Boston Society of Civil Engineers in 1909, served in various offices, and became President in 1925, and was elected an honorary member of the Society in 1950.

Of General Interest

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

JANUARY 23, 1957.—A Joint Meeting of the Boston Society of Civil Engineers and the Structural 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 December 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.

The Secretary announced the names of applicants for Membership in the BSCE, and that the following had been elected to membership on January 21, 1957.

Grade of Member—William E. Brooks, Robert A. Carleo, Alexander M. Gibson, Ross E. McKinney, Stiles F. Stevens.

Grade of Junior—Peter S. Parsonson, John D. Goodrich.

Grade of Student—Alfred W. Hoadley.

President Thomas requested the Secretary to present a recommendation of the Board of Government to the Society for action. The President stated that this matter was before the Society in accordance with the provisions of the By-Laws and notice of such action published in the ESNE JOURNAL dated January 14, 1957.

The Secretary presented the following recommendation of the Board of Government to the Society for initial action to be taken at this meeting.

MOTION "to recommend to the Society that the Board of Government be authorized to transfer an amount not to exceed $1800 from the Principal of the Permanent Fund to the Current Fund for current expenditures".

On motion duly made and seconded it was *VOTED* "that the Board of Government be authorized to transfer an amount not to exceed $1800 from the Principal of the Permanent Fund to the Current Fund for current expenditures".

President Thomas stated that final action on this matter would be taken at the February 20, 1957 meeting of the Society.

President Thomas announced that this was a Joint Meeting with the Structural Section and called upon Augustine L. Delaney, Chairman of that Section to conduct any necessary business at this time.

President Thomas then introduced the speakers of the evening,

Robert J. Basso, Corps of Engineers, U. S. Army; Frank L. Lincoln, Partner, Fay, Spofford & Thorndike; Joseph Peraino, Chief Engr., Merritt-Chapman & Scott.

Subject—"Renovations to South Boston Army Base".

A short discussion period followed the talk.

Fifty one members and guests attended the dinner and eighty six attended the meeting.

A rising votes of thanks was given the speakers.

The meeting adjourned at 9:00 P.M.

ROBERT W. MOIR, *Secretary*

FEBRUARY 20, 1957.—A regular meeting of the Boston Society of Civil Engineers 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 7:10 P.M.

President Thomas stated that the minutes of the previous meeting held on January 23, 1957 would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes would be waived unless there was objection.

The Secretary announced the names of applicants for Membership in the BSCE, and that the following had been elected to membership on February 18, 1957.

Grade of Member—Harold Bateson, George Bradley, Daniel Collins, James A. Fife,* Louis Katona, Norman E. Jackson, Thomas A. LaCava, Francis J. Lariviere, Robert H. Lubker, John D. Marr, Jr., John C. Matte, Robert W. Muther, Jr., Gerald S. Parker, Chester C. Pease, Jr., Maynard D. Spekin, Joseph Sideransky, William C. Vose, Robert L. Yocum.

Junior Member—Arthur E. Sullivan.

Student Member—Govindaraju J. Mohanrao.

President Thomas requested the Secretary to present a recommendation of the Board of Government to the Society for action. The President stated that this matter was before the Society in accordance with the provisions of the By-Laws and notice of such action was published in the ESNE JOURNAL dated February 4, 1957.

*Transfer from Grade of Junior.

The Secretary presented the following recommendation of the Board of Government to the Society for final action to be taken at this meeting.

MOTION—"to recommend to the Society that the Board of Government be authorized to transfer an amount not to exceed $1800 from the Principal of the Permanent Fund to the Current Fund for current expenditures".

On motion duly made and seconded it was *VOTED* "that the Board of Government be authorized to transfer an amount not to exceed $1800 from the Principal of the Permanent Fund to the Current Fund for current expenditures".

President Thomas stated that this was the final action on this matter.

President Thomas then introduced the speaker of the evening, Mr. John Clarkeson, of Clarkeson Engineering Company who gave a most interesting paper on "Use of Photogrammetry in Highway Engineering".

A brief discussion period followed after which a collation was served.

Eighty nine members and guests were present at this meeting.

The meeting adjourned at 8:15 P.M.

ROBERT W. MOIR, *Secretary*

MARCH 20, 1957.—The one hundred ninth annual meeting of the Boston Society of Civil Engineers was held today at the Hotel Vendome, 160 Commonwealth Avenue, Boston, Mass., and was called to order at 4:15 P.M., by President John G. W. Thomas.

President Thomas announced that the reading of the minutes of the Society meetings had been omitted during the year. The minutes of the January and February 1957 meetings will be published in a forthcoming issue of the JOURNAL. The minutes of the April, May, October, November and December 1956 meetings to be declared approved as published.

It was *VOTED* "to approve the minutes as published".

President Thomas announced the death of the following member:—

Irving E. Moultrop, who was elected a member December 18, 1895 and who died February 24, 1957.

The Secretary announced the following had been elected to membership:—

Grade of Member — Clifford E. Adams, Adin B. Bailey, Leonard J. Edel, Robert G. Esterberg,* Walter R. Ferris, Charles D. Hopkins, Jr., Robert F. Pelletiere, Arthur L. Quaglieri.*

Grade of Junior Member—Gino N. Cosimini,# John W. Cossart.

The Secretary also announced the names of applicants for membership in the Society.

The Annual Reports of the Board of Government, Treasurer, Secretary and Auditors were presented. Reports were also made by the following Committees: Hospitality, Library, John R. Freeman, Membership, Advertising, Publicity, and Joint Committee on Legislative Affairs.

It was *VOTED* "that these reports be accepted and placed on file".

The Annual Reports of the various Sections were read and it was *VOTED* "that the Annual Reports of the various Sections be accepted and placed on file".

President Thomas stated that all foregoing reports would be published in the April issue of the JOURNAL.

The Report of the Tellers of Election, George M. Reece and Charles M. Anderson, presented and in accordance therewith the President declared the following had been elected officers for the ensuing year.

President — Arthur Casagrande

V.-President (for two years) — Frank L. Flood

Secretary (for one year) — Robert W. Moir

Treasurer (for one year) — Charles O. Baird, Jr.

Directors (for two years) — Arthur T. Ippen, Kenneth F. Knowlton

Nominating Committee (for two years) — John C. Rundlett, George W. Hankinson, Frank L. Heaney

The retiring President, John G. W. Thomas, then gave his address entitled "The Proprietors of the Locks and Canals on Merrimack River".

Thirty-eight members and guests attended the business meeting.

The meeting adjourned to assemble at 8:00 P.M., the Annual Dinner being held during the interim.

The President called the meeting to order at 8:05 P.M.

Following general remarks and the introduction of the newly elected President, Arthur Casagrande, and other guests at the head table the various prize awards were made.

The Secretary read the various prize awards and asked the recipient to come forward and President Thomas presented the following awards:—

Award	*Recipient*	*Paper*
Desmond FitzGerald Medal	T. William Lambe	"The Storage of Oil in an Earth Reservoir".
Clemens Herschel Award	Jacob Feld	"Structural Failures and How to Avoid Them".
Sanitary Section Award	John S. Bethel, Jr.	"Comprehensive Sanitary Survey of Woonsocket, Rhode Island".
Surveying & Mapping Section Award	Charles L. Miller	"Some New Conceptions and Old Misconceptions of Photogrammetry".

*Transfer from Grade of Junior.
#Transfer from Grade of Student.

President Thomas then introduced the speaker of the evening, Mr. George F. Potier, who gave a most interesting illustrated talk on "Europe is a State of Mind".

At the conclusion of the address President Thomas on behalf of the Society thanked Mr. Potier for a most enjoyable talk and then turned the meeting over to President-elect Arthur Casagrande.

President-elect Arthur Casagrande presented retiring President John G. W. Thomas with a certificate for services rendered and then adjourned the meeting.

One hundred thirty-three members and guests attended the dinner meeting.

ROBERT W. MOIR, *Secretary*

STRUCTURAL SECTION

JANUARY 23, 1957.—A joint meeting of the Boston Society of Civil Engineers and the Structural Section of the BSCE was held in the Adams Room of the United Community Service Building on this date. Dinner was served at 6:00 P.M., after which the meeting was called to order. After transacting necessary business of the Society, President Thomas introduced the speakers: Mr. Robert J. Basso of the Corps of Engineers, Mr. Frank L. Lincoln of Fay, Spofford & Thorndike, Inc., and Mr. Joseph Peraino of Merritt-Chapman and Scott Corp., all of whom spoke on "Renovations to the South Boston Army Base".

Mr. Basso, the first speaker, gave a brief history of the construction of the Army Base, followed by a more detailed account of the deterioration that occurred and the corrective construction done in the period from the original construction to the present reconstruction. He also outlined the events leading to a decision to renovate the Base.

Mr. Lincoln, the second speaker, outlined the various possible methods of rebuilding that were investigated and the reasons why each was discarded or retained for more complete study. The four most practical methods were covered in detail and the reasons for the selection of the actual method used were discussed.

