The New Boston Outfall

Consisting of a 9.5-mile tunnel with a 6,600-foot diffuser and 55 risers, the outfall is designed to minimize adverse environmental impacts at the lowest cost and with maximum dependability.

DOMINIQUE N. BROCARD, BRIAN J. VAN WHEELE & LAWRENCE A. WILLIAMSON

Many studies and analyses led to the siting and design of the tunnel outfall that will discharge treated effluent from the new Deer Island wastewater facilities approximately 9.5 miles offshore in Massachusetts Bay. This outfall is an important element of the overall Boston Harbor Project, both in terms of expected environmental enhancement and cost.

Siting

Wastewater from the 43 communities that are serviced by the Massachusetts Water Resources Authority (MWRA) is presently discharged offshore of the two primary treatment plants located on Deer Island and Nut Island (see Figure 1). Effluent discharges through several multiport diffusers in entrance passages to Boston Harbor where the higher currents and water depths provide dilution water to the effluent. In addition, each plant has emergency pipe outfalls that are situated in relatively shallow water. These outfalls are utilized for discharging the high flows that occur during rain storms.

One of the shortcomings of the existing outfall system is that during flood tide effluent is drawn into Boston Harbor, an area ill-equipped to assimilate pollution because of its confined and quiescent nature compared to open waters. The discharges are also in close proximity to areas of high value for recreation, fishing and shellfishing. Therefore, one of the goals of the new outfall was to discharge the effluent in the general circulation system of Massachusetts Bay sufficiently far from the harbor and the coast.

The outfall siting was based on extensive field studies and mathematical modeling, with oversight by regulatory agencies and input from independent experts, communities and citizens groups. The basic siting criteria were:

- Compliance with Massachusetts and U.S. Environmental Protection Agency (USEPA) water quality standards and criteria;
- Acceptable impacts on shorelines and seafloor (benthos);
- Constructability;
- Operability; and,
- Cost.

The bulk of this work was performed during 1987 as part of the facilities planning phase of the project,[1] with piggy-back analyses done for the Environmental Impact Statement (EIS).[2]

The siting process was structured around the initial selection of five alternative outfall sites (shown in Figure 1). These sites were lo-

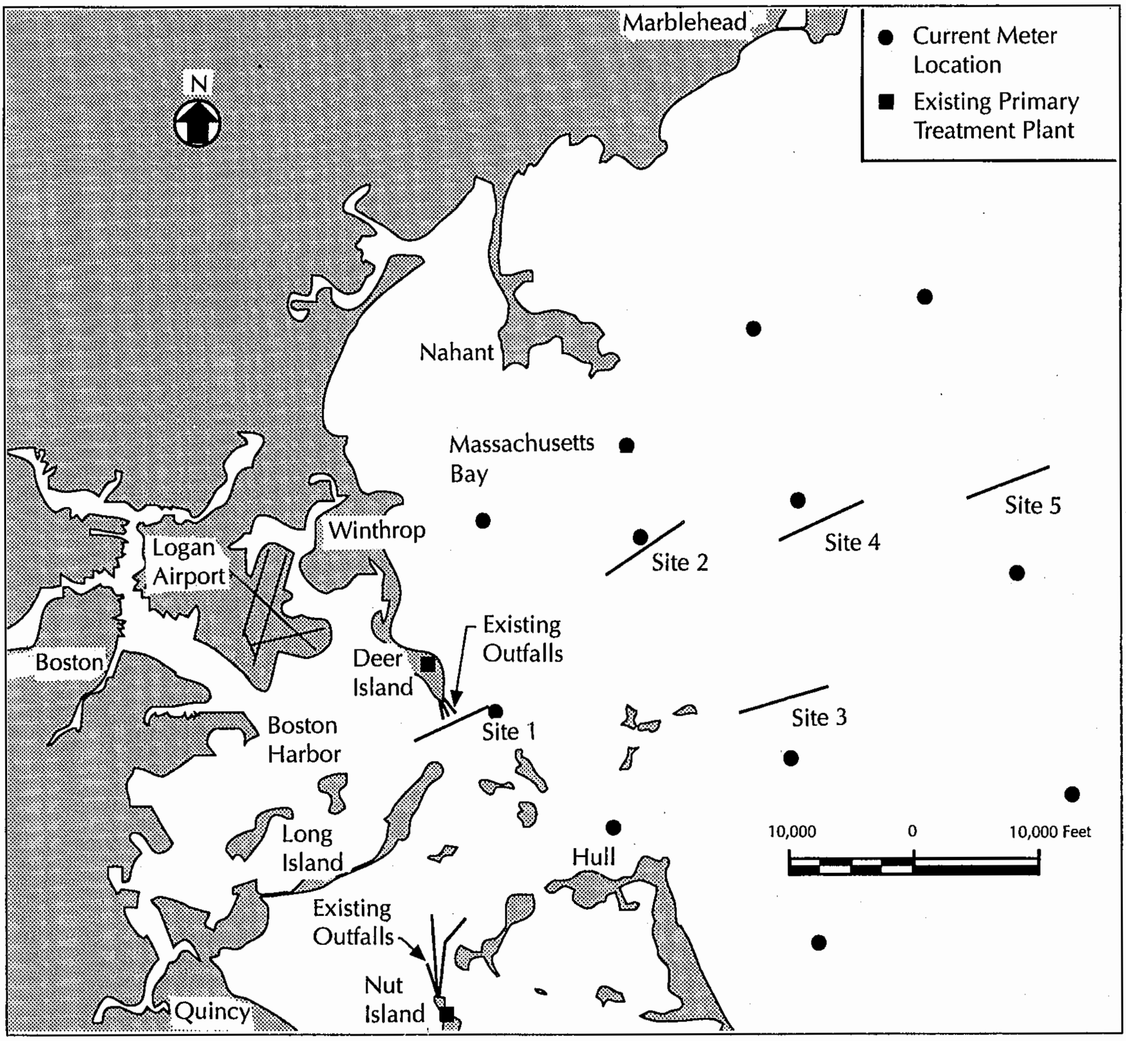

FIGURE 1. Site map with alternative outfall sites and current meter stations.

cated in areas of relatively flat ocean bottom. However, they were only intended to represent different general areas within the range of feasible options. The final position of the outfall was established within the selected area following detailed bathymetric and geophysical surveys.

The design parameters for the outfall included a flow range of 320 to 1,270 million gallons per day (mgd) of secondary-treated wastewater, with an interim period of five years during which the effluent would only receive primary treatment.

Environmental Field Studies. The field studies were used to complement existing data for the characterization of Massachusetts and Cape Cod bays, and for input to the mathematical models. The field studies included:

- Deployment of current meters and tide gages;
- Tracking of drogues and drifters (floating markers carried by the currents);
- In-situ vertical profiling for temperature, salinity and dissolved oxygen;
- Water sampling for chemical analyses; and,
- Biological reconnaissance.

The current measurements were particularly important. Current speed and direction were recorded at 15-minute intervals using moored current meters at 13 stations (shown in Figure 1). Most offshore stations included an upper and a lower current meter and the total deployment period was nine months (although all stations were not covered for the entire period). Massive quantities of data resulted from this measurement program.

