PRUDENTIAL CENTER FOUNDATIONS

By Donald G. Ball,* Member

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In Boston's Back Bay, the Prudential Insurance Company is building a complex of high-rise office, hotel, and apartment buildings surrounded by low level shops and a landscaped plaza, Fig. 1. The development will cover 31 acres in the area previously occupied by the Boston & Albany railroad yards and Mechanics Building.

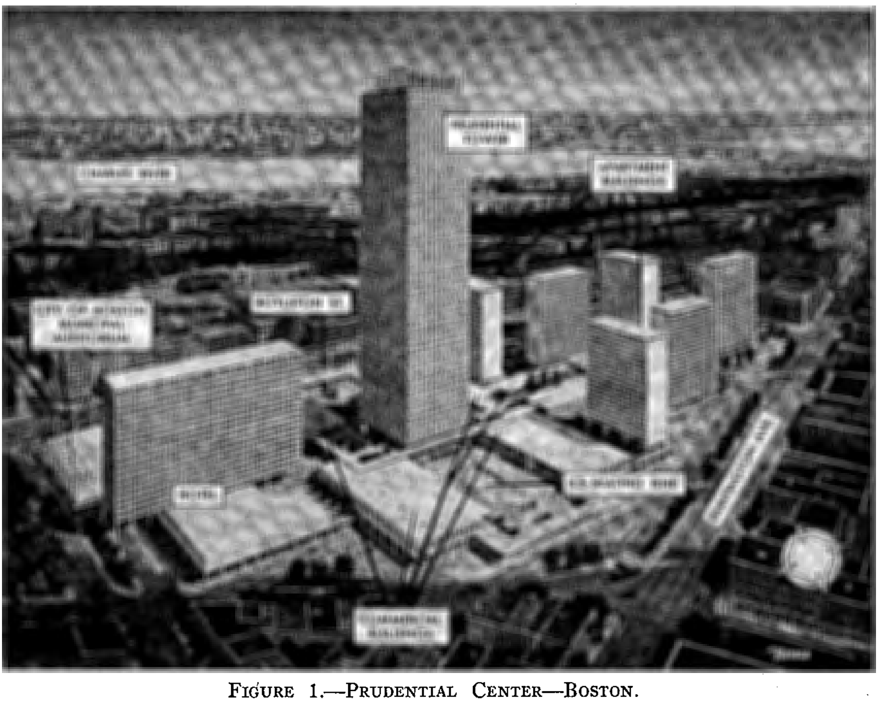

Figure 1.—Prudential Center—Boston.

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History of the Area

The land at the site came into use less than 100 years ago when the offensive mud flats of Back Bay were filled to eliminate the nuisance and to create usable land for the expanding city. Originally Boston was built on a peninsula of much smaller dimensions than it now occupies. It was nearly surrounded by water being connected to the mainland by a narrow neck. The road along the neck is now in the general location of Washington Street running from downtown Boston southwest to Roxbury. Gradually the Boston area was enlarged by filling with borrow taken first from the tops of hills on the peninsula and later from areas to the west.

The first change in topography of the Back Bay came in the early 1800's when two railroads were built across the mud flats on piling and fill; the Providence & Boston railroad from the southwest, and the Boston & Worcester from due west along the line now occupied by the Boston & Albany. About this same period, the Back Bay was used to develop water power, a dam having been built along the present location of Beacon Street. This connected the Boston peninsula with Gravelly Point in the vicinity of Massachusetts Avenue and continued westward across another bay now identified as the Fens. The power developed was from tidal flow, water entering the filling basin from the Charles River west of Massachusetts Avenue when the tide was high and running through mill races into the receiving basin where the Prudential site and Copley Square are now situated. In 1852, earth filling of the basins was started and continued steadily until the 1870's when building construction in Copley Square began.

Description of Project

At the Prudential Center, a plaza will be constructed over the entire site. This plaza will create a new surface level at a height sufficient to give clearance to railroad and toll road easements running below and a large underground garage.