Mr. Peraino, the final speaker, covered the construction method developed to carry out the work. The magnitude of the project was such that considerable study of the various construction problems was justified. The special construction methods used were outlined in some detail as was the scheduling and carrying out of the actual work.

The talks were illustrated by slides showing details of construction and the actual construction. At the conclusion of the talks, the speakers answered questions by the members of the audience.

Fifty-one members and guests were present at the dinner and eighty-six members and guests attended the meeting.

The meeting adjourned at 9:00 P.M.

RICHARD W. ALBRECHT, *Clerk*

FEBRUARY 13, 1957.—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. After reading the minutes of the previous Structural Section meeting, a motion was passed under which the Chairman appointed a Nominating Committee to nominate a slate of officers for the Structural Section for the coming year. The committee appointed is as follows:—Charles H. Norris, Ruth D. Terazaghi, Casimir J. Kray.

At the conclusion of the business meeting, Chairman Delaney introduced the speaker, Mr. Nelson Aldrich, AIA of Cambell and Aldrich, who spoke on the subject "Interdependence of the Engineer and Architect with some Observations on the Aesthetics of Structures".

Mr. Aldrich stressed the need for

preliminary planning by the architect and the engineer in order to obtain an aesthetically pleasing structure that is functionally proper. During the course of the talk, the principles of several schools of architectural thought were discussed, slides being used to illustrate various points. The necessity for the architect and the engineers to act as a team, in order to obtain the most satisfactory structure, was stressed. At the conclusion of the formal presentation, the subject was discussed generally, with the members of the audience participating.

Forty members and guests attended this meeting.

The meeting adjourned at 8:15 P.M.

RICHARD W. ALBRECHT, *Clerk*

MARCH 13, 1957.—A meeting of the Structural Section was held in the Boston City Club on this date. The meeting was called to order at 7:15 P.M. by Chairman A. L. Delaney. After the reading of the minutes of the previous meeting, the report of the nominating committee was presented by Mr. Casimir J. Kray. The following officers were nominated by the committee and were elected for the coming year:

Chairman	John M. Biggs
Vice Chairman	Richard W. Albrecht
Clerk	William A. Henderson
Executive Committee	
	Paul S. Crandall
	Myle J. Holley, Jr.
	John C. Rundlett

At the conclusion of the business meeting the speaker, Dr. Laurits Bjerrum, Director of the Norwegian Geotechnical Institute, Visiting Professor at the Massachusetts Institute of Technology, was introduced by Chairman Delaney.

Dr. Bjerrum presented an illustrated paper on "Norwegian Experience with Foundations on Steel Piles." The similarity of subsoil conditions in Oslo, Norway with those encountered in Boston was discussed. The clays encountered are similar except that in Oslo the clay is much softer. The special pile point and driving method used in Norway to anchor steel piles on sloping rock surfaces was outlined. The speaker presented and discussed data concerning the curvature of long piles after driving, buckling loads, corrosion, displacement of clay and settlement.

At the conclusion of the talk various questions were discussed by the speaker and the members of the audience.

Seventy-two members and guests attended this meeting.

The meeting adjourned at 8:45 P.M.

RICHARD W. ALBRECHT, *Clerk*

SANITARY SECTION

MARCH 6, 1957.—The meeting was called to order at 7:15 P.M. by Vice Chairman John F. Flaherty, acting in the absence of Chairman Darrell A. Root. The meeting was preceded by an informal dinner at Patten's Restaurant. Fifty-two members and guests attend-the meeting and twenty attended the dinner.

The Annual Report of the Executive Committee was read by the Clerk of the Section, and the report was accepted as read.

The report of the Nominating Committee was read by Mr. Fozi M. Cahaly. The following people were submitted for consideration as officers and Executive Committee members for the coming year.

Chairman	John F. Flaherty
Vice-Chairman	Clair N. Sawyer
Clerk	Joseph C. Knox
Members	Harold A. Thomas, Jr.
	George M. Reece
	James L. Dallas

The report of the Nominating Committee was accepted. The nominations were closed by vote and the Clerk cast one ballot for the nominees as presented.

Announcement was made of the vote of the Executive Committee to accept an invitation of the New England Sewage and Industrial Wastes Association to hold our Spring Outing by meeting with them at Newport, Rhode Island, on June 6, 1957.

The speaker of the evening was Dr. Conrad P. Straub of the U. S. Public Health Service who gave a very informative talk on the "Disposal of Atomic Power Plant Wastes". The speaker reviewed the details of the proposed "Yankee Power Plant" to be constructed in Western Massachusetts, giving reasons for the site selection and an insight to disposal problems which are apt to occur.

After a short discussion, the meeting was adjourned at 8:55 P.M.

CLAIR N. SAWYER, *Clerk*

HYDRAULICS SECTION

NOVEMBER 7, 1956.—The meeting was held at the Society Headquarters at Tremont Temple. The meeting was called to order at 7:25 P.M. by Mr. Clyde W. Hubbard, Vice Chairman.

The minutes of the meeting of May 2, 1956 were read and approved.

Professor A. T. Ippen of M.I.T. and a member of the Executive Committee of the Hydraulic Division of the American Society of Civil Engineers announced that a convention of the Hydraulic Division of the ASCE would be held in August 1957 at M.I.T. Professor Ippen invited the Hydraulic Section of the BSCE to act as a co-sponsor of this convention. It was moved, seconded and passed that the Hydraulic Section of the BSCE express its interest and support of the 1957 ASCE convention described by Professor Ippen and agree to act as co-sponsor.

The chairman introduced Professor Leslie Hooper of the Alden Hydraulic Laboratory and a member of the Freeman Committee of the Society. Professor Hooper explained the purpose of the Freeman Fellowships and then introduced Mr. Lawrence Neale, the speaker of the evening who held the BSCE Freeman Fellowship for the year 1955. Mr. Neale spoke on the subject "European Hydraulic Laboratory and Hydraulic Practice in 1955."

As a Freeman Fellow during 1955, Mr. Neale traveled throughout Europe visiting hydraulic laboratories and hydraulic installations. He showed color slides of many of the places he had visited and gave his impressions of the work being done in a variety of areas of hydraulic interest. During the question period which followed, the audience expressed its interest in the details of many of the topics which Mr. Neale had mentioned. On completion of his presentation, the speaker was given a vote of thanks as an appreciation by all those present. The meeting was adjourned at 8:35 P.M. Total attendance was 31.

JAMES W. DAILY, *Clerk*

FEBRUARY 6, 1957.—The meeting which was held at the Society Headquarters at Tremont Temple was called to order at 7:05 P.M. by Mr. Joseph Lawlor, Chairman.

The minutes of the meeting of November 7, 1956 were read and approved.

Mr. Ralph S. Archibald, as Chairman, reported for the Nominating Committee consisting of Arthur T. Ippen, Lincoln W. Ryder and Archibald. The recommended slate of officers for 1957 includes: Chairman, Clyde W. Hubbard; Vice Chairman, James W. Daily; Clerk, Lee G. M. Wolman; Executive Committee, John B. McAleer, Robert S. Kleinschmidt, Lawrence C. Neale.

There being no further nominations, it was moved, seconded, and passed that nominations be closed and the Clerk being instructed to cast one ballot for the above slate of officers.

Mr. Lawlor introduced the speaker, Mr. William Rheingans, Manager of the Hydraulic Department of the Allis-Chalmers Mfg. Co., Milwaukee, Wisconsin. Mr. Rheingans spoke on the two topics "Pump Storage Systems and Pump-Turbines" and "Cavitation." He described the purpose and history of pump storage systems and the use of pump-turbine units in such systems. He then showed a movie entitled "Captive Kilowatts" explaining how the pump storage system works and describing the installation of the generator and turbine units at the Hiwassee TVA Dam. The speaker concluded with a discussion of some new ideas on the velocity effect of cavitation pitting. A long question period followed the author's presentation.

Following the program, the meeting was turned over to the Chairman elect, Mr. Clyde Hubbard. Mr. Hubbard announced plans for holding the May meeting of the Section at the Alden Hydraulic Laboratory at Worcester Polytechnic Institute. On a straw vote, the audience expressed its preference for a Saturday meeting on that occasion.

There being no further business, the meeting was adjourned at 8:45 P.M. Total attendance was 88.

JAMES W. DAILY, *Clerk*

CONSTRUCTION SECTION

JANUARY 30, 1957.—The second meeting was called to order at 7:40 P.M., at the Society Rooms after an informal dinner at Pattens' Restaurant. Eighteen members and guests attended the dinner and thirty nine members and guests attended the meeting.

The slate of the nominating committee was unanimously elected as follows: Chairman, Robert J. Hansen; Vice Chairman, Steven R. Berke; Clerk, Albert A. Adelman; Executive Committee, William A. Fisher, William A. Hooper, Anthony J. S. Tomasello.

The speakers of the evening were Donald E. McElman and Peter S. Milinazzo of the Perkins Milton Company. A very interesting film was shown of construction on the St. Lawrence Seaway project. The economy of the largest bull dozer ever built, the D-9, was presented.