Instantaneous current speeds and directions are important parameters for determining the initial dilution that will be achieved at the outfall. Instantaneous currents were characterized by time history plots, frequency distributions and scatter plots.

A major driving force for currents is the tidal influence. This component was evaluated by Fourier analysis of the records and the plotting of tidal ellipses (end of velocity vector at one location as a function of time during the tide cycle).

Net drift is the principal means by which effluent is removed from the discharge area. Thus, net drifts are very important. Tides tend to produce symmetrical current patterns (ellipses) with no net drift, resulting in a background buildup of effluent in the discharge area. Net drift results from large-scale circulation patterns, storms and fresh water discharges as well as non-linearities in the driving forces such as inertia and friction. Net drift in the study area was assessed by progressive vector plots and by calculating running averages of current speed over a period of exactly ten tide cycles. The results revealed that periods with relatively negligible net drifts could exist, and those periods were considered in the mathematical modeling analyses.

Periods of sustained shoreward currents were also identified in order to evaluate shoreline impacts.

Several of the current meters also had the capability of measuring and recording temperature and dissolved oxygen concentration at 15-minute intervals.

Stratification is another phenomenon that greatly influences the fate of effluent. Starting in late spring and lasting into the fall, the waters of Massachusetts and Cape Cod bays are stratified. An upper layer of warmer (and sometimes slightly fresher) water lies over the colder, denser bottom waters. These two layers are separated by a thermocline that inhibits the vertical exchange of water. During stratification periods the effluent tends to remain trapped below the thermocline. While this effect is positive relative to shoreline and beach impact, the available height of rise of the effluent plume is also reduced (which decreases initial dilution). Stratification was documented by vertical salinity and temperature profiles along several transects that were surveyed several times during 1987. The results indicated a thermocline depth of ten to 15 meters during the summer with significant tidal fluctuations.

Water samples were collected for chemical analyses to characterize the background conditions. These surveys revealed that there were high background levels of copper and PCBs. The latter were in excess of the USEPA human health criteria. These criteria correspond to the level at which cancer risks are increased by one in one million for daily lifetime consumption of seafood from the area.

Mathematical Modeling. The purpose of the mathematical modeling was to predict the distributions of effluent constituent concentrations for the different outfall options. Two models were used for the analyses:

- A "nearfield" model to predict the initial dilution that would occur as the effluent plume rises through the water column; and,
- A "farfield" model to simulate the transport, dispersion and decay of constituents in Massachusetts Bay.

The nearfield model was run for combinations of five discharge flowrates, three current speeds and nine stratification profiles, the latter two factors being dependent on the outfall site.[3] Based on the field measurements, a probability of occurrence was associated with each of these parameters from which the probability of each of the resulting 135 sets of conditions was determined (by assuming that the base parameters were statistically independent). Based on the diffuser characteristics of other comparable discharges, a diffuser length of 2,000 meters was selected and kept constant for all the simulations. The results of the nearfield modeling indicated that the lowest initial dilutions were comparable at the different sites, but

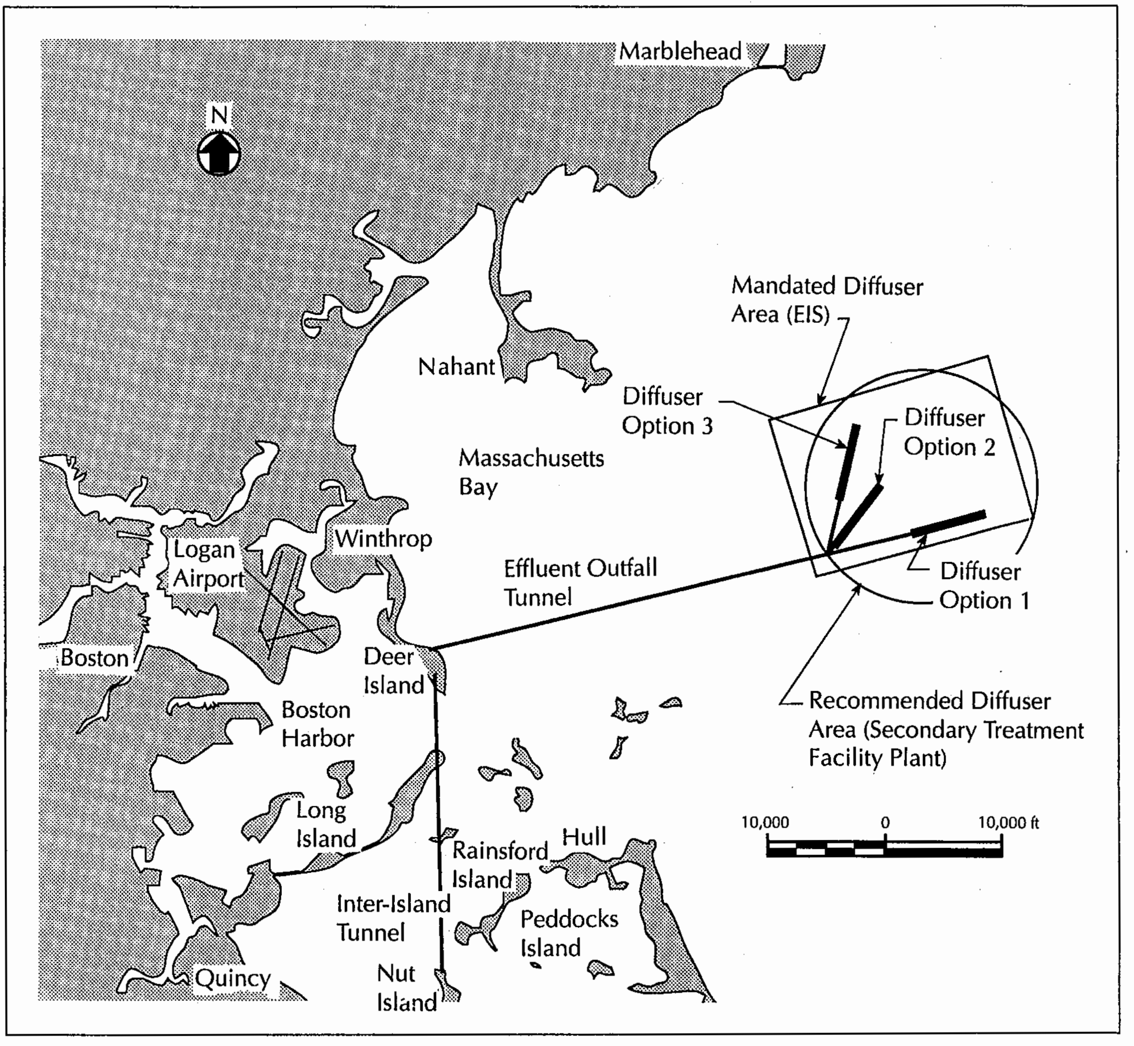

FIGURE 2. Recommended diffuser siting areas and alternative diffuser options.

those occurred less frequently at the offshore sites.

The farfield modeling was conducted using two-dimensional (vertically averaged) finite element models. These models, one for hydrodynamics[4] and one for transport,[5] were used to calculate:

- Background build-ups of toxic constituents in the discharge area;
- Depletion of dissolved oxygen; and,
- Effluent solids deposition.