Buildings will occupy about 25 percent of the area. The remaining surface will be devoted to parking and landscaping that will include reflecting pools, terraces, and covered walkways. The Prudential tower, a 52-story structure, will rise more than 750 feet above the ground and become the tallest building in the world outside of Manhattan. At the foot of the tower will be constructed several low buildings to be occupied by banking facilities, restaurants, shops, and retail stores. Two spacious plazas will extend north and south from the base of the tower building to Boylston Street and Huntington Avenue. Plans call for an ice-skating rink on the south plaza.

On the western section will be erected a 26-story hotel. Adjoining the hotel will be the combined exhibition hall and 5,000-seat auditorium of the City of Boston. Prudential has planned a group of six apartment houses each 26 stories high for the eastern section of the site. These will provide approximately 1,750 apartment units. Main tracks of the Boston & Albany railroad will continue to run across the site diagonally northwest to southeast on an easement under the plaza. Adjoining this is a second easement for a six-lane highway, an extension of the Massachusetts turnpike from Weston to Boston's South Station. Combined width of the two easements will be 132 feet. Parking facilities on the surface and on three levels under the plaza will accommodate about 4,000 automobiles. Escalators and elevators will provide access to the various buildings from the parking areas.

Geology

A generalized soil profile, starting from the surface of the railroad yards at an elevation of 11 feet above Boston City Base, shows eight feet of sand and gravel fill over 3 feet of organic silt and peat that formed the old bay bottom. Under this is 20 feet of sand and gravel over 100 feet of Boston Blue clay. The clay rests on three feet of glacial till over bedrock. Average depth from the railroad yard surface to rock is 145 feet.

The bedrock is an argillite, locally known as Cambridge slate. It is a hard, well indurated rock with a fine grain texture and well defined bedding. It has a dip of 40 degrees to the north or northeast. The rock is gray-blue in color, ranging from quite light to dark, and contains many small veins of calcite. The thickness of the slate in the Back Bay is not known, but at places in the Boston basin may exceed 3,500 feet. At the site, the top of rock lies at about elevation minus 135.

Over the rock lies the glacial till or "hardpan." The till extends over most of Boston, varying in thickness from a few feet to probably well over 100 feet. It forms the distinctive hills on the Boston peninsula and islands in the harbor. Boston Blue clay was deposited over the till closely following the retreat of the glacial ice. The clay is of glacial origin and was deposited in marine or brackish water when sea level stood more than 30 feet higher than at present. Subsequently, the clay was exposed to weathering and erosion when the sea level dropped to between 90 and 100 feet below that of the present. Weathering and desiccation formed the stiff yellow clay layer that is generally found at the top of the soft Blue clay.

Groundwater

One of the early problems in developing the design and construction procedure at Prudential was control of groundwater. Lowering groundwater can have two effects on existing structures. One is to increase the load on the underlying soft clay by taking away the buoyant lift of water on the upper soils, thereby causing settlement. The second is to expose untreated wooden piles to the ravages of decay normally held in check by complete and continual submergence.

In 1910 the Charles River dam was completed and has maintained in the Charles River basin a fairly constant water level close to elevation plus eight in contrast to the previous tidal fluctuations from 0 to plus 10. Because the Back Bay is filled land, most of the existing structures rest on wood piles. The City, recognizing the importance of keeping the wood piles immersed, set elevation plus five as the maximum cutoff elevation.

Occasionally the possibility of groundwater falling below plus five was considered but since no serious foundation failure had ever occurred, very little was done to study the problem until 1929. In that year, the Boston Public Library suffered damage due to pile deterioration costing over $500,000 to repair. Needless to say, this caused grave concern to the owners of other structures in the area, and subsequently several groups began regular groundwater observation programs in the vicinity of their own structures, while the City of Boston began a widespread program covering the entire city. Since the damage to the Library, there is no record of any other major foundation failure due to pile deterioration in Boston. Some property owners still observe groundwater near their own structures, but the City program was abandoned in 1939 except for limited specific areas including a few observation wells in Copley Square.