ALBERT A. ADELMAN, *Acting Clerk*

MARCH 27, 1957.—The third program meeting of the Construction Section was held in the Society Rooms.

Chairman Robert J. Hansen introduced the guest speaker, Mr. James J. Kenney, Structural Engineer with the Portland Cement Association, whose subject was "Fabrication and Erection of Prestressed Concrete Bridges".

Two types of casting yards were described and 35 mm slides were shown of the casting years and erection procedures. It was an informative talk on a subject of great current interest.

Chairman Hansen announced a change of date for the next meeting from May 29 to May 22.

Prior to the meeting, 28 members and guests had dinner at Pattens' Restaurant.

Forty two members and guests attended the meeting.

ALBERT A. ADELMAN, *Clerk*

ADDITIONS

Members

Adin N. Bailey, 3 Wildwood Terr., Winchester, Mass.
Harold Bateson, 102 Underwood Ave., Lakewood, Rhode Island.
George E. Bradley, 14 Perkins Manot S.4, Jamaica Plain, Mass.
Daniel Collins, 15 Margin St., Peabody, Mass.
Leonard J. Edel, 8 Hillside Road, Wellesley Hills Mass.
Alexander M. Gibson 45 Apthorp St. Wollaston Mass.
Louis Katona, Box 432, Kenburma, Mass.
Norman E. Jackson, 16 Bonwood Road, Needham, Mass.
Thomas C. LaCava, Suncook, N. H.
Francis J. Lariviere, 61 So. Spring St., Concord, N. H.
Robert H. Lubker, 430 Lloyd Ave., Providence, R. I.

John D. Marr, Jr., 22 Washington St., Needham, Mass.
Ross E. McKinney, 7 Auburn Road, Wellesley, Mass.
Robert W. Muther, Jr., 37 Grosvenor Road, Needham, Mass.
Gerald S. Parker, 390 Highland Avenue, Somerville, Mass.
Chester C. Pease, Jr., 5 Lothrop Road, Reading, Mass.
Joseph M. Sideransky, 27 Stearns Road, Brookline, Mass.
Maynard Spekin, 7 Strathcona Road, Dorchester, Mass.
William C. Vose, Box 310, Amherst, N. H.
Robert L. Yocum, 465 Sylvan Knolls, Stamford, Conn.

Juniors

Peter S. Parsonson, 20 Torre Street, Reading, Mass.
Arthur E. Sullivan, 45 Newbury Street, Boston, Mass.

Student

Govindaraju Mohanarao, 1-083 Mass. Inst. Technology, Cambridge, Mass.

DEATHS

Irving E. Moultrop, February 24, 1957.

ANNUAL REPORTS

REPORT OF THE BOARD OF GOVERNMENT FOR YEAR 1956 - 1957

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

Pursuant to the requirements of the By-Laws the Board of Government presents its report for the year ending March 20, 1957.

The following is a statement of the status of membership in the Society:

Honorary	7
Members	960
Associates	5
Juniors	86
Students	9
Total	1,067

Student Chapters 2

Summary of Additions

New Members	54
New Juniors	25

Reinstatements

Members	5

Summary of Transfers

Junior to Members	10

Summary of Loss of Members

Deaths	10
Resignations	3
Dropped for non-payment of dues	11
Dropped for failure to transfer	2

Life Members	106
Members becoming eligible today for Life Membership	9
Remission of dues	1
Applications pending on March 20, 1957	24

Honorary Membership is as follows:

Frank M. Gunby, elected, February 15, 1950
Karl R. Kennison, elected, February 7, 1951
Charles W. Sherman, elected, February 19, 1947
Charles M. Spofford, elected, December 19, 1945
Howard M. Turner, elected, February 18, 1952
Karl Terzaghi, elected, March 3, 1952
William F. Uhl, elected, February 7, 1955

The following members have been lost through death:

Thomas A. Berrigan, May 11, 1956
David Bornstein, Sept. 12, 1956
Otis D. Fellows, April 13, 1956
Howard A. Gray, April 23, 1956
Richard K. Hale, Sept. 17, 1956
Arthur C. King, Mar. 4, 1956
Langdon Pearse, July 17, 1956
Charles F. Morse, Aug. 4, 1956
Irving E. Moultrop, Feb. 24, 1957
Donald O. Coe, 1956

Meetings of the Society

March 21, 1956. Address of the retiring President, Edwin B. Cobb, "Some Observations on Civil Engineering Education".

April 23, 1956. Joint Meeting with Massachusetts Section, American Society of Civil Engineers. "Panel Discussion on Mandatory Registration of Engineers and Land Surveyors in Massachusetts." Ralph W. Horne, Moderator. Panelists, Oscar S. Bray (ASCE), Thomas R. Camp (BSCE), Edward H. Barry (ASME and Mass. Board of Registration), Charles T. Chave (MSPE, ASME, AICHE) and Frederick S. Bacon, Jr. (AIEE).

May 16, 1956. Joint Meeting with Massachusetts Section, American Society of Civil Engineers and Sanitary Section, BSCE. "Some Experiences of a Sanitary Engineer as an Expert Witness." E. Sherman Chase, Partner, Metcalf & Eddy.

September 29, 1956. Inspection Trip to the Massachusetts Turnpike.

October 30, 1956. Joint Meeting with the Massachusetts Section American Society of Civil Engineers and Surveying and Mapping Section BSCE. "Science Studies the Antarctic." Father Daniel J. Linehan, Director, Seismological Observatory, Weston College, Weston, Mass.

November 19, 1956. Joint Meeting with the Massachusetts Section, American Society of Civil Engineers and Hydraulics Section, BSCE. Guided tour through the Computer Laboratory, M.I.T. "The Use of Analog and Digital Computers in Civil Engineering," Saul Namyet of Mass. Institute of Technology.

December 19, 1956. Joint Meeting with the Sanitary Section, BSCE, "Civil and Sanitary Engineering Trends in Europe and Latin America." Prof. Harold A.

Thomas, Jr., Gordon McKay Professor of Civil and Sanitary Engineering, Harvard University.

January 23, 1957. Joint Meeting with the Structural Section. "Renovations to South Boston Army Base". Robert J. Basso, Corps of Engineers, U. S. Army; Frank L. Lincoln, Partner, Fay, Spofford & Thorndike; Joseph Peraino, Chief Engr., Merritt-Chapman & Scott.

February 20, 1957. "Use of Photogrammetry in Highway Engineering." John Clarkeson, Clarkeson Engineering Corporation.

Attendance at Meetings

DATE	PLACE	MEETING	DINNER
March 21, 1956	Hotel Vendome	43	182
April 23, 1956	Faculty Club, M.I.T.	112	94
May 16, 1956	United Community Services Building	93	55
September 29, 1956	Field Trip Over Mass. Turnpike		
October 30, 1956	Northeastern University	386	359
November 19, 1956	Mass. Inst. Technology	109	92
December 19, 1956	United Community Services Building	40	*
January 23, 1957	United Community Services Building	86	51
February 20, 1957	United Community Services Building	89	*

*Collation served after meeting.
Average Attendance—117

Sections

Twenty-four meetings were held by the Sections of the Society during the year. These meetings of the Sections offering opportunity for more detailed discussions continue to demonstrate their value to their members and to the Society. A wide variety of subjects was presented and large attendance at these meetings has continued. The Annual Reports of the various Sections will be presented at the Annual Meeting and will be published in the April issue of the JOURNAL.

*Funds of the Society**

Permanent Fund. The Permanent Fund of the Society has a present value of $63,551.22. The Board of Government authorized the use of as much as necessary of the current income of this fund in payment of current expenses. By vote of the Society (as prescribed by the By-Laws) at the January 23, 1957 and February 20, 1957 meetings, the Board of Government was authorized to transfer an amount not to exceed $1800 from the Principal of the Permanent Fund for Current Expenditures. The amount necessary to transfer for Current Expenditures was

John R. Freeman Fund. In 1925 the late John R. Freeman, a Past President and Honorary Member of the Society, made a gift to the Society of securities

*Details regarding the value and income of these funds are given in the Treasurer's Report.

which was established as the John R. Freeman Fund. The income from this fund is to be particularly devoted to the encouragement of young engineers. Mr. Freeman suggested several uses, such as the payment of expenses for experiments and compilations to be reported before the Society; for underwriting meritorious books or publications pertaining to hydraulic science or art; or a portion to be devoted to a yearly prize for the most useful paper relating to hydraulics contributed to this Society; or establishing a traveling scholarship every third year open to members of the Society for visiting engineering works, a report of which would be presented to the Society. The Committee this year voted to contribute a sum not to exceed $1000 to help finance the cost of the "Seminar on Waste Water Disposal and Treatment", conducted by the Sanitary Section.

Edmund K. Turner Fund. In 1916 the Society received 1,105 books from the library of the late Edmund K. Turner, and a bequest of $1,000, "the income of which is to be used for library purposes". The Board voted to use $50 of the income for the purchase of books for the library. The expenditures from this fund during the year was $39.50.