The different toxic constituents in the effluent were divided into three classes with half lives of 20 days, 60 days and infinity (conservative constituents). These half lives represented decay due to several mechanisms including vaporization, hydrolysis, photolysis and biodegradation. Solids were divided into four classes with different settling velocities and different decay rates were used for carbonaceous and nitrogenous biochemical oxygen demand (BOD). Simulations were conducted with an average net drift as well as with zero net drift. For stratified conditions, transport calculations were performed for the lower of the two layers (below the thermocline) within which the plume would be trapped.

Site Area Selection. For each of the alternative outfall sites, evaluations were performed based on the discharge of primary effluent (which is to occur between 1995 and 1999) as well as the discharge of secondary effluent.

To evaluate compliance with USEPA water quality criteria, toxic constituent concentrations were calculated at the edge of the mixing zone using the results of both the nearfield and farfield models. Based on extensive influent concentration records, 100 different discharge rates with appropriate statistics were considered for each constituent. Combined with the 135 sets of nearfield conditions and two net drifts, a total of 27,000 cases was considered, each with an associated probability of occurrence. Compliance was evaluated for each constituent by determining the concentration that would be exceeded with a probability of one day in three years and four days in three years. The data from these analyses were then compared with the chronic and acute toxicity criteria set in the USEPA Gold Book.[6]

Distributions of dissolved oxygen deficits were calculated using the farfield models, accounting for carbonaceous and nitrogenous BOD, sediment oxygen demand and surface reaeration. Calculations were made for unstratified and stratified conditions as well as for resuspension events during which additional oxygen demand is exerted by resuspended sediments.

An important issue relating to the outfall was the potential for detrimental impacts such as algal blooms due to the discharge of nutrients. Nitrogen was considered the critical nutrient relative to eutrophication and all forms of nitrogen in the effluent were considered available for phytoplankton growth. In the model, as a conservative assumption, nitrogen was assumed not to decay but recycle internally from one usable form to another, the only loss being by export from Massachusetts Bay. Predicted nitrogen concentrations with the discharge, C_N, were then compared to values reported in the literature, particularly investigations conducted at the University of Rhode Island/EPA Marine Ecosystem Research Laboratory (MERL).[7] From these comparisons, areas of increased primary productivity, where C_N is between 0.14 and 0.57 milligrams per liter (mg/l), were delineated, as well as areas of degradation due to excessive phytoplankton growth (C_N greater than 0.57 mg/l). For the selected discharge site, these areas were quite small and limited to the immediate vicinity of the outfall.

Potential impacts on the benthic environment due to the deposition of organic material were also evaluated. Deposition rates of particulate organic carbon were calculated for each discharge alternative and, as with nutrients, these values were compared to values reported in the literature and MERL investigations.[8] These comparisons were used to delineate areas where increased density of benthic invertebrates (deposition rate of 0.1 to 1.5 grams of Carbon per square meter per day [$gC/m^2/day$]) and degraded benthic conditions (deposition rate greater than 1.5 $gC/m^2/day$) could be expected.

As a result of the environmental analyses, an area for the diffuser siting was established in about 100 feet of water. Actually, the facilities plan and EIS resulted in two distinct areas (respectively circular and rectangular), but those areas were in the same general vicinity and overlapped to a large extent (see Figure 2).

It was also decided, for environmental as well as constructability reasons, that the outfall would be a tunnel with vertical risers leading to multiport discharge structures on the ocean floor. The option of a buried pipe was rejected in part because of its environmental impact on the ocean bottom, but mainly because weather conditions would preclude year-round construction, making it difficult to meet the court-mandated schedule. The option of a single large riser at the end of the tunnel leading to buried pipe diffuser was also considered. This configuration would have eliminated the issue of seawater purging. However, it was considered excessively difficult and costly to build. Also, it did not minimize environmental impact on the ocean floor.

Within the selected site area, three outfall alternatives (shown in Figure 2) were developed based on the requirement for a near-horizontal seafloor in the diffuser area. The final diffuser location (Option 1) was established based on the results of the geotechnical investigation program.

Recently, following opposition to the proposed outfall siting from some residents of Cape Cod, a formal biological assessment was

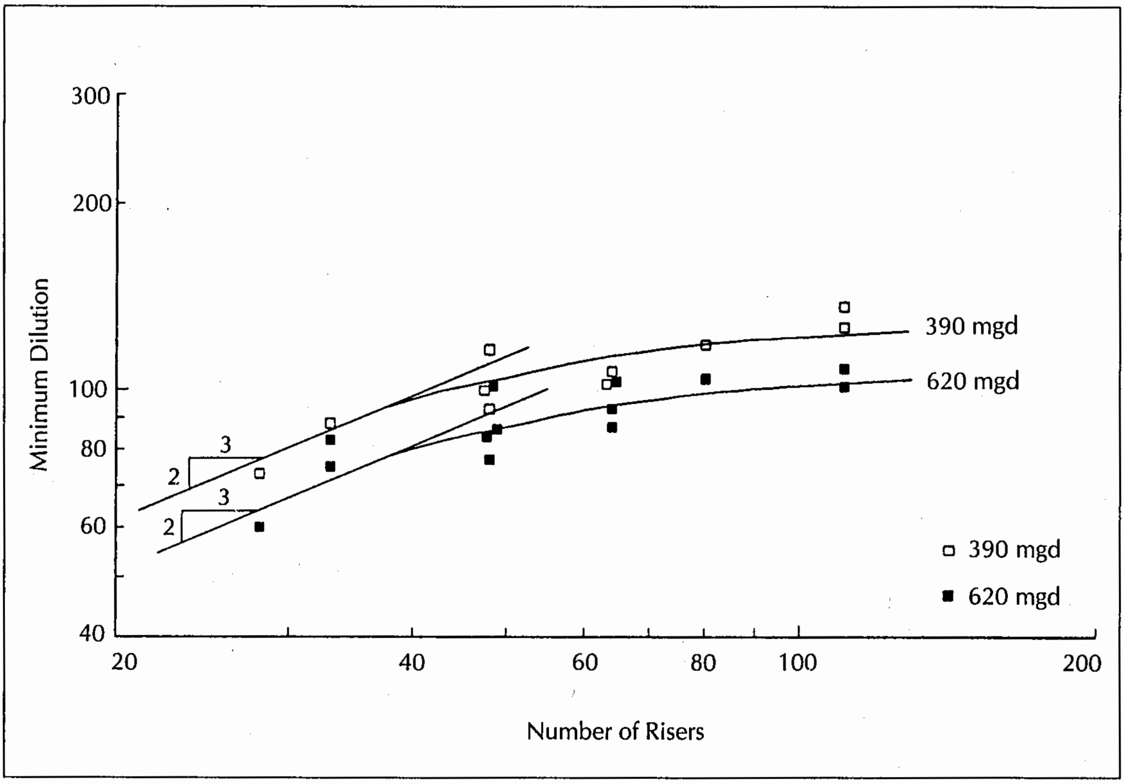

FIGURE 3. Minimum nearfield dilution as a function of riser number (late summer stratification, zero current speed).

conducted by the USEPA that concentrated on the potential impacts of the outfall on endangered species.[9] The main endangered or threatened species of concern were the right, humpback and fin whales as well as the Kemp's ridley and leatherback turtles. The biological assessment sought to establish whether these species would be affected negatively by modifications of the food chain resulting from increased nutrient and toxic chemical loadings to Massachusetts Bay.