A study was made of groundwater conditions in the Back Bay, past and present. Several sources of data on previous groundwater observations were available. The most comprehensive set of records were those kept by the City during the period 1929 to 1939. These contained readings at least monthly on wells installed throughout the City. Additional data were obtained from the records kept by Trinity Church in Copley Square and the Public Library in Copley Square. To observe the present groundwater trends, regular observations were also begun on thirty-seven existing wells in the vicinity of the site and nine observation wells installed on the site. One set of readings was taken on existing wells within the sector of the City between the Charles River basin and South Bay, from Massachusetts Avenue to Dartmouth and Union Park Streets. From these data, time elevation graphs of groundwater contour plans were prepared.

The elevation at which groundwater will stand in any given location is the result of several variable factors: the relative elevation of ground surface, the runoff coefficient, permeability of the soil and proximity of local influences such as water mains or sewers, and adjacent bodies of surface or subsurface waters. Many of these factors could not be exactly known. In spite of these limitations, certain facts were developed.

It was evident that low groundwater conditions existed near many of the low level sewers due to infiltration. At some points the sewers were more effective in lowering the groundwater level than at others. Although differential fluctuations occurred due to seasonal and other changes, the depressing effect of sewers remains fairly constant. As for the long term trend, a study of the groundwater graphs to date led us to conclude that groundwater in this area is not significantly lower than it was under similar climatic conditions 25 years ago.

A study of the groundwater graphs and contour plans reveals areas where groundwater continually stands higher than either the Charles River basin at elevation plus 8 or mean sea level in South Bay at elevation plus 5.65. These areas are large enough to preclude the likelihood of being caused by water or steam main leaks. Infiltration of groundwater from adjacent higher ground in Brookline or Beacon Hill is possible, but both of these areas are one mile or more away across the flat man-made land of Back Bay. It is also possible that precipitation combined with somewhat impervious soils maintain these areas of higher groundwater. This possibility is further indicated by the high surface of the clay between Tremont and Washington Streets east of Massachusetts Avenue. Whatever their cause, these areas of high groundwater level appear in all the years studied and always in the same general location.

To analyze the possible effects on groundwater of the construction operations, the effects of similar operations in the past were investigated. Two major construction projects in the area were carried out during the time for which records are available. They were the Huntington Avenue subway and the Christian Science Publishing House. The data show that some sections of the Back Bay are less susceptible to drawdown than others presumably as a result of less permeable soil, a perched water table, or a thinner layer of pervious soil on the clay. Large scale dewatering operations lower the groundwater level by a significant amount over extensive areas of Back Bay.

Records indicate that since the filling of Back Bay, groundwater has been fluctuating generally in the range from elevation plus five to elevation plus eight. This range has been normal during the past 20 to 30 years since the Back Bay has been highly developed and its storm runoff coefficient high.

Infiltration from rainfall and the surrounding bodies of water have combined to produce an equilibrium for this heavily developed sector, consisting of approximately 1,000 acres, which the more complete development of 31 acres of the Prudential site is not likely to alter materially.

Subsurface Investigation

The 26-story and 52-story buildings are of such height that it is necessary to carry their foundations down through the soft Blue clay to rock, but the garage and plaza with the low level structures, one and two stories in height, are light enough so that they can be carried by foundations above the soft Blue clay. Because both types of foundations are involved and because of the large area, the extent of the soil investigations on this project exceed in magnitude any that have been carried out thus far in the Boston area. A total of 67 borings were put down, of which 45 were only carried into the top of the Blue clay. Twenty-two borings went down to the rock and continued as core borings 20 feet to 30 feet into the bedrock. In the shallow borings split spoon samples were taken with a standard 2-inch sampler. In the 22 deep borings, 3-inch diameter undisturbed samples were taken with a stationary type piston sampler except in two of the deep holes which were 6 inches in diameter, making it possible to take 5-inch diameter undisturbed samples. From the smaller borings, 2-inch diameter rock cores were taken and from the two larger borings 3½-inch rock cores.

Professors Arthur and Leo Casagrande were engaged as soil and foundation consultants on the project and all samples were tested at the Harvard University laboratories. Of special interest is the following taken from their report of the subsurface investigation. (In the following quotation reference is made to the water-plasticity ratio of the clay. This is the relation of the natural water content of the soil to the water content of the same soil at its plastic and liquid limits. If the natural water content is at the plastic limit the water-plasticity ratio is zero percent; if at the liquid limit, it is 100 percent; half way in between, 50 percent.)