Alexis H. French Fund. The Alexis H. French Fund, a bequest amounting to $1,000 was received in 1931, from the late Alexis H. French of Brookline, a former Past President of the Society. The income of the fund is "to be devoted to the library of the Society". The Board voted to use $50 of the available income for the purchase of books for the Library. The expenditure from this fund during the year was $50.

Tinkham Memorial Fund. The "Samuel E. Tinkham Fund", established in 1921 at Massachusetts Institute of Technology by the Society, "to assist some worthy student of high standing to continue his studies in Civil Engineering", had a value of $2,681.86 on June 30, 1956. Atis A. Liepins of Brentwood, New York, a student in Civil Engineering, class of 1957 was awarded this Scholarship of $100 for the year 1956-57.

Desmond FitzGerald Fund. The Desmond FitzGerald Fund established in 1910 as a bequest from the late Desmond FitzGerald, a Past President and Honorary Member of the Society, provided that the income from this fund shall "be used for charitable and educational purposes". The Board voted on April 9, 1956 to appropriate from the income of this fund the sum of $100 to be known as the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald, and to be given to a student at Northeastern University. It was voted on May 14, 1956 "to adopt the recommendation of the Committee at Northeastern University, namely, a $100 scholarship be given to Donald E. Cullivan. The presentation was made by President Thomas at the Dean's List Dinner, June, 1956.

Clemens Herschel Fund. This fund was established in 1931, by a bequest of $1,000 from the late Clemens Herschel, a former Past President and Honorary Member of the Society. The income from this fund is "to be used for presentation of prizes for papers which have been particularly useful and commendable and worthy of grateful acknowledgment". The Board of Government voted on April 9, 1956 that payment of the Herschel Prize Award and Section Prize

Awards be appropriated from the income of this fund. The expenditures made during the year from the income of this fund was $111.51.

Edward W. Howe Fund. This fund, a bequest of $1,000 was received in 1933 from the late Edward W. Howe, a former Past President of the Society. No restrictions were placed upon the use of this money, but the recommendation of the Board of Government was that the fund be kept intact, and that the income be used "for the benefit of the Society or its members". The Board of Government voted on April 9, 1956 "that a sum not to exceed $200 be appropriated from the income of this fund for purchase of a Map File. The expenditures made during the year from the income of this fund was $200.

William P. Morse Fund. This fund, a bequest of $2,000 was received in 1949 from the late William P. Morse, a former member of the Society. No restrictions were placed upon the use of this money but the recommendation of the Board of Government was "that the fund be kept intact and that the income be used for the benefit of the Society or its members". Upon recommendation of a Committee appointed by the President, the Board voted on April 5, 1954 "to use a sum not to exceed $100 from the income of this fund for a Scholarship to a worthy student in Civil Engineering at Tufts University to be known as the Boston Society of Civil Engineers Scholarship in memory of William P. Morse. The Board voted on April 9, 1956 "to appropriate from the income of this fund the sum of $100 to be known as the William P. Morse Scholarship to a worthy student at Tufts University". The Board of Government voted on May 14, 1956 "that this Scholarship in the amount of $100 be awarded to Gerald Fain, student at Tufts University, in recognition of his scholastic attainments". The Board of Government also voted that a sum not to exceed $95 be appropriated from the income of this fund towards purchase of Map File and reconditioning mimeo machine. The expenditure made during the year from the income of this fund was $200.53.

Prizes

AWARD	RECIPIENT	PAPER
Desmond FitzGerald Medal	T. William Lambe	"The Storage of Oil in an Earth Reservoir".
Clemens Herschel Award	Jacob Feld	"Structural Failures and How to Avoid Them".
Sanitary Section Award	John S. Bethel, Jr.	"Comprehensive Sanitary Survey of Woonsocket, Rhode Island".
Surveying & Mapping Section Award	Charles L. Miller	"Some New Conceptions and Old Misconceptions of Photogrammetry".

Library

The report of the Library Committee contains a complete account of the Library Committee activities during the past year.

Committees

The usual special committees dealing with the activities and conduct of the Society were appointed. The membership of these committees is published in the JOURNAL and the reports of the committees will be presented at the Annual Meeting.

In addition to routine business of the Society the Board of Government took action as follows:

April 9, 1956	Voted that the President be authorized to appoint a committee to report on Massachusetts Public Building Commission Procedures and to make specific recommendations covering establishment of fees for civil engineering services.
October 15, 1956	Voted to request the Joint Legislative Committee to prepare (with the aid of Legislative Counsel) a form of bill for the Mandatory Registration of Engineers in Massachusetts and to submit same forthwith to the Legislature.

Your Board, in conclusion, wishes to express its appreciation of the excellent work done by the Officers of the Sections and by the Committees of the Society.

JOHN G. W. THOMAS, *President*

REPORT OF THE TREASURER

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

The fiscal year ending March 1, 1957, has been a busy one for our Society and the following report, by your treasurer, outlines the financial transactions from March 1, 1956 to March 1, 1957.

As in previous years, the Boston Safe Deposit and Trust Company continues to be custodian of our securities and to be our investment counsel. Each month during the year, the treasurer receives a statement of the income and principal accounts. Once a year, dated close of business as of March 1st, the treasurer receives from the Boston Safe Deposit and Trust Company a certified statement of Bonds, Stocks, Interest and Dividends held by the Bank in the name of the Society and all payments made by the Bank in the name of the Society. The sale or purchase of all securities shown in this report have been made upon the recommendation of the above named Bank and by approval of the Board of Government. The financial standing of the Society as of March 1, 1957, is shown by six tables, namely:—

Table I	Record of Investments—Purchased
Table II	Record of Investments—Sold
Table III	Distribution of Funds—Receipts and Expenditures
Table IV	Record of Investments—Bonds
Table V	Record of Investments—Stocks
Table VI	Record of Investments—Cooperative Bank

This year it was necessary to transfer $3,821.39 from the Permanent Fund. The net income to the Permanent Fund was $3,284.60, therefore a sum of $536.79 was transferred from the principal of the Permanent Fund. Below, the current fund receipts, expenditures and deficits are compared for the past five years.

	1952-1953	1953-1954	1954-1955	1955-1956	1956-1957
Receipts					
Dues	$6,627	$7,085	$7,428	$7,589	$7,995
Other than dues	5,505	5,754	5,782	6,340	6,546
Total Receipts	$12,132	$12,829	$13,210	$13,929	$14,541
Total Expenditures	14,576	14,724	15,814	17,075	18,362
Deficit	$2,444	$1,885	$2,604	$3,146	$3,821

The book value of our securities, bank deposits, and cash are compared for this fiscal year and the previous four fiscal years. The Secretary's petty cash fund of $30.00, the Society's Library, and other physical properties are not included in this comparison. An increase of $2,128.42 is noted for this year.

	March 1 1953	March 1 1954	March 1 1955	March 1 1956	March 1 1957
Bonds	$39,475.48	$40,643.42	$39,617.01	$48,713.37	$51,907.25
Stocks	45,858.01	46,112.29	49,359.89	50,406.00	51,465.51
Coop. Bank	10,221.96	5,425.68	5,559.44	5,697.24	4,845.16
Cash	2,322.89	4,605.98	6,269.11	5,661.13	4,388.24
Totals	$97,878.34	$96,787.37	$100,805.45	$110,477.74	$112,606.16

Three of the funds shown in Table III, measure some of our important activities. The investment fund shows continued interest and a healthy growth of sales for Volumes I and II, "Contributions to Soil Mechanics". Secondly, the structural lecture series has been successfully concluded, and thirdly, the sanitary lecture series is now in full swing.

At the close of business March 1, 1957, our total market value of all securities and cash was $167,489.64.

CHARLES O. BAIRD, JR., *Treasurer*

TABLE I—RECORD OF INVESTMENTS—PURCHASED
March 1, 1956 to March 1, 1957

Bonds	Maturity	Interest Rate	Date Purchased	Cost
Scott Paper Co. Conv. Deb.	March 1, 1971	3.00%	March 30, 1956	$1123.79
Tide Water Asso. Oil Co. S. F. Deb.	April 1, 1986	3.50%	April 21, 1956	$2032.50
Florida Power Corp. 1st Mtge.	July 1, 1986	3.875%	July 30, 1956	$5037.59
Superior Oil Co. Deb. S. F.	July 1, 1981	3.75%	July 30, 1956	$4000.00

Stocks	Classification	Number of Shares	Date Purchased	Cost
Hartford Fire Insurance Co.	common	3	May 21, 1956	$456.00
American Telephone & Telegraph Co.	common	7	Oct. 7, 1956	754.24

TABLE II—RECORD OF INVESTMENTS—SOLD
March 1, 1956 to March 1, 1957

Name	Date Sold	Book Value March 1, 1956	Amount Received	Profit
U. S. of America Treasury Bond	July 30, 1956	$8,000.00	$8,170.00	$170.00
U. S. of Savings Bond Series	Nov. 1, 1956	1,000.00	1,000.00	0.00
204 Rights New England Electric System	May 2, 1956		11.60	
100 Rights Pacific Gas & Electric Company	July 5, 1956		21.94	
Cooperative Bank, Matured Cert.	July 2, 1956	1,000.00	1,000.00	0.00