The biological assessment concluded that the outfall relocation would not significantly change the nutrient loading to Massachusetts Bay since most of the nutrients now discharged by the existing outfalls are flushed out to the bay. In the bay, only a local increase of nutrient concentration would result near the outfall. Thus, it was concluded that the outfall would not likely alter the food web that the endangered and threatened species depended on. The assessment also concluded that the new outfall was not likely to increase the incidence of toxic phytoplankton blooms such as red tide, or cause concentrations of toxic chemicals likely to affect the endangered species. The biological assessment was accepted by the National Marine Fisheries Service, with recommendations for increased monitoring.

Hydraulic Design

Number of Risers. Multiport diffusers for wastewater discharge typically involve numerous discharge ports, approximating a continuous line discharge. With the selected outfall configuration and an estimated cost on the order of $1.5 million for each riser, their number needed to be minimized.

An initial estimate of 80 risers was introduced in the facilities plan, but physical hydraulic model studies were commissioned during the preliminary design phase to optimize this number. The objective was to determine the minimum number of risers that could be used without impairing the nearfield dilution performance of the diffuser.

The physical modeling was conducted in a large flume (four feet deep, eight feet wide and 200 feet long) at the USEPA Atmospheric Testing Laboratory in Research Triangle Park, North Carolina.[10] This flume could be stratified by filling it slowly with water containing varying concentrations of salt. Sections of diffusers with different riser spacings were built and mounted on a trolley that could be dragged along the flume to simulate currents. For practical reasons, the model was inverted, discharging a negatively buoyant effluent near the surface. The discharge flow contained a known concentration of dye and dilution was determined by measuring dye concentrations in water samples withdrawn downstream of the zone of initial mixing.

Data from the testing indicated that the diffuser performance was not impaired if the number of risers was reduced to 55 (see Figure 3), yielding a considerable cost savings compared to the initial design. In general, the measured dilutions were close to those predicted by the nearfield mathematical model, from a minimum of 56 for peak design flow during periods of maximum stratification and zero current speed to 560 or more for low flow, unstratified receiving water, with currents. The testing also indicated that eight nozzles per riser provided better dilution than twelve, a somewhat counter-intuitive result.

Seawater Purging. At commissioning and following any effluent flow stoppage, the outfall will be full of seawater. Commissioning without letting any seawater enter the outfall would be theoretically possible, but is not practical since the effluent flowrate would need to be gradually increased while the 440 nozzles are successively uncapped. When the effluent flow stops, or drops below a very low value (on the order of 150 mgd for the final design), seawater will intrude into the outfall. As effluent is introduced in the seawater-filled outfall, it will displace some of the seawater, but some may remain. Under these conditions, a situation can develop where effluent discharges out of some risers while seawater is drawn into the outfall through other risers.[11] This complex situation is due to the density difference between seawater and the effluent and the large height of the risers (on the order of 250 feet).

To purge the risers, the static head (pressure) in the tunnel (measured in feet of water) must exceed the static head at the sea floor by at least $(\Delta\rho/\rho)H$, where $\Delta\rho$ is the density difference between seawater and effluent, ρ is the effluent density and H is the height of the risers. This condition is called the Munro criterion. This head difference (6.75 feet for 250-foot risers) is achieved through head losses in the risers and mostly at the nozzles. These head losses are a function of the flowrate and, thus, for a given design, purging will be achieved at a fixed flowrate. To avoid long delays before the outfall is purged, the purging flow should be relatively low, but this implies high head losses at high flows. Therefore, the purging flow is constrained by the overall head available.

For the Boston outfall the purging flow was selected at 1,000 mgd. To verify the Munro criterion and investigate alternative means of purging the outfall, a physical hydraulic model study was commissioned. This study was conducted at the Massachusetts Institute of Technology. Among other results, the study led to the inclusion of a venturi restriction in the tunnel just before the first riser. This venturi slightly reduces the purging flow, but it also allows purging to be effected by rapidly dumping the content of the chlorination tanks into the outfall.

Conveyance Capacity and Tunnel Diameter. The outfall was designed to allow a peak discharge of 1,270 mgd during 100-year storm conditions, with a corresponding seawater level of 116 feet (MDC Datum). The water level in the chlorination tanks effluent channel was set at el. 140.4 feet, so that 24.4 feet of head were available to drive the flow. Of these 24.4 feet, 3.1 feet were used to compensate for the hydrostatic head due to the density difference between effluent and seawater. The remaining 21.3 feet were to provide for friction and other head losses in the tunnel, risers and discharge nozzles. The option of using an effluent pumping station was considered but rejected because of its inherent complication and cost.

Another design criterion was a minimum velocity of one foot per second in the tunnel to prevent the deposition of solids. This criterion led to a maximum tunnel diameter of 25.1 feet. The tunnel diameter finally selected was 24.25

feet, giving a maximum tunnel head loss of 9.2 feet and leaving 12.1 feet for riser and exit losses at peak flow.[12] In turn, this design resulted in a purging flow of approximately 1,000 mgd. To maintain the flow velocity in the diffuser area, the tunnel cross-section area was gradually decreased. The details of the diffuser design, including nozzle diameters and elevations, were developed based on extensive computer modeling. An objective was to have approximately uniform flow distribution among all the risers and to minimize the flowrate below which seawater intrusion would occur. This intrusion flowrate was calculated to be on the order of 150 mgd, well below the normal minimum flow of 320 mgd.

Since the outfall was designed to pass the peak design flow at extreme high water level, excess head was available under more normal conditions. To recoup some of this energy, a hydropower station was designed that would be located between the chlorination tank and the outfall shaft. This hydrostation will include two one-megawatt bevel gear bulb turbines.

Geotechnical Investigations

Geology. The Boston Basin is composed of local marine sedimentary deposits overlying the sedimentary and volcanic rocks of the regional basement rock. Of the rock formations making the Boston Basin group, the Cambridge argillite is the only one observed along the outfall alignment. The Cambridge argillite is a grey, layered, slightly calcareous rock (slightly metamorphosed mudstone or siltstone) with bed thicknesses generally ranging from 0.05 to 3 inches and occasionally up to 5 feet. Volcanic flows of ash fall tuffs, with average thicknesses of 0.05 inch to about 1 foot, are occasionally interbedded with the argillite. Occasional tuff deposits are up to 275 feet thick. Intruding the Cambridge argillite are igneous dikes and sills, predominantly of diabase, with minor amounts of basalt and/or andesite, and felsite.

The overburden material and thickness of the different layers is controlled primarily by the bedrock topography and the erosional processes that were active during the glacial period. The following soil types are present over the outfall alignment:

- Sand: silty fine to medium sand, with some clayey silt, often mixed with gravel and cobbles;
- Silty Clay: olive grey silty clay, medium to high plasticity, occasional lenses of sand, medium to stiff consistency; and,
- Till: unsorted mixture of fine to coarse gravel, fine to coarse sand, and clay.

An understanding of the geologic setting with its inherent complexity and variations was the basis for formulating an accurate set of design documents and, more importantly, successfully constructing the project. The marine exploration program sought to improve this understanding using borings and a variety of geophysical techniques.