"Although a large amount of work has been carried out in Boston during the past 25 years on development of sampling tools and technique of sampling for taking undisturbed samples of the Boston Blue clay, an excessive percentage of such samples were found to be disturbed on all boring projects. Similar difficulties developed on this project. Even the strictest adherence to what was considered the best procedures, by experienced and reliable boring foremen, did not prevent satisfactory samples and samples showing excessive disturbance from following each other erratically and for no apparent reason. In an effort to analyze the possible cause of disturbance to samples, we requested the contractor to equip himself with dynamometers and to measure the force during sampling. Such measurements were made in connection with the taking of 5-inch diameter samples.

"From a systematic comparison of the penetration resistance as measured with the dynamometer, of the quality of the samples and of the sensitivity of the clay to remolding as reflected by the water-plasticity ratio, we were able to conclude that in general satisfactory samples were obtained only for clay having a relatively high sensitivity, i.e., for water-plasticity ratios of the order of 80 percent or greater; and that for relatively low sensitivities, i.e., for water-plasticity ratios of less than 50 percent, the samples were disturbed excessively. Satisfactory as well as excessively disturbed samples were obtained for clay having water-plasticity ratios between 50 percent and 80 percent.

"The relationship between the maximum load observed on the dynamometer and the water-plasticity ratio has been plotted. It can be seen that for high water-plasticity ratios, i.e., for relatively high sensitivity, the applied force ranged between one and two tons. For low water-plasticity ratios, i.e., low sensitivity, a force of three to four tons was required. Even greater forces were necessary for samples containing stones. From the plotted data, one can also extrapolate that for clays having water-plasticity ratios much greater than 100 percent, i.e., for extra-sensitive clays, much smaller forces than one ton would be required, in fact, in such clays (e.g., the soft Laurentian clay in Quebec and marine clays of Scandinavia) often the weight of the sampling equipment is sufficient to effect penetration because the remolded clay that forms in immediate contact with the surface of the sampling tube acts as a lubricant.

"The probable explanation for the manner in which sensitivity influences sample disturbance is illustrated in Fig. 2. For a clay of low sensitivity, the total friction force that builds up along the outside surface of the sampling