Table III—Report of Treasurer—March 1, 1957

	Distribution of Funds			Receipts and Expenditures				
	Book Value March 1, 1956	Interest & Cash	Dividends Credit	Net Profit or Loss at Sale or Maturity		Transfer of Funds		March 1, 1957 Book Value
	1	2	3	4	5	6	7	8
Bonds	$48,713.37	$1,379.03		$170.00		$12,193.88	$9,000.00	$51,907.25
Coop. Bank	5,697.24	15.00	$147.92				1,000.00	4,845.16
Stocks	50,406.00	4,413.45				1,210.24	150.73	51,465.51
Available for Investment	4,151.13						1,272.89	2,888.24
	$108,977.74	$5,807.48	$147.92	$170.00		$13,404.12	$11,423.62	$111,106.16

Columns 1 + 3 + 6 — 7 — 8

Funds	Book Value March 1, 1956	Allocation of Income—Profit and Loss: Income Columns 2 & 3	Net Profit Columns 4 & 5	Received	Expended	Book Value March 1, 1957
Permanent	$63,380.99	$3,585.27	$102.34	$605.00	$4,122.08	$63,551.52
John R. Freeman	31,878.96	1,803.29	51.48		1,151.24	32,582.49
Edmund K. Turner	1,190.16	67.32	1.92		45.15	1,214.25
Desmond FitzGerald	2,368.30	133.97	3.82		113.22	2,392.87
Alexis H. French	1,204.92	68.16	1.95		55.72	1,219.31
Clemens Herschel	1,198.98	67.82	1.94		117.19	1,151.55
Edward W. Howe	1,238.93	70.08	2.00		205.87	1,105.14
William P. Morse	2,440.52	138.05	3.94		212.10	2,370.41
Publication	3,121.95			790.09	76.25	3,835.79
Surveying Lectures	379.07	21.44	0.61		1.81	399.31
Sanitary Lectures				1,753.00	757.88	995.12
Structural Lectures	574.96			183.50	470.06	288.40
	$108,977.74	$5,955.40	$170.00	$3,331.59	$7,328.57	$111,106.16
Current	1,500.00	3,284.60*		15,077.74*	18,362.34	1,500.00
Totals	$110,477.74	$9,240.00	$170.00	$18,409.33	$25,690.91	$112,606.16

Secretary's change fund of $30.00 should be added to show total cash.

Cash Balance March 1, 1957

Investment Fund	$2,888.24	$3,821.39*	Transferred from Permanent Fund
Current Cash	1,500.00	3,284.60	Transferred from Income of Permanent Fund
Total cash	$4,388.24	$536.79	Transferred from principal of Permanent Fund

TABLE IV—RECORD OF INVESTMENTS—BONDS
March 1, 1956 to March 1, 1957

Bonds	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest Received	Par Value	Book Value March 1, 1957	Market Value March 1, 1957
U. S. Savings Bond Ser. G	Nov. 1, 1956	2½%	$25.00	$1,000.00*		
U. S. Savings Bond Ser. G	May 1, 1958	2½%	100.00	4,000,00	$4,000.00	$4,000.00
U. S. Savings Bond Ser. G	May 1, 1960	2½%	25.00	1,000.00	1,000.00	1,000.00
U. S. of Am. Treasury Bond	June 15, 1983	3¼%	159.13	8,000.00**		
U. S. Savings Bond Ser. K	Aug. 1, 1966	2.76%	193.00	7,000.00	7,000.00	7,000.00
Columbia Gas System, Deb. Ser. D	July 1, 1979	3.50%	70.00	2,000.00	2,066.17	1,846.24
Consumers Power Co. 1st Mtge.	Sept. 1, 1975	2⅞%	43.11	3,000.00	3,140.35	2,700.00
Florida Power Corp. 1st Mtge.	July 1, 1984	3.125%	31.25	1,000.00	1,017.50	860.00
Province of Ontario	Sept. 1, 1972	3¼%	48.75	3,000.00	2,936.25	2,685.00
Public Service Elec. & Gas Co. 1st & Ref.	June 1, 1979	2⅞%	115.00	4,000.00	4,097.50	3,410.00
Southern Pacific 1st Ser. A, Oregon Lines	March 1, 1977	4.50%	90.00	4,000.00	4,191.30	3,980.00
Georgia Power Co., 1st Mtge.	Dec. 1, 1977	3.375%	168.75	5,000.00	5,162.50	4,525.00
General Motors Accept. Corp. Deb.	Sept. 1, 1975	3.625%	90.60	5,000.00	5,101.80	4,762.50
Scott Paper Co., Conv. Deb.	Mar. 1, 1971	3.00%	12.34	1,000.00	1,123.79	975.00
Tide Water Assoc. Oil Co. Deb. S.F.	Apr. 1, 1986	3.50%	35.00	2,000.00	2,032.50	1,930.00
Florida Power Corp. 1st Mtge.	July 1, 1986	3.875%	96.90	5,000.00	5,037.59	4,937.50
Superior Oil Co. Deb. S.F.	July 1, 1981	3.75%	75.00	4,000.00	4,000.00	3,920.00
Totals			$1,379.03	$51,000.00	$51,907.25	$48,531.24

*U. S. Savings Bond Series G, matured November 1, 1956.

**U. S. of America Treasury Bond sold July 30, 1956.

TABLE V—RECORD OF INVESTMENTS—STOCKS
March 1, 1956 to March 1, 1957

Stocks	Classification	Number of Shares	Dividend Received	Book Value March 1, 1957	Market Value March 1, 1957
American Tel. & Tel. Co.	Common	69	$573.75	$7,923.06	$12,299.25
Consolidated Edison Co. of N. Y.	Common	50	120.00	2,556.12	2,200.00
Continental Insurance Co.	Common	136	272.00	3,483.73	7,038.00
General Electric Co. of New York	Common	150	300.00	2,341.47	8,400.00
Hartford Fire Insurance Co.	Common	26	75.75	1,472.75	3,822.00
Jewel Tea Co., Inc.	Common	30	60.00	1,442.90	1,530.00
Nat'l Dairy Products Corp.	Common	100	175.00	1,154.74	3,675.00
New England Electric System	Common	198	198.00	3,095.00	3.341.25
Pacific Gas & Electric Co.	Preferred	100	150.00	2,704.89	3,187.50
Pacific Gas & Electric Co.	Common	100	240.00	3,365.34	4,900.00
Radio Corporation of America	Preferred	20	70.00	1,720.75	1,472.50
Southern Calif. Edison Co., Ltd.	Preferred	40	96.00	1,140.24	2,100.00
Southern Calif. Edison Co.	Common	45	108.00	1,350.65	2,160.00
Standard Oil of New Jersey	Common	324	680.40	3,328.16	18,144.00
Southern Railway Co.	Preferred	75	75.00	1,136.80	1,425.00
Texas Company	Common	208	488.80	2,956.32	12,428.00
Union Carbide & Carbon Corp.	Common	100	240.00	2,958.44	10,887.50
Union Pacific Railroad	Common	220	352.00	2,473.29	6,215.00
Scott Paper Company	Common	75	138.75	4,860.86	4,500.00
Total		2,066	$4,413.45	$51,465.51	$109,725.00

Table VI—Record of Investments—Cooperative Bank
March 1, 1956 to March 1, 1957

Cooperative Banks	Classification	Number of Shares	Dividend Received	Book Value. March 1, 1957	
Suffolk Cooperative Federal Savings and Loan Association	Matured Certificate	5	$15.00		Sold July 2, 1956
Suffolk Cooperative Federal Savings and Loan Association Account No. S-631	Savings Account		147.92	$4,845.16	

REPORT OF THE SECRETARY

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

The following is a statement of cash received by the Secretary and of the expenditures approved by the President in accordance with the Budget adopted by the Board of Government.

FOR THE YEAR ENDING MARCH 20, 1957

	Account Number	Expenditures	Receipts
Office			
Secretary, salary & expense	(20)	$499.69	
Treasurer's Honorarium	(21)	200.00	
Stationery, printing & postage	(22)	484.25	
Incidentals & Petty Cash	(23)	107.15	
Insurance & Treasurer's Bond	(24)	83.25	
Quarters, Rent, Lt. & Tel.	(26)	3,296.11	$1,400.00
Office Secretary	(28)	3,851.20	
Social Security	(31)	93.90	
Meetings			
Rent of Halls, etc.	(40)	200.00	
Stationery, printing & postage	(41)	58.00	
Hospitality Committee	(42)	1,271.12	1,076.87
Reporting & Stereopticon	(43)	13.50	
Annual Meeting, March, 1955	(44)	814.75	647.50
Sections			
Sanitary Section	(45)	23.85	
Structural Section	(46)	66.87	
Transportation Section	(47)	10.00	
Hydraulics Section	(48)	20.00	
Construction Section	(49)	4.00	
Surveying & Mapping Section	(50)	12.00	
Journal			
Editor's salary & expense	(60)	512.88	
Printing & postage	(62)	5,121.16	
Advertisements	(64)	57.60	2,027.55
Sale of Journals & Reprints	(63)		1,328.89
Library			
Periodicals	(69)	73.42	
Binding	(70)	195.60	
Fines	(6)		1.52
Binding Journals for Members	(81)	12.90	12.90
Badges	(80)		18.00
Forward		$17,083.20	$6,513.23

	Account Number	Expenditures	Receipts
Brought forward		$17,083.20	$6,513.23
Bank Charges	(82)	4.85	
Miscellaneous	(84)	319.90	32.72
Engineering Societies Dues and charge for Journal Space	(88)	930.39	
Public Relations Committee	(67)	15.00	
Flood Committee	—	9.00	
Dues from B.S.C.E. Members			7,995.00
Trans. Income Perm. Fund			3,284.60
Trans. Principal			536.79
		$18,362.34	$18.362.34

Entrance fees to Permanent Fund $605.00

Fifty-four New Members; 25 New Juniors; 2 New Students; 10 Juniors transferred to Member.