Boring Program. A total of 56 offshore borings were made from the conceptual design phase in 1988 to the final design in 1989. During the summer of 1989, a total of 31 borings were drilled along the tunnel alignment with an average spacing of approximately 1,000 feet. Thirteen borings from the 1988 site investigations were located less than 1,000 feet from the alignment. All boreholes were vertical and were offset from the tunnel alignment by a minimum of 200 feet. There were three exceptions, however, where the offset was between 5 to 20 feet from the tunnel centerline.

The borehole information was augmented by conducting 94 vibracores of surface sediments and 15 piezocone tests in order to determine overburden properties. Borehole tests included oriented cores, in situ stress, permeability and downhole geophysics. Samples of soil and rock were obtained from the borings and vibracores. These samples were analyzed for physical characteristics and tested in the laboratory to determine their range of engineering properties and establish design criteria.

Geophysical Surveys. A detailed marine geophysical survey was also conducted during the summer of 1989. The program provided a continuous stratigraphic profile along the alignment as well as areal information in the diffuser section. The information gathered included:

- Bathymetric contours of the seafloor in the diffuser areas, isopach maps of sediment and till layers;

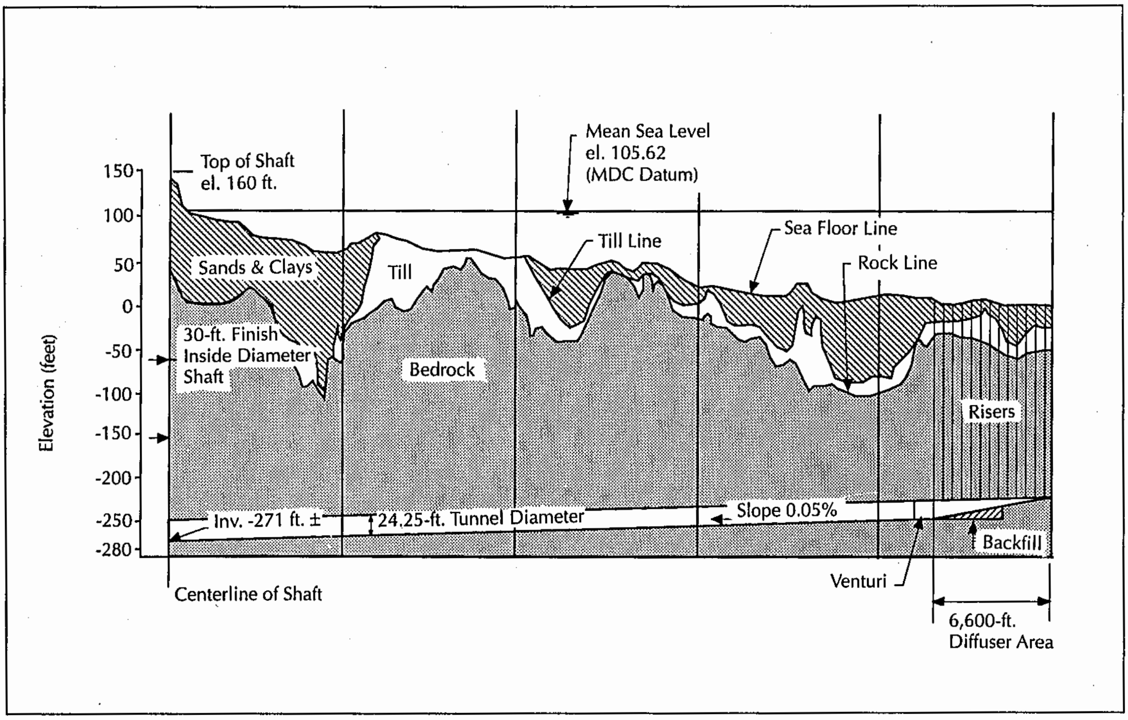

FIGURE 4. Outfall tunnel profile.

- Contour maps of the bedrock elevation; and,
- Geologic profiles along the alignment and in the diffuser area.

The elements of the geophysical survey were:

- Seismic Reflection: Three energy levels were employed to differentiate the upper and lower sediment layers, and the rock surface.
- Seismic Refraction: This test used a 250 cubic centimeter air gun and low-frequency hydrophones on the sea floor connected to sonobuoys that transmitted the data to a shipboard receiver. Seismic refraction covered the same area as the reflection surveys to further define the sediment layers.
- Side Scan Sonar & Magnetometer: Surveys were performed as part of the historic and archaeological searches.

To supplement the single-channel geophysical survey, a multi-channel seismic survey was performed to determine if major structural discontinuities, igneous intrusions or stratigraphic contacts could be identified at the tunnel elevation. Results of the survey revealed evidence of some acoustic anomalies and discontinuities at depth in bedrock. Some of the features identified correlated with zones of more highly fractured rock in the borings. However, in the diffuser section, few anomalies were identified with the seismic data.

Final Design

Tunnel. The outfall tunnel was designed with a finished inside diameter of 24.25 feet and a length of 8.15 miles from the outfall shaft to the beginning of the diffuser area. In general, the Cambridge argillite bedrock quality was determined to be good or excellent, requiring little or no support. However, it was realistic to assume that zones of poor quality rock would be encountered.

A profile of the tunnel and diffuser is shown in Figure 4. The tunnel begins on Deer Island over 400 feet below the ground surface and rises on a small upslope grade of 0.05 percent that is designed to facilitate the drainage of infiltrating water. The depth of the start of the tunnel was determined so that the crown of the tunnel would be at least 100 feet below the top of rock. This cushion is equivalent to three times the excavated tunnel diameter plus a margin of about 20 feet.

Two lining options were designed: a fully precast lining and a cast-in-place lining. The design criteria included:

- Hydraulics: Manning's coefficient of 0.016;
- Watertightness: The combined system consisting of the lining, the contact grouting, and the surrounding rock, grouted if required;
- Durability: Minimum design life of 100 years; and,
- Loading Conditions: Maximum external water pressure at the deepest point when dewatered would be approximately 162 pounds per square inch (psi) and maximum internal water pressure under normal operating conditions plus wave transients of 25 psi in excess of the external pressure.

The contractor chose the fully precast lining using six tapered precast concrete segments (see Figure 5). Each ring is connected to the previous one by a series of dowels.

The tunnel is being bored through the Cambridge argillite using a tunnel boring machine (TBM). A new TBM was built by the contractor for this purpose (see Figure 6). To meet the court-mandated deadline, the tunnel advance rate would have to exceed 100 feet per day. The TBM penetration rate depends primarily on:

- Rock hardness, strength, and resistance to cutter penetration;
- TBM mechanical and operational variables such as cutter load, cutter head rotational rate, available torque and muck cleaning efficiency; and,
- TBM operator skill.

The design also concentrated on the potential for water inflow. Although conditions appear to be benign and great water inflows are not expected, the specifications provide for probehole drilling in certain areas and the grouting of pervious zones that could cause difficulty in tunneling.

Risers. The diffuser section starts 8.15 miles from the Deer Island shaft and is 6,600 feet long. In this section, a total of 55 risers 124.5 feet apart deliver the effluent from the tunnel to the diffuser caps on the sea floor. The diffuser tunnel has a decreasing internal cross section to ensure an even distribution of the effluent among the risers.