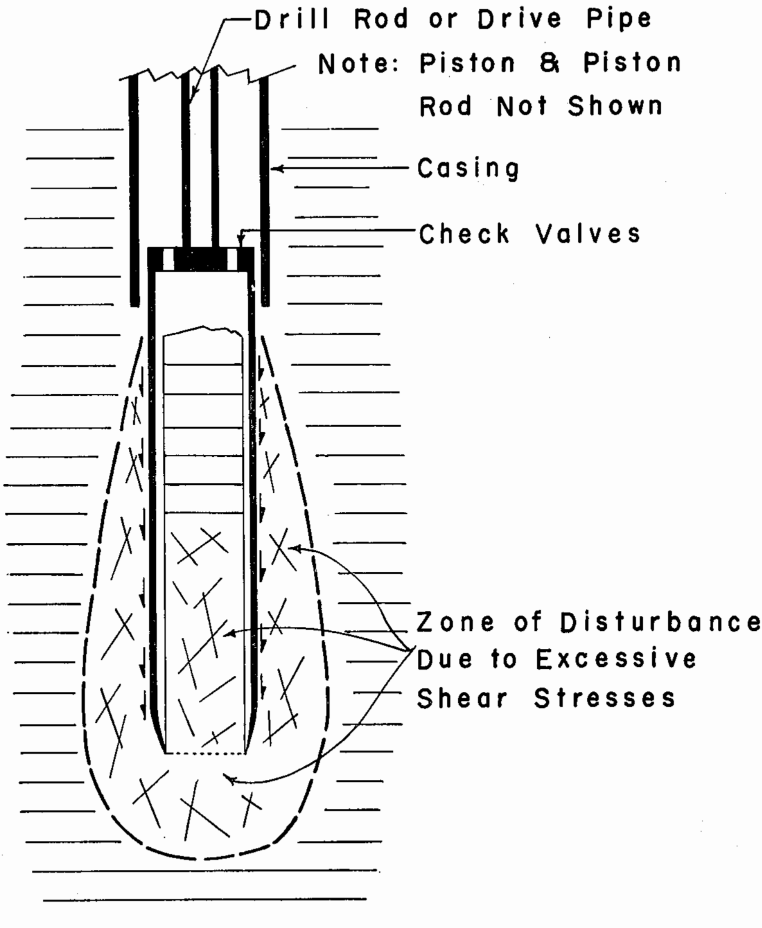

FIGURE 2.—SKETCH ILLUSTRATING SAMPLE DISTURBANCE DUE TO EXCESSIVE FRICTION FORCES TRANSMITTED BY SAMPLER INTO SURROUNDING SOIL. (From Casagrande report.)

tube, while the tube is being pushed into the clay, can reach values which exceed the compressive strength of the clay below the cutting edge. This can readily be checked by computation.

"When the water-plasticity ratios were determined, it was not anticipated that they would be useful in connection with this study. These values were determined for a section of clay a few inches thick, and it may not be representative for the entire length of a sample. On the other hand, the friction that builds up on the outside of the sampling tube is due to the character of the clay over a height of several feet, as well as the presence of sand layers and of stones. Therefore, it is likely that a deliberate effort to obtain all information needed to evaluate the outside friction would develop a clearer relationship than that indicated by the coarse relationship in the plot of dynamometer load and water-plasticity ratio.

"In retrospect, we can now conclude that all development work during the past twenty years has hinged on the assumption that it is chiefly the friction between the sample and the inside of the sampling tube which causes disturbance, and that if this friction were to be eliminated, satisfactory samples would be assured. Hence, almost all important improvements in the design of the sampler and in sampling technique, such as the angle of the cutting edge, the inside clearance, etc., were designed to eliminate this inside friction, and successfully so. But now we realize that in clay of low sensitivity sample disturbance will result in spite of all these precautions because of the excessive friction which is built up along the outside of the sampling tube and which is transmitted in the form of excessive stresses into the clay below the cutting edge. In other words, the clay is already disturbed before it enters the sampling tube."

The report continues with the recommendation that a rotary coring type sampler may be the proper sampling device to use. One of the best methods of determining sample disturbance is described thus:

"A thin longitudinal slice was cut from the center of each clay sample and allowed to dry slowly to the optimum condition for visual inspection. In this state the most plastic clay is still very dark in color, while clays of different degrees of plasticity range in color tone to light-gray, whereas nonplastic silts and sands are then already completely dry and of very light color. In that state representative sections were photographed with process film in order to exaggerate slight differences in color tone."

It can be seen in Fig. 3 that by using processed film and taking the picture at the optimum state of partial drying one can clearly bring out every small detail of stratification and disturbance. In its original state, the samples had fairly uniform gray color and only the sand layers could be identified as such. After complete drying the same samples were also of uniformly light gray color.