The above receipts have been paid to the Treasurer, whose receipt the Secretary holds. The Secretary holds cash amounting to $30 included as payment under item 23 (Petty Cash) to be used as a fixed fund or cash on hand. $90 withholding tax and $45.66 Social Security which is payable to the Collector of Internal Revenue in April, 1957 is not included in the above tabulation.

ROBERT W. MOIR, *Secretary*

REPORT OF THE AUDITING COMMITTEE

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

We have reviewed the records and accounts of the Secretary and Treasurer of the Boston Society of Civil Engineers and we have compared the bank statement of securities held by the Boston Safe Deposit and Trust Company with the enumeration submitted by the Treasurer.

We have found them to be in order and to account accurately for the Society's Funds.

ERNEST L. SPENCER
GEORGE G. BOGREN

REPORT OF THE EDITOR

March 20, 1957

To the Board of Government
Boston Society of Civil Engineers:

The JOURNAL was issued quarterly, in the months of April, July and October, 1956 and January, 1957 as authorized by the Board of Government on December 20, 1935.

During the year there have been published 12 papers presented at meetings of the Society and Sections.

The four issues of the JOURNAL contained 302 pages of papers and proceedings, 14 pages of Index and 51 pages of advertising, a total of 367 pages. An average of 1410 copies per issue were printed.

The cost of printing the JOURNAL was as follows:

Expenditures	
Composition and printing	$3,986.89
Cuts	845.77
Wrapping, mailing & postage	288.50
Editor	496.88
Advertising Solicitor	57.60
Copyright	16.00
	$5,691.64
Receipts	
Receipts from sale of JOURNAL and reprints	$1,328.89
Receipts from Advertising	2,027.55
	$3,356.44
Net cost of JOURNAL to be paid from Current Fund	$2,335.20

CHARLES E. KNOX, *Editor*

REPORT OF THE LIBRARY COMMITTEE

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

On behalf of the Library Committee, I submit this report for the year 1956-1957.

During the year 232 books were loaned and a total of $1.52 was collected in fines. The number of books loaned was slightly higher than that of the previous year. In addition, more use was made of the library as a source of reference material.

The following expenditures were made during the year:

Subscriptions to Periodicals	$73.42
New Books	80.75
Binding Periodicals, including back issues of Water Supply Papers	195.60

The following books were purchased for the library:

American Civil Engineering Practice, Volumes I and II, 1956, Robert W. Abbett.
Biological Treatment of Sewage and Industrial Wastes, 1956, Joseph McCabe and W. W. Eckenfelder, Jr.
Design and Practice—Foundations, 1956, Elwyn E. Seelye.
Air Pollution Handbook, 1956, Magill, Holden & Ackley.
Source Book of Atomic Energy, 1950, Samuel Glasstone.
Professional Engineering Registration Laws, Alfred L. McCawley.

In addition, Professor Gordon M. Fair has donated to the library the second edition of "Sewage Treatment", which was written by Karl Imhoff and Prof. Fair.

An additional file for the U. S. Geological Survey maps has been purchased, as was recommended last year.

GEORGE M. REECE, *Chairman*

REPORT OF THE HOSPITALITY COMMITTEE

March 12, 1957

To the Boston Society of Civil Engineers:

The Hospitality Committee submits the following report for the year 1956-57:

Four regular meetings, an annual meeting, two joint meetings with the Massachusetts Section of the American Society of Civil Engineers, a Student Night meeting and a field trip were held during the year at the locations noted in the following table.

Two catered dinners were served in the Adams Room of the United Community Services Building, with an increase in attendance over similar meetings last year at each one. The attendance at collations served by this committee after meetings has decreased during the past two years, with less than half of those attending the meetings remaining for the collation.

The average attendance at all nine meetings was 117 members and guests, an increase of 17. The average attendance at the four regular meetings held at the United Community Services Building was 77, a decrease of 18 over the previous year.

ATTENDANCE AT MEETINGS

DATE	PLACE	MEETING	DINNER
Mar. 21, 1956	Hotel Vendome	43	182
Apr. 23, 1956	Faculty Club, M.I.T.	112	94
May 16, 1956	United Community Services Building	93	55
Sept. 26, 1956	Field Trip over Massachusetts Turnpike	96	90
Oct. 30, 1956	Northeastern University	386	359
Nov. 19, 1956	Mass. Inst. Technology	109	92
Dec. 19, 1956	United Community Services Building	40	*
Jan. 23, 1957	United Community Services Building	86	51
Feb. 20, 1957	United Community Services Building	89	—

*Collation served after meeting.

DAVID H. HAMILTON, *Chairman*

REPORT OF MEMBERSHIP COMMITTEE

March 20, 1957

To the Boston Society of Civil Engineers:

The Membership Central Committee held two meetings during the past year.

Membership has continued in an upward trend. As in the previous year, in cooperation with the Public Relations Committee, application forms accompanied by attractive color brochures have been sent to numerous prospective members. Particular attention has been given to recent engineering graduates employed in this area; qualified engineers in consulting offices; and members of other engineering organizations on municipal and state levels. Many individual members of the Society have cooperated in distributing applications and brochures within their organizations.

Interest in membership has received particular impetus from the presentation of the Seminar on Waste Water Treatment and Disposal by this and other Societies.

The status of new membership for the current year is as follows:

Elected to Grade of Member	54
Elected to Grade of Junior	25
Elected to Grade of Student	2
Total	81

As of this date, the total membership of the Society is 1,067. There are, in addition, 18 applications on hand pending action.

JOSEPH C. LAWLER, *Chairman*

REPORT OF ADVERTISING COMMITTEE

March 20, 1957

To the Boston Society of Civil Engineers:

The Advertising Committee held one formal meeting during the year at which the problems of soliciting advertisements for the JOURNAL were discussed at considerable length. It was voted that, with the approval of the Board of Government, Mrs. Virginia S. Boudia be appointed advertising solicitor.

The following advertising has been carried on in the JOURNAL during the year:

	April	July	Oct.	Jan. (1957)
Professional Cards	37	38	38	37
½ page ads	2	1	1	1
¼ page ads	24	24	24	22
Inside Front Cover		1	1	1
Outside Back Cover		1	1	1
Full page	1			
Total Pages of Advertising	12	13	13	13

During the past year $2,027.55 was collected from advertisers from which commissions totalling $57.60 were paid to the solicitor.

An increase in advertising material is earnestly desired but difficult to attain. This Committee concurs with the suggestion of the 1955-56 Advertising Committee that consideration be given to inserting pages of advertising between published papers as an attraction to potential advertisers.

The Committee wishes to express its appreciation to all those who have supported the JOURNAL through advertising.

RALPH M. SOULE, *Chairman*

REPORT OF THE JOHN F. FREEMAN FUND COMMITTEE

March 18, 1957

*To the Boston Society of Civil Engineers*s

Mr. Lawrence C. Neale, who was a recipient of a Scholarship for travel in Europe during 1955, read a paper on it at a meeting of the Hydraulics Section in November. His final report is being completed.

Last October the Committee voted to appropriate not exceeding $1,000.00 towards the expenses of the Seminar on Water Treatment and Disposal which is now being carried on.

Mr. Harvey B. Kinnison who has been a member of the Freeman Fund Committee since 1950 resigned last autumn, as he was leaving to take up his residence in California. At its last meeting the Board of Government appointed Thomas R. Camp to fill Mr. Kinnison's place.

HOWARD M. TURNER, *Chairman*

REPORT OF THE JOINT COMMITTEE ON LEGISLATIVE AFFAIRS

March 4, 1957

To the Boston Society of Civil Engineers:

The Joint Committee on Legislative Affairs has held many meetings during the year. Meetings were held with committees of an architects' association, of other engineering societies, and of land surveyors' associations, for the purpose of discussing proposed legislation.