The risers were installed prior to the construction of the tunnel. They have been designed to be strong enough to withstand the external hydrostatic pressures in the longterm and in the dewatered condition as well as to withstand the grout pressures during the initial installation.

The construction material selected for the risers was fiberglass reinforced plastic (FRP). It was chosen for its ease of handling, durability, non-corrosive properties and its ability to provide hydraulically smooth internals. Riser segments of 40-foot lengths were chosen for ease of handling onshore and offshore, during road transport and on the installation vessels. The 2.5-foot internal diameter risers average about 240 feet in length.

The risers connect to the diffuser tunnel via a riser offtake tunnel. The design of these offtake tunnels involved considerable construction planning and careful construction specifications. The sequence for connecting the riser to the tunnel is depicted in Figure 7. The first step is to probe with a small diameter drill to locate the riser filled with dyed seawater to assist in location. Offtake adits are then excavated laterally from the tunnel to the installed risers, which are then exposed and connected to the diffuser tunnel. The void is filled with concrete.

Non-explosive excavation is preferred for the offtake adits in order to avoid damage to the risers and to decrease the potential for leakage. If blasting is proposed for part of this excavation, the contractor will be required to demonstrate that it can be done without damaging the riser.

One of the most critical offshore operations was the positioning survey for the seabed diffusers and corresponding riser shafts. The bot-

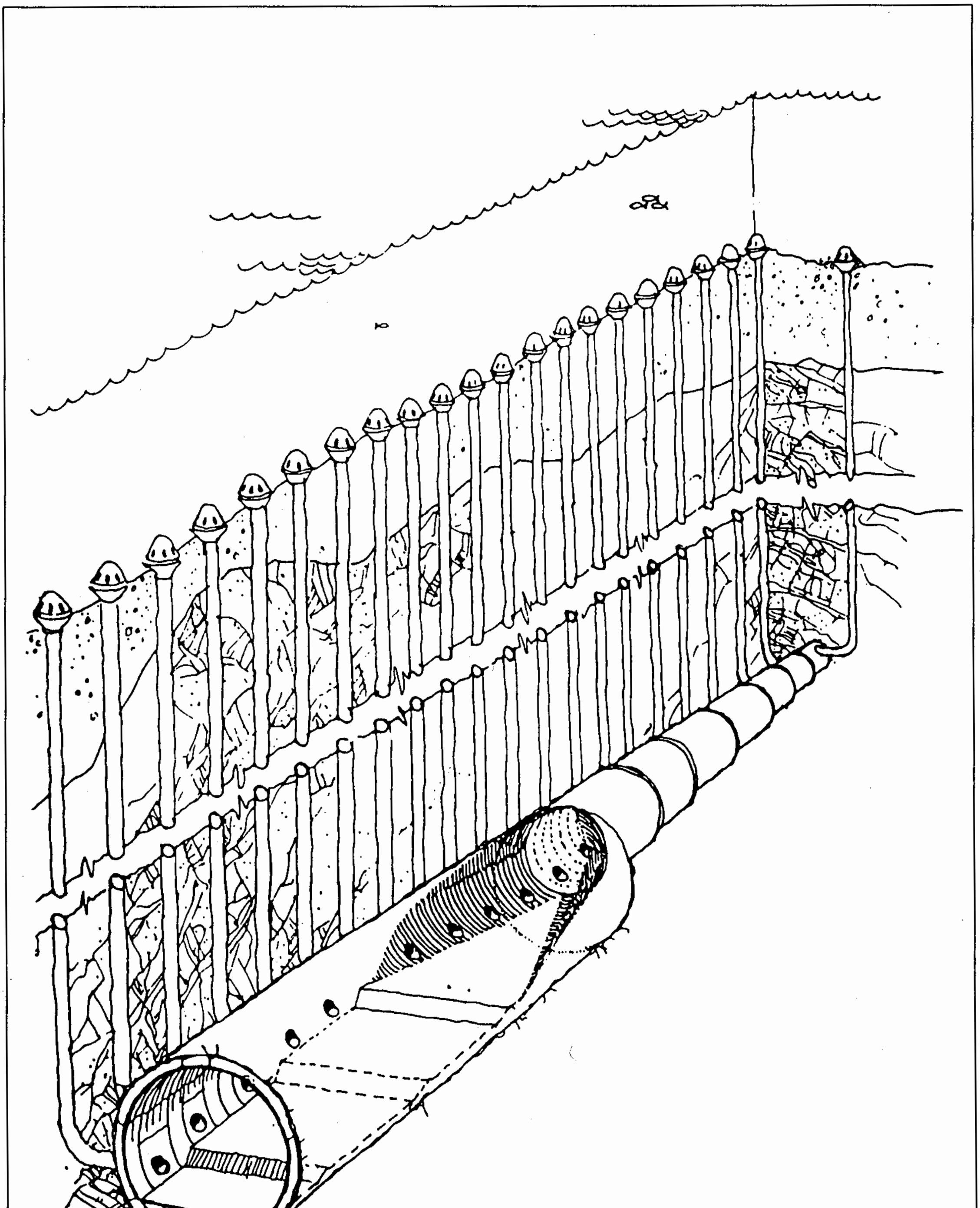

FIGURE 5. Diffuser tunnel, risers and discharge caps.