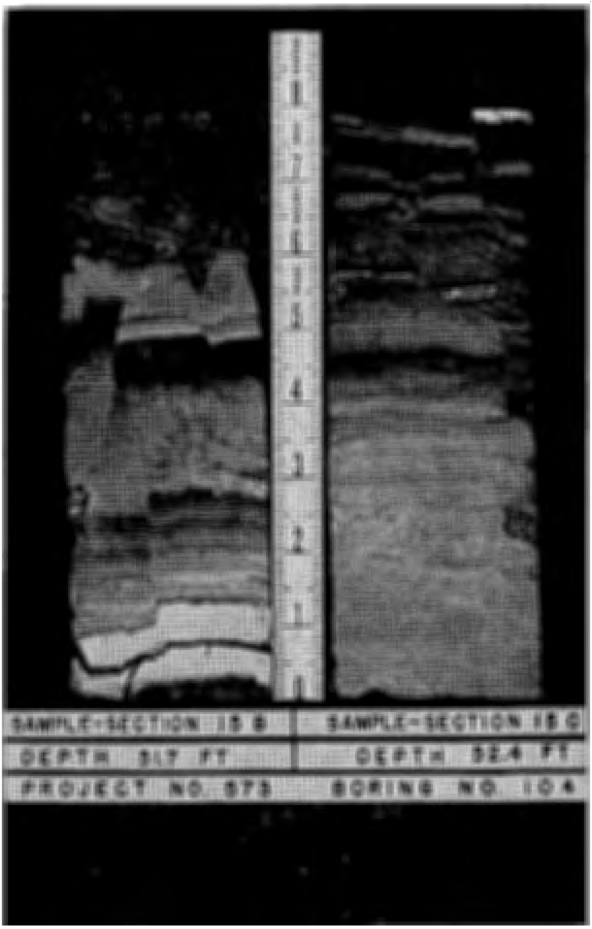

FIGURE 3.—SAMPLE SECTIONS 15B AND 15C: INTENSELY STRATIFIED CLAY, SILT AND FINE SAND; LOWER PART OF 15C IS SILTY FINE SAND; NUMEROUS SHEAR PLANES, PARTICULARLY IN 15B, ARE PROBABLY DUE TO SAMPLING DISTURBANCE. (From Casagrande report.)

FOUNDATION DESIGN

The present contract is only for constructing the foundations for the center section. This is a strip including the tower building running through the middle of the site from Boylston Street to Huntington Avenue. Since the tall buildings are twenty-six stories or over and the low building four stories or less, it is obvious that the foundations for the heavy structures must go down through the clay to bedrock while the light ones can be supported on the sand and gravel over the clay.

Several factors influenced the choice of foundation going to rock: economy, durability (corrosion, if steel were used), and effect of the disturbance of the blue clay on consolidation and settlement of structures immediately adjacent.

In the Casagrande report, it was pointed out that disturbances of the clay due to volume displacement by driving piles to rock would result in additional settlement in a narrow area surrounding the new high buildings. To keep such settlements to a minimum, the types of piles considered were those effecting the least displacement. A cost study narrowed the choice down to two foundation types: steel H-piles and drilled-in caissons.

Bids for the tower building foundation were taken for each as alternates and drilled-in caissons were cheaper by about 20 percent. The latter was chosen for construction. Some steel H-piling is being used to carry loads of intermediate value from the reflecting pool and heavy planter beds immediately around the tower building, but all the tower buildings column loads are to be on drilled-in caissons.

The rest of the structure in the center section (that is, the area around the garage, plaza and low level commercial buildings) is to be founded on a heavy concrete slab covering the entire area resting on the sand and gravel layer that overlies the blue clay. The additional unit load on the blue clay on this area over and above the weight of the material that was removed in making the excavation for the garage ranges from 100 to 300 pounds per square foot. According to consolidation data from tests, the settlement due to this additional load is predicted to be from 2 to 3-½ inches. The load is not uniform, being somewhat greater in the areas of the low level buildings. Some of these buildings are immediately adjacent to the tower building which will be founded on rock so a differential settlement between the plaza area and the tower building is expected. The joints between structures are designed to take this differential settlement without distress.

A complication in the arrangement of the garage is the location of the railroad and highway easement that cuts diagonally across the area. This easement is 132 feet wide and will accommodate two tracks for the railroad and six lanes of the toll road that will come from Route 128 into Boston bringing the traffic from the Massachusetts Turnpike.

To discover if corrosion of steel piling was to be a factor in design, a corrosion survey of the site was conducted. Horizontal and vertical stray current potential gradients were measured and soil and water resistivity and pH measurements were made at several depths. During the soil boring program, zinc reference electrodes were installed in nine test borings. Three electrodes were placed in each of the borings, one 10 feet below the surface, one half-way down to rock and the third five feet above the rock surface. The horizontal and vertical stray current potential gradients were measured at these nine locations. The corrosion consultant, as a result of his study, recommended that all steel H-piles be coated with a coal-tar epoxy resin in the soils above the Blue clay strata. Also that all piles be electrically isolated by avoiding metallic contact with other parts of the structure. The stray currents measured were probably caused by the subways on Huntington Avenue and Boylston Street which bracket the area. It was decided to use cathodic protection in the form of zinc anodes for the steel H-piles in addition to the coating. On the drilled-in caissons only the coal-tar epoxy was used. The zinc reference electrodes are being left in place in the nine boring locations to facilitate corrosion testing of the piles at a later date after construction is completed.