The results of a poll, taken in October 1956, showed about 90 per cent of returns in favor of mandatory registration for engineers. Detailed results of said poll are as follows:

	Mass. Sec.A.S.C.E.	B.S.C.E.
Ballots sent out	1133	846
Ballots received	590	514
Results are as follows:		
In Favor		
ASCE Members	342	252
ASCE-BSCE Members	189	198
Totals	531	450
No Preference		
ASCE Members	8	14
ASCE-BSCE Members	1	1
Totals	9	16
Opposed		
ASCE Members	34	26
ASCE-BSCE Members	16	22
Totals	50	48

The principal business of the Joint Committee has been the preparation and filing of a bill to provide for mandatory registration of engineers. This bill is now identified as Senate 442. A Hearing was scheduled before Committee on State Administration on March 13, 1957. The proposed legislation closely follows the Model Law and should receive the wholehearted support of all Massachusetts engineers.

Bills before the current session of the General Court, which are of concern to civil engineers, are as follows:

Senate 442. Relating to mandatory registration, sponsored by BSCE-ASCE.

Senate 449. Relating to preparation of plans, etc., sponsored by Legislative Committee, Massachusetts State Association of Architects.

Senate 450. Relating to registration of architects, sponsored by Massachusetts State Association of Architects.

House 912. Relating to a bureau of registration for professional engineers, landscape architects, and of land surveyors, sponsored by J. R. DeNormandie.

House 1272. Relating to registration of land surveyors, sponsored by Massachusetts Association of Land Surveyors and Civil Engineers.

House 1275. Relating to award of contracts on public buildings, sponsored by Legislative Committee, Massachusetts State Association of Architects.

House 1765. Relating to Professional Status of certain engineers in the employ of the Commonwealth of Massachusetts, sponsored by Local No. 780, AFL-CIO.

The sudden passing of our Chairman, Thomas A. Berrigan, was a very great loss to the Joint Committee. Being both an engineer and lawyer, his leadership was a most valued asset.

The present membership of the Committee is James F. Brittain, Ernest A. Dockstader, Charles L. Miller, Edward Wright, and the writer.

FRANK L. HEANEY, *Chairman*

REPORT OF PUBLIC RELATIONS COMMITTEE

Boston, Mass., March 20, 1957

To the Boston Society of Civil Engineers:

The Public Relations Committee held two meetings during the past year. Besides the usual notices of society meetings contained in the JOURNAL of the Engineering Societies of New England, it has been the practice to send out bulletin board notices to be posted where they might attract the attention of society members, or others with similar interests.

Some publicity was obtained by sending notices of the meetings to the local newspapers and to various publications catering to the construction industry, particularly, "The Dodge Bulletin", "Nerba", "New England Construction", and "Gainey's Construction News Letter". In this connection it is to be noted that publications such as these are published at various intervals, and to obtain satisfactory coverage it is necessary to have the material available a considerable period of time in advance of the event.

It is therefore recommended that efforts be made to establish as close a relationship as possible between the Board of Government and the Public Relations Committee, in order that the latter might have all the data pertinent to the events being planned at the earliest possible moment.

THOMAS C. COLEMAN, *Chairman*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SANITARY SECTION

February 13, 1957

*To the Sanitary Section, Boston Society of Civil Engineers*s

1. March 7, 1956. Annual Meeting. The following officers and members of the Executive Committee were elected:

Darrell A. Root	Chairman
John F. Flaherty	Vice-Chairman
Clair N. Sawyer	Clerk
Joseph C. Knox	Executive Committee
Harold A. Thomas, Jr.	Executive Committee
George M. Reece	Executive Committee

Speaker of the evening was Mr. Richard Hazen who discussed "Competitive Water Uses: Sanitary Engineering Aspects". Eighty members and guests attended the meeting and thirty members and guests attended the informal dinner at Patten's.

2. May 2, 1956. Joint Meeting with Hydraulics Section was held at Hydrodynamics Laboratory at M.I.T. Prof. A. T. Ippen and D. R. F. Harleman presented discussions of "Density Current Problems in Hydraulic and Sanitary Engineering". Interesting laboratory demonstrations were part of the program. Fifty-eight members and guests were in attendance.

3. May 16, 1956. Joint Meeting with B.S.C.E. and Massachusetts Section of ASCE. Mr. E. Sherman Chase gave a talk on "Some Experiences of a Sanitary Engineer as an Expert Witness". Ninety-three members and guests were present and fifty-five were present at an informal dinner.

4. June 12, 1956. Joint Meeting with Hydraulics Section. Mr. Langdon Pearse gave an illustrated talk on "Development and Operations of the Sanitary District of Chicago". Seventy-five members were present at the meeting and twenty-five at the dinner.

5. October 3, 1956. Regular meeting of the Sanitary Section. Mr. Ariel A. Thomas spoke on the "Raritan River Project". Fifty-two were present at the meeting and fourteen at the informal dinner.

6. December 19, 1956. Joint Meeting with BSCE. Prof. Harold A. Thomas, Jr. gave an illustrated talk on his recent trip to Europe and Brazil.

A nominating committee consisting of John S. Bethel, Jr., E. W. Moore and F. M. Cahaly was elected and directed to present a slate of officers and members of Executive Committee at the Annual Meeting of the Section to be held March 6. 1956. Forty members and guests were in attendance.

Total attendance at the six meetings was 398 or an average per meeting of 66.

Four meetings of the Executive Committee were held during the year.

The section in cooperation with the Massachusetts Section of the American Society of Civil Engineers and the Eastern Massachusetts Association of Professional Engineers and Land Surveyors sponsored a seminar on Waste Water Treatment and Disposal consisting of a series of 12 lectures on selected topics by authorities in the field. A total of 277 persons registered for the seminar and average attendance at the first six lectures was 159. Facilities for accommodating the large numbers at meetings were provided at the Public Works Building of the Commonwealth of Massachusetts through the courtesy of Commissioner Carl A. Sheridan.

The subjects of discussion and speakers at the seminar meetings were as follows:

Date	Subject	Lecturer
Jan. 8	Concepts and Functional Outline of Treatment Processes	Prof. G. M. Fair
Jan. 14	Fluid Mechanics Applied to Waste Treatment Problems	Mr. T. R. Camp
Jan. 21	Sedimentation-Flotation (Including Pretreatment Process: Screens, Grit Chambers)	Mr. F. L. Flood
Jan. 28	Hydrology and Oxygen Economy in Stream Purification	Prof. H. A. Thomas, Jr.
Feb. 4	Sludge and Treatment Disposal	Mr. R. S. Rankin
Feb. 11	Disinfection and Chlorination for Control	Prof. R. Eliassen
Feb. 18	Pumps, Measuring Devices, Hydraulic Controls	Mr. A. J. Burdoin
Feb. 25	Water-borne Trade Wastes of New England	Prof. E. W. Moore
Mar. 4	Bichemistry and Billogy in Waste Water Treatment	Prof. R. E. McKinney
Mar. 11	Biological Treatment Processes	Prof. R. E. McKinney
Mar. 18	Hydraulic Problems of Ocean Disposal	Mr. Frank L. Heaney
Mar. 25	Pollution Control Practice in an Expanding Economy	Mr. E. J. Cleary

CLAIR N. SAWYER, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE STRUCTURAL SECTION

Boston, Mass, March 13, 1957

To the Structural Section, Boston Society of Civil Engineers:

Nine meetings of the Structural Section were held during the year as follows:

April 11, 1956.—Dr. T. William Lambe, Associate Professor and Director of Soil Stabilization Laboratory, Massachusetts Institute of Technology, spoke on "The Storage of Oil in an Earth Reservoir." Attendance 30.

May 9, 1956.—Mr. Ralph Riddle, Bethlehem Steel Company presented films and a short talk on the rolling of steel plates and shapes and on the Chesapeake Bay Bridge. Attendance 26.

October 10, 1956.—Mr. Richard C. Kasser, Structural Section of the Sales Development Division, Aluminum Company of America, spoke on the subject "Structural Aluminum—Past, Present and Future." Attendance 37.

November 14, 1956.—Dr. Harl P. Aldrich, Assistant Professor of Soil Mechanics, Massachusetts Institute of Technology, spoke on the subject of problems encountered in the design of oil tank foundations. Attendance 45.

December 5, 1956.—Mr. Howard Simpson, Associate Professor of Structural Engineering, Massachusetts Institute of Technology, presented an illustrated paper on "The New A.C.I. Code—Its Implications and Ramifications." Attendance 88.

January 9, 1957.—Dr. Karl Terzaghi, Professor Emeritus of Civil Engineering, Harvard University, delivered an illustrated talk on the design and construction of the Vermillion Dam across Mono Creek in California. Attendance 97.

January 23, 1957.—Joint meeting with the Main Society. Mr. Robert J. Basso, Corps of Engineers; Mr. Frank L. Lincoln, Fay, Spofford & Thorndike, Inc. presented talks on "Renovations to the South Boston Army Base." Attendance 86.

February 13, 1957.—Mr. Nelson Aldrich, A.I.A. of Campbell and Aldrich spoke on the subject "Interdependence of the Engineer and Architect with some Observations on the Aesthetics of Structures." Attendance 40.