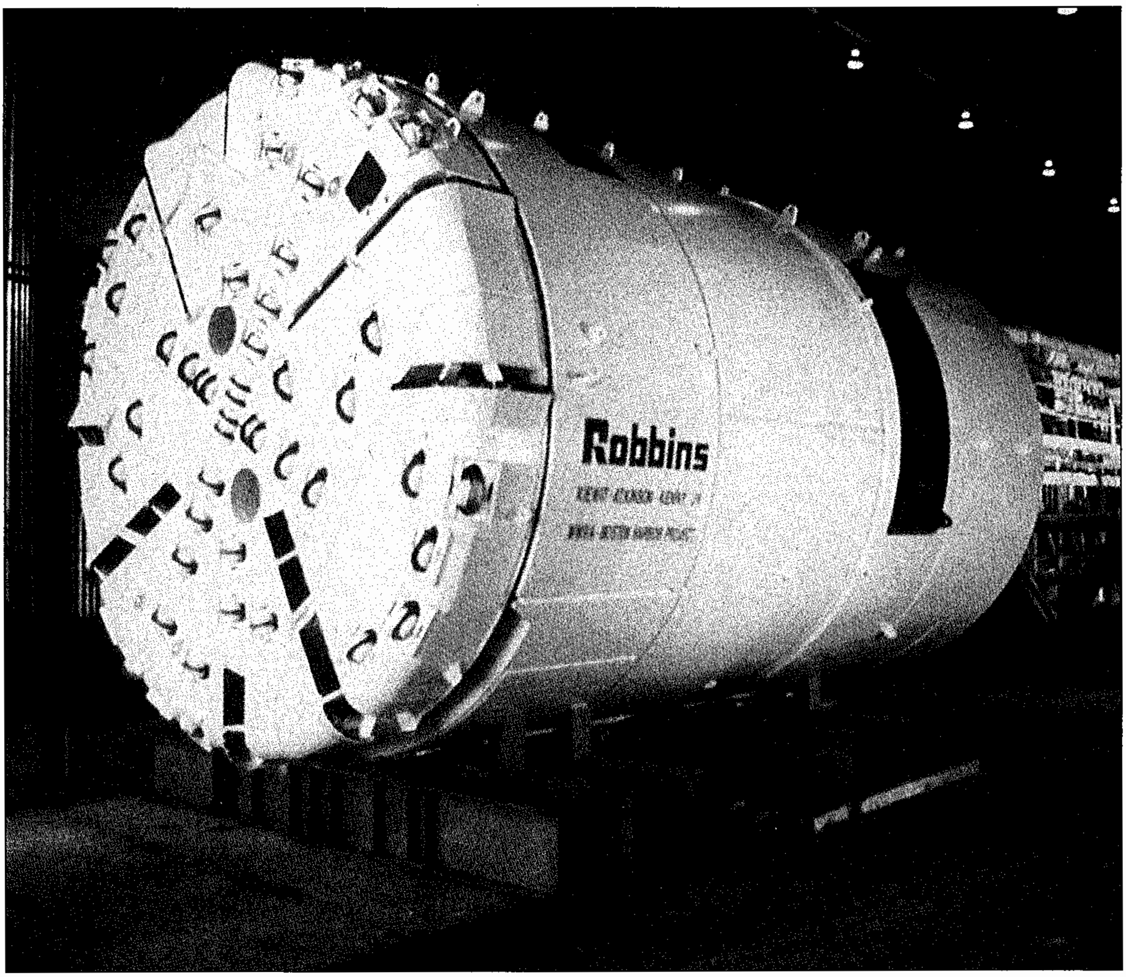

FIGURE 6. A tunnel boring machine comparable to the one used in the project.

tom hole coordinates of all risers had to be determined within a tolerance of 34 inches to minimize the amount of probe drilling. The offshore surveying was carried out in two stages. The first stage was the high accuracy survey of the jackup rig position and drilling templates to the baseline coordinates using differential global positioning systems and conventional geodetic survey methods. The second stage comprised separate downhole surveys that provided the relative position of the bottom hole coordinates using a two-axis wireline deployed gyroscope.

Diffusers. Diffuser heads are attached to the risers. The diffuser head has been designed as three components: base, cap and protective dome. The reinforced concrete base acts as a foundation for the diffuser cap and as a transition structure from the riser. The base must resist and transfer accidental vertical and lateral impact loads from the dome to the foundations. The cap containing eight evenly spaced ports comprises a molded FRP manifold encased in reinforced concrete. Each port has a cast nylon nozzle attached.

A primary concern during design was the potential for vertical and horizontal impact loads from ship anchors and chains. To accommodate these concerns, a protective dome shell was designed (see Figure 8). The caps are also surrounded by rock riprap to provide protection against scour and anchor drag. To provide the needed flexibility, durability and energy-absorption characteristics, cross-linked high density polyethylene was selected as a construction material.

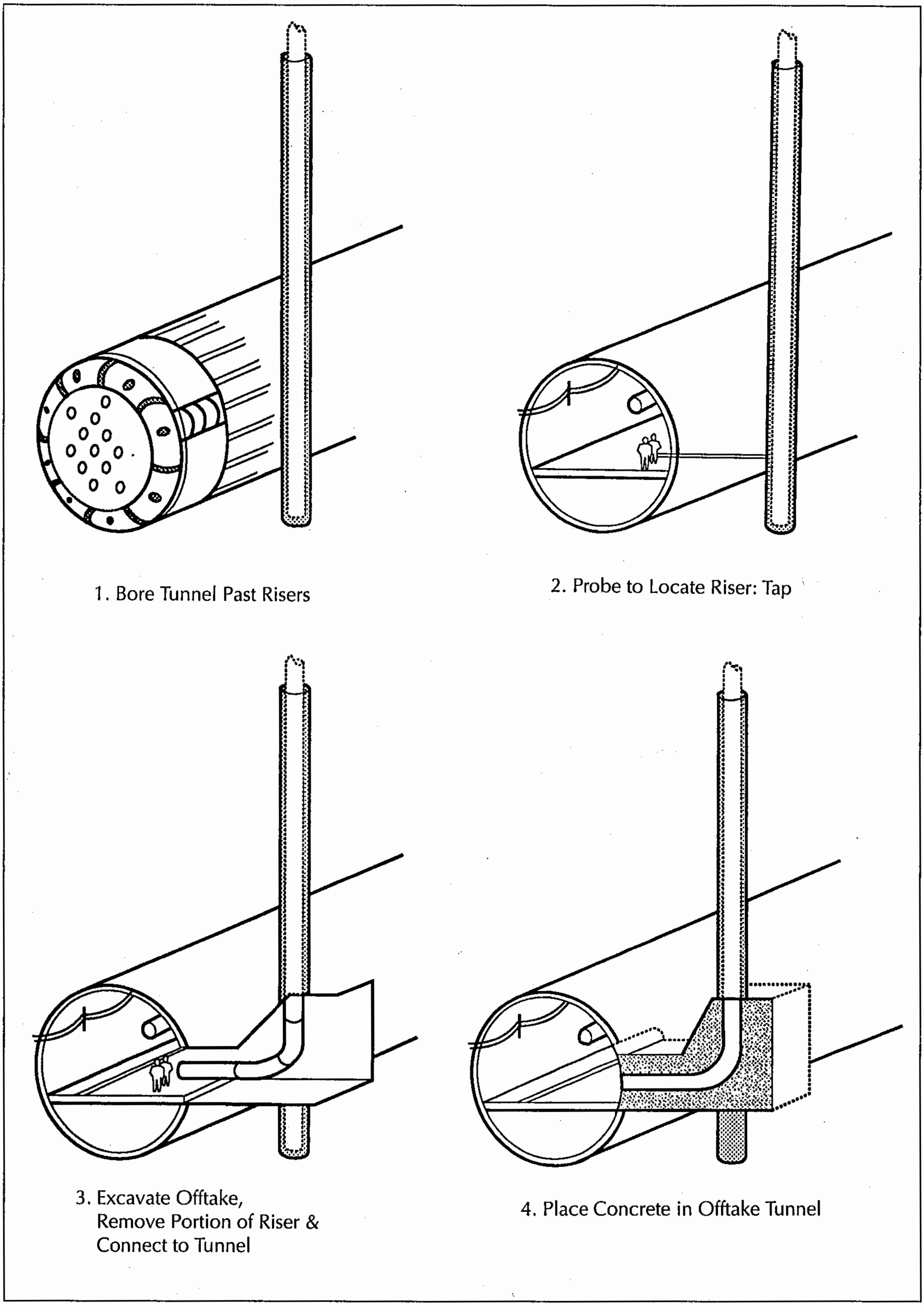

FIGURE 7. The construction sequence for connecting the riser pipes to the tunnel.