Drilled-in caissons are new to the Boston area. With heavy column loads such as we have at Prudential, the drilled-in caisson seems an ideal type of foundation. The caisson shells for the tower building are 30 inches in diameter, ½ inch thick, steel cylinders driven down to the top of rock. Soil inside the shell is excavated and a 29-inch diameter socket drilled into the rock for a distance of 16 to 23 feet. A heavy steel H-section is lowered into the caisson down to the bottom of the rock socket. This H-section extends from the bottom of the socket to the top of the steel shell. The space between the steel core and the shell is then concreted. Stress is carried by all members. Design unit stress in the steel shell is 8,000 pounds per square inch; in the concrete, 900 pounds per square inch; and in the steel core, 15,000 pounds per square inch. The depth of the rock socket is dependent on the total load carried by the caisson. Caissons for the tower building carry from 1,300 to 1,650 tons each. Four caissons are used for each of the heaviest columns. For design purposes, it is assumed that the load carried by the caisson is distributed to the bedrock by end bearing of the steel H-member in the bottom of the socket and by both end bearing of the concrete filling the caisson and by bond stress of the concrete against the rock socket. In accordance with the requirements of the Boston Building Code, the design load taken by the bedrock in end bearing of the concrete and steel should not exceed 70 tons per square foot. The remainder of the stress is assumed to be transferred to the rock by bond of the concrete at a unit stress of 100 pounds per square inch. It is recognized that these values are very conservative.

Construction

One interesting problem concerned with the construction phase has been the control of groundwater. In order to protect the foundations of the existing structures in the area, which are principally on untreated wooden piles, it was specified that the contractor drive steel sheet piling completely around the working area down to clay to form a tight cofferdam. He was also required to install a backfeed system outside the steel sheet pile cofferdam to return water, pumped from the inside, in order to maintain the level of groundwater in the area.

In general, the sheet piling was driven to the clay without much difficulty except along one section of West Newton Street near the former location of Mechanics Hall. Here the sheet piling ran into subsurface obstructions that made it impossible to get the piling seated into the clay. To tighten areas where piling was not reasonably watertight, neat cement grout was pumped into the soil outside the piling just above the clay layer. It has inhibited the flow to a rate which has made it possible to maintain proper groundwater levels outside the cofferdam. Present rate of pumping is about 500 gpm. At the beginning of construction, backfeed water was placed in horizontal perforated pipes buried just outside the sheet piling at the elevation of the top of the water table. It was found that the original Back Bay bottom made up of mud and organic silt and peat formed a horizontal barrier in some areas keeping the upper water table perched and apparently unaffected by pumping, while groundwater levels in sections some distance away from the site responded immediately to pumping from inside the sheeting. To overcome this, most of the backfeed water is now put down below the silt layer into the sand and gravel above the clay. Daily groundwater readings are taken in observation wells around the site. When lower groundwater is observed, the contractor either stops pumping or adds to or revises the backfeed system.

Through a period of trial and error a backfeed arrangement has been developed which is now effectively maintaining the groundwater levels above elevation plus five while water is drawn down inside the sheeting to elevation minus 12.

An early problem discovered in the combination dewatering and backfeed system was the clogging of backfeed points with an organic slime which formed quite rapidly in the pipes as the water was being transferred from the suction to the backfeed. As the organic slime built up, it broke free in chunks and clogged the points. To overcome this, the contractor installed a chlorination unit treating the water as it leaves the well point pumps. The treated water is passed into a sedimentation tank and there picked up by a pressure pump putting it into the backfeed system. Even with this treatment, periodic cleaning of the backfeed points is necessary. At present, they are being cleaned every two weeks by disconnecting and reversing the flow.