March 13, 1957.—Dr. Lauritz Bjerrum, Director of the Norwegian Geotechnical Institute, Visiting Professor at the Massachusetts Institute of Technology gave an illustrated talk on "Norwegian Experiences with Foundations on Steel Piles." Attendance 72.

The total attendance for the year was 521; average attendance 58.

RICHARD W. ALBRECHT, *Clerk*

REPORT OF THE EXECTIVE COMMITTEE OF THE HYDRAULIC SECTION

February 13, 1957

To the Hydraulic Section, Boston Society of Civil Engineers:

The following meetings were held during the past year.

May 2, 1956.—A joint meeting of the Hydraulic and Sanitary Sections was held at the Hydrodynamics Laboratory of the Massachusetts Institute of Technology. Professors A. T. Ippen and D. R. F. Harleman presented a program under the title "Density Current Problems in Hydraulic Engineering." A discussion of related problems of density currents and jet mixing was followed by laboratory demonstrations of these phenomena. Attendance 58.

June 12, 1956.—A special joint meeting of the Hydraulic and Sanitary Sections was held at the Society Rooms. Mr. Langdon Pearse, Sanitary Engineer of the Metropolitan Sanitary District of Chicago spoke on the subject "Seventh Engineering Wonder of America—The Metropolitan Sanitary District of Chicago." He described the founding of the Sanitary District and gave an illustrated commentary on the Sewage Treatment Works in Chicago. Attendance 75.

November 2, 1956.—A meeting was held in the Society Rooms at which Mr. Lawrence Neale of the Alden Hydraulic Laboratory of Worcester Polytechnic Institute spoke. Mr. Neale held the BSCE Freeman Fellowship for the year 1955 and he described his experiences with an illustrated lecture entitled "European Hydraulic Laboratories and Hydraulic Practice in 1955." Attendance 31.

November 19, 1956.—A joint meeting of the Massachusetts Section of the American Society of Civil Engineers, the Boston Society of Civil Engineers and the Hydraulic Section of the BSCE was held in the Graduate House of M.I.T. Professor Saul Nemyet spoke on the subject "Use of Analogue and Digital Computers in Civil Engineering." The meeting was preceded by a late afternoon visit to the Computer Laboratory in the M.I.T. Barta Building. Attendance 109.

February 6, 1957.—The Annual Meeting and Election of Officers for the Hydraulic Section was held in the Society Rooms. Elected officers for 1957 are:

Chairman	Clyde M. Hubbard
Vice Chairman	James W. Daily
Clerk	Lee G. M. Wolman
Executive Committee	John B. McAleer
	Robert S. Kleinschmidt
	Lawrence E. Neale

Mr. W. Rheingans, Manager of the Hydraulic Department of the Allis Chalmers Manufacturing Co., Milwaukee, Wisconsin gave an illustrated discussion of the topics of "Pump Storage Systems and Pump-Turbines" and "Cavitation." Attendance 88.

The total attendance for the year was 361; the average 72.

JAMES W. DAILY, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE SURVEYING AND MAPPING SECTION

Boston, Mass., February 15, 1957

To the Surveying and Mapping Section
Boston Society of Civil Engineers:

The following meetings of the Surveying and Mapping Section, Boston Society of Civil Engineers were held during the past year:

April 4, 1956.—Charles L. Miller, Asst. Prof. of Surveying at Massachusetts Institute of Technology spoke on "Some New Conceptions and Old Misconceptions of Photogrammetry". Attendance—52.

October 30, 1956.—Father Daniel J. Linehan, Director of the Seismological Observatory, Weston College, gave a very interesting illustrated talk on "Science Studies the Antarctic". This was a joint meeting with the Main Society and the Mass. Section of the American Society of Civil Engineers. Attendance—359.

February 14, 1957.—Commander Kenneth S. Ulm, Boston District Officer of the Coast & Geodetic Survey, gave a talk on the General Functions of the Coast & Geodetic Survey, and showed a film on "Tides and Currents". Attendance—39.

Total attendance—450; average attendance—150.

George A. McKenna, *Clerk*

REPORT OF THE EXECUTIVE COMMITTEE OF THE TRANSPORTATION SECTION

Boston, Mass., March 12, 1957

To the Transportation Section
Boston Society of Civil Engineers:

The Transportation Section of the B.S.C.E. held four meetings during the 1956-1957 season as follows:

February 20, 1956.—Joint meeting with Massachusetts Section of the American Society of Civil Engineers. Col. S. H. Bingham spoke on "Transportation for Greater Metropolitan Areas". Attendance—150 members and guests.

April 26, 1956.—Russell Ziegenfelder, partner of the consulting engineering firm of Parsons, Brinckerhoff, Hall & McDonald of New York, presented an engineering report on "Regional Rapid Transit for the Nine Counties Surrounding San Francisco Bay". Attendance—38.

September 28, 1956.—A field trip was conducted by the Massachusetts Turnpike Authority over the Toll Road from Route 128 to Sturbridge. Attendance—128 members and guests.

November 28, 1956.—This meeting was not held because of a conflict in dates with the A.A.S.H.O. meeting being held at Atlantic City.

March 11, 1957.—This meeting was postponed from February 26, 1957 because of the conflict in dates with the North Atlantic States Highway Officials meeting.

Annual Meeting. Officers for the ensuing year were elected.

Chairman	Paul A. Dunkerley
Vice-Chairman	Leo F. DeMarsh
Clerk	Marcello J. Guarino
Executive Committee	Robert A. Snober
	Joseph W. Lavin
	James W. Kelley

James T. Shotwell, Chief of Design Committee of the Urban Section of the Bureau of Public Roads spoke on "Federal Aid Highway Program". Attendance—56 members and guests.

The average attendance per meeting for the year was 93.

While it is recognized that highway transportation is not the sole interest of this Section, it is a major interest and must be given due consideration. Your Committee has found that the present dates allowed for the November and February meetings conflict with major national and sectional meetings of state highway officials. It is recommended that the Transportation Section be given new dates for these meetings which will eliminate this conflict, and at the same time, not conflict with other activities of the Society.

LEO F. DEMARSH, *Clerk*

REPORT OF EXECUTIVE COMMITTEE OF THE CONSTRUCTION SECTION

Boston, Mass., February 4, 1957

To the Construction Boston Society of Civil Engineers:

The following meetings were held during the year:

October 17, 1956.—Mr. George H. McDonnell, President of Tighe & Bond of Holyoke, Mass., spoke on the "Construction of the Manham Dam". Attendance was 34.

January 30, 1957.—Mr. Donald E. McElman and Mr. Peter S. Milinazzo of the Perkins Milton Company presented a film showing construction on the St. Lawrence Seaway Project and discussed the economy of the D-9 bulldozer. Attendance was 39.

The total attendance was 73; average attendance 37.

ALBERT A. ADELMAN, *Acting Clerk*

PROFESSIONAL SERVICES
AND
ADVERTISEMENTS

Volume 44 APRIL, 1957 Number 2

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROFESSIONAL SERVICES

INDEX TO ADVERTISERS

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1957-1958

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Albert Haertlein

HYDRAULICS SECTION AWARD

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Harry L. Kinsel
Olcott L. Hooper

SURVEYING & MAPPING SECTION AWARD

John D. M. Luttman-Johnson, *Chairman*

Llewellyn T. Schofield
Charles L. Miller

TRANSPORTATION SECTION AWARD

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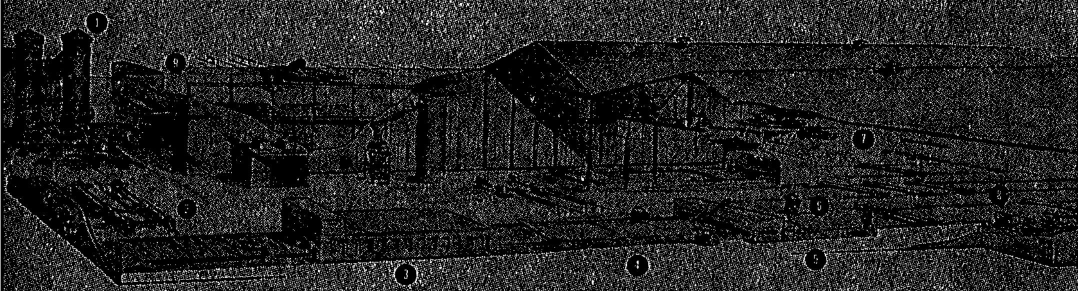

1. CUPOLAS
2. PIPE SPINNERS
3. ANNEALING FURNACE
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5. TAR DIP TANK
6. HYDRAULIC TESTERS
7. CEMENT LINING MACHINE
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100 years ago—WARREN was founded in Phillipsburg, New Jersey, dedicated to manufacturing a *quality cast iron pipe* for transfer of water to a growing nation. For the next 75 years pit-cast pipe in all sizes and in 12-foot laying length served as the industry standard. In fact, even in this atomic age, pit-cast pipe has no peer in larger sizes up to 84" in diameter.

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That 73 utilities are still using cast iron pipe installed over 100 years ago is proof of high quality and dependability of cast iron pipe.

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