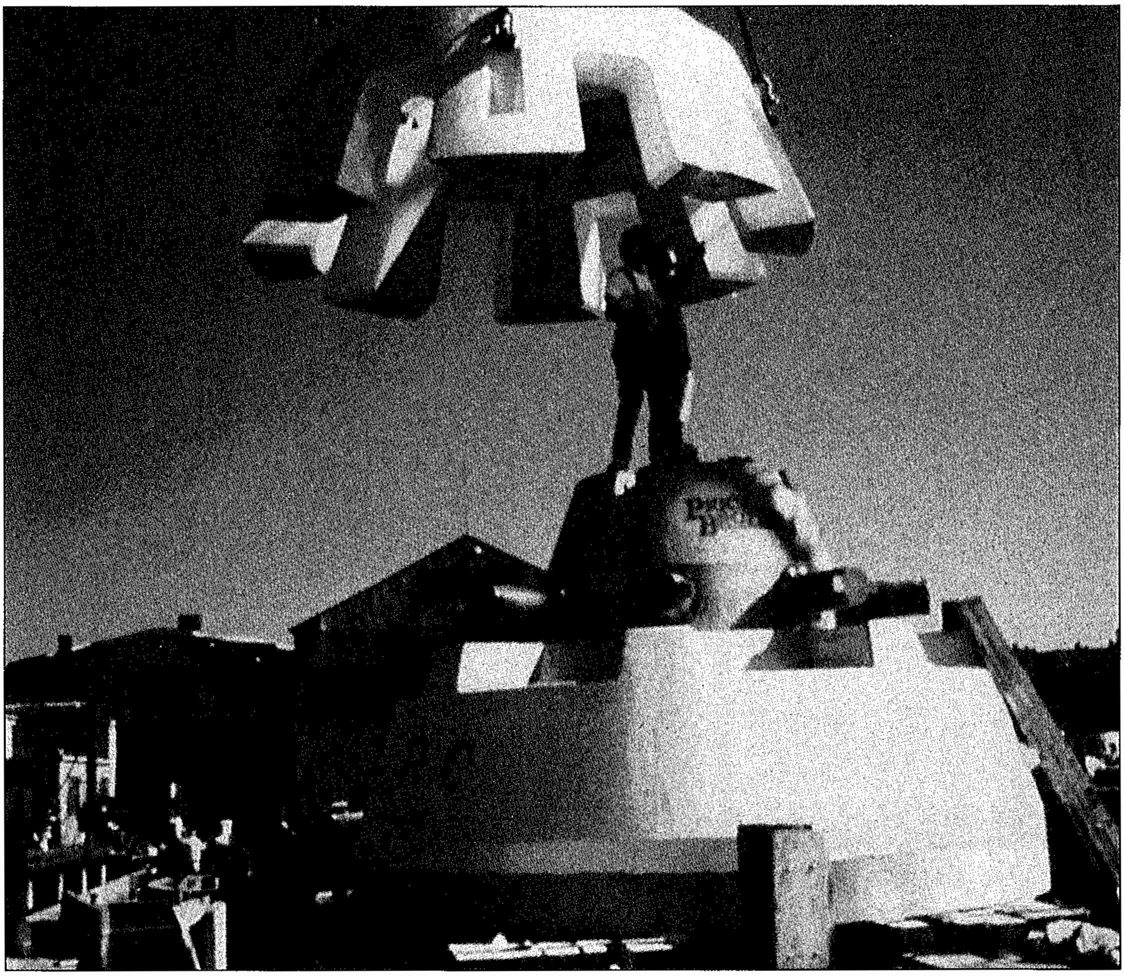

FIGURE 8. The riser cap and protective dome.

Diffuser caps, manifolds or nozzles are designed to be removed and replaced in the event of severe damage requiring repair or replacement. The cap is attached to the base with four bolts. In addition, a manhole cover is designed into the cap to facilitate inspection.

Startup Planning

There are two distinct operations involved in the outfall system startup. The first is the initial flooding of the tunnels and risers. The second involves the removal of the riser port caps, outflow initiation and saltwater purging of the system. The first operation is included in the tunnel construction contract. The second operation requires sufficient effluent flow from the treatment plant and close coordination among the MWRA, the Project Design Engineer, the Lead Design Engineer and the Program/Construction Manager.

Initial Flooding. The outfall is currently planned to be flooded with either effluent from the treatment plant or chlorinated seawater. The use of chlorinated seawater would minimize the potential for marine growth should there be any delay between the filling of the outfall and its startup.

The rate of filling was very important in the design of the risers and port caps, where release of air pressure and prevention of water hammer were concerns. The high rate of air pressure increase as a result of rapid filling could lead to an increase in air temperature in the risers. The derivation of the filling rate was predicated on the maximum pressure and temperature buildup that could be tolerated in the risers. Air relief valves have been incorporated in the design of the port caps to aid in pressure control. The construction specifications require the temperature to remain below 160 degrees Fahrenheit, the maximum temperature that can be accepted by the materials.

Startup & Commissioning. The startup and commissioning of the outfall basically involves the diversion of effluent flows from the treatment plant to the outfall tunnel. To be performed successfully, the flows must be sufficiently large to ensure the purging of the seawater in the outfall. If the available flow is below the 1,000 mgd required for the seawater purging, the riser caps may be removed in two steps. In the first step, only part of the caps would be removed to build up sufficient back pressure in the tunnel to achieve seawater purging. Then, the remaining caps would be removed.

The MWRA and its consultants are currently developing the detailed procedures that will be followed during the actual startup and commissioning. The following issues will be addressed:

- Responsibilities of the various parties involved;
- Treatment plant procedures;
- Final seabed inspection requirements;
- Sequence of port cap removal;
- Seabed monitoring requirements; and,
- Necessary modifications to procedures and requirements as a result of any additional mathematical or physical modeling tests of the hydraulic purging that may be conducted in the future.

DOMINIQUE N. BROCARD *is a Vice President of Metcalf & Eddy, Inc., in Wakefield, Mass., specializing in water quality, hydraulics, groundwater and modeling. He participated in the siting of the new Boston outfall during the preparation of the EIS for the USEPA and, working under the lead design engineer contract for the MWRA, he supervised the concept design of the outfall hydraulics. He holds a Ph.D. and an M.S. in Civil Engineering from MIT and is an ingénieur of the Ecole Nationale Supérieure d'Arts et Métiers in Paris, France.*

BRIAN J. VAN WHEELE *has been with Parsons Brinckerhoff for 25 years, beginning in 1969 and heading the Water Resources Group in New York before moving to California in 1976. After 12 years as Technical Director for Water Resources, he was appointed President of Parsons Brinckerhoff Gore & Storrie, Inc., Parsons Brinckerhoff's jointly owned water and wastewater engineering affiliate. He has been a project manager on such major water and wastewater tunnels as the Boston Outfall Tunnel Project, the Govalle Wastewater Interceptor Project, and as Deputy Project Manager on the San Francisco Southwest Ocean Outfall Project.*

LAWRENCE A. WILLIAMSON *is a Senior Project Manager with Sverdrup Civil, Inc., in Boston, Mass. He was formerly Senior Design Manager with the MWRA Program Management Division where he was responsible for management engineering services for the Boston outfall tunnel and diffusers as well as for the inter-island wastewater conveyance tunnel. He received a B.S. in Civil Engineering from Lowell Technological Institute in 1974 and an M.S. in Computer Information Systems from Boston Univ. in 1985. He has over 20 years of experience in the management of major engineering projects and is a Registered Professional Engineer in Massachusetts.*

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