A fluorescent dye was an effective tool in locating places in the steel sheet piling where the principal leakage was taking place. The dye was introduced into the observation well that was most affected by pumping. In some cases this was some distance from the sheeting. Additional observation wells were put inside the excavation just inside the steel sheeting, and samples taken periodically until the dye showed up. Inspection of samples under ultraviolet light made it possible to detect very small quantities of dye.

In the caisson construction, a pervious bedrock combined with a high water table has made inspection of the drilled rock sockets impractical to accomplish in the dry. It is expected that a few caissons can be pumped out for direct inspection by a man lowered into the bottom, but in general most of the caissons have too rapid an inflow of water. To carry out a satisfactory inspection of these sockets, an underwater television apparatus, Fig. 4, was specified capable of operating in about 160 feet of water. The use of underwater TV is not new, but we believe that this is the first time it has been used for inspection of caisson sockets. The apparatus has just been given its first trial run. Fig. 5 is a photograph of the TV monitor screen showing the lower edge of a caisson shell and the upper part of the rock socket.

Coordinating Architect for the project is Charles Luckman Associates; Associate Architect, Hoyle, Doran & Berry; Foundation Engineer, Metcalf & Eddy.

Center section foundation contractor is George A. Fuller Co.,

FIGURE 4.—UNDERWATER TELEVISION CAMERA DESIGNED TO OPERATE 160 FEET BELOW WATER SURFACE.

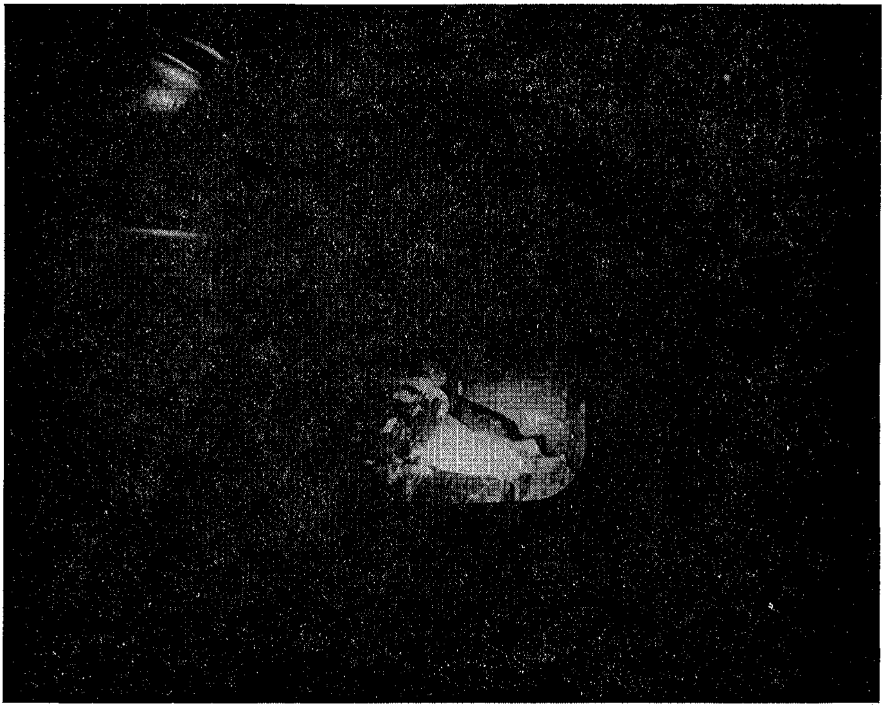

FIGURE 5.—PHOTO OF TV MONITOR SCREEN. UNDERWATER TV CAMERA IS VIEWING UPPER PART OF DRILLED-IN CAISSON ROCK SOCKET.

with subcontractor Western Foundation Co. putting in the drilled-in caissons.

Borings were done by C. L. Guild Construction Co., with H. A. Mohr as consultant. Seismic survey was by Weston Geophysical Engineers. Corrosion survey was by Electro Rust-Proofing Corporation.

Arthur and Leo Casagrande are soil and foundation consultants.