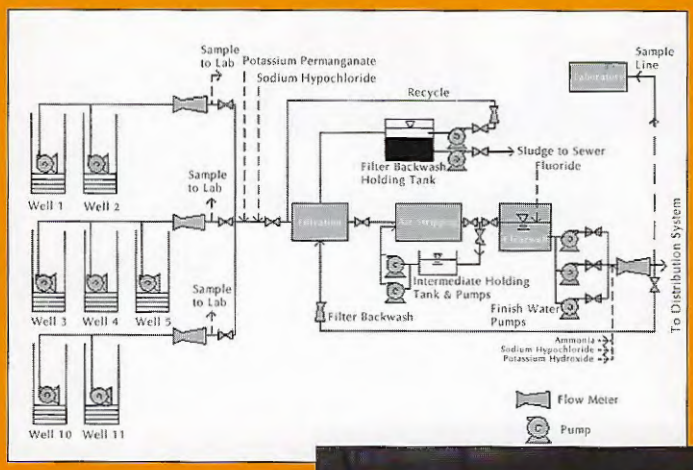
CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

FALL/WINTER 2000 VOLUME 15, NUMBER 2 ISSN: 0886–9685



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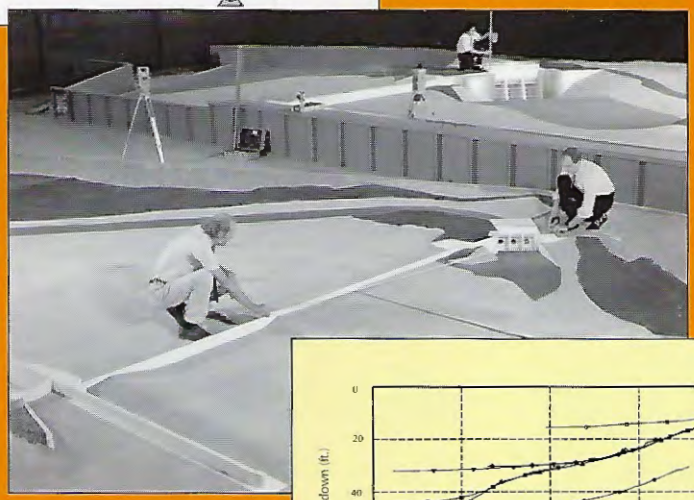

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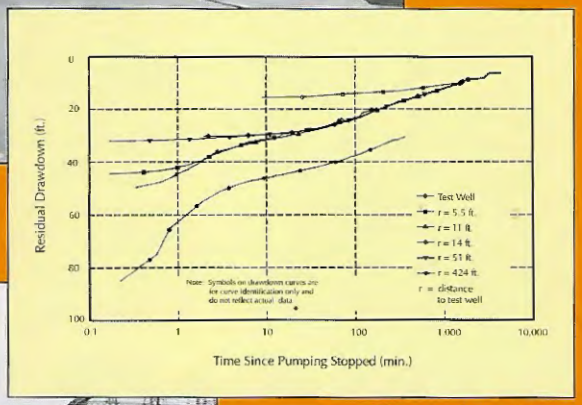

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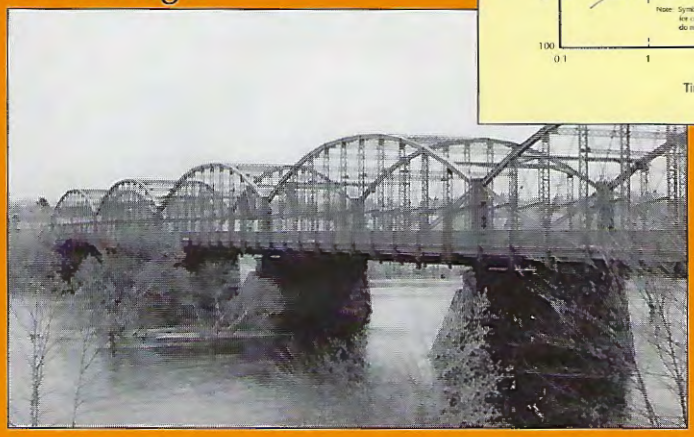

Lower Merrimack River Bridges

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CIVIL ENGINEERING PRACTICE: JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE (ISSN: 0886-9685) is published twice yearly in the Spring and Fall by the Boston Society of Civil Engineers Section/ASCE (founded in 1848). Editorial, circulation and advertising activities are located at: Boston Society of Civil Engineers Section/ASCE, The Engineering Center, One Walnut St., Boston, MA 02108; (617) 227-5551. Known as *The Journal of the Boston Society of Civil Engineers Section/ASCE* until 1985, Vol. 71, Nos. 1 & 2. Third-class non-profit bulk postage paid at Dexter, Michigan.

Subscription rates are: *U.S.* — Individual, $34.00/year; Library/Corporate, $40.00/year. *Foreign* — Individual, $40.00/year; Library/Corporate, $45.00/year.

Back issue rates for *Civil Engineering Practice* and *The Journal of the BSCE Section/ASCE* are available at $18.00 per copy, plus postage.

Please make all payments in U.S. dollars drawn on a U.S. bank.

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Printed on recycled paper.

Editorial, Circulation & Sales Office:

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The Leonard Zakim Bridge

Bridges are one of the most prominent and appreciated physical representations of what civil engineers do. A bridge is also a metaphor for connecting things, not just physically but spiritually. Indeed, it is a great honor to have a bridge named after you.

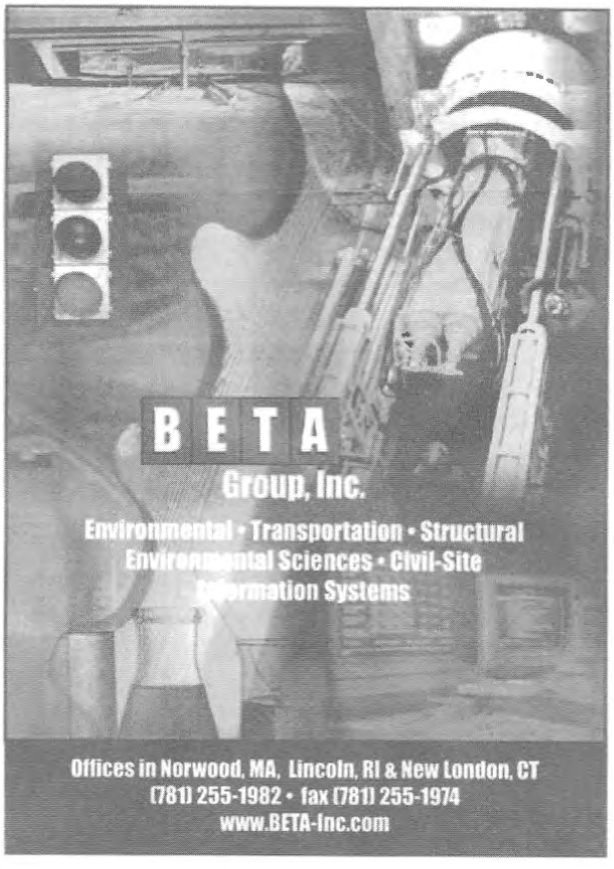
The late Leonard Zakim died of cancer two years ago. A symbol of his accomplishments will live on in the Central Artery cable-stayed bridge that will be named after him. For 15 years, Zakim was the director of Anti-Defamation League of New England, an organization dedicated to opposing hate crimes and bigotry. He was widely regarded as a bridge builder between Boston's often-fractious racial, ethnic and religious groups. He created the annual black-Jewish seder, an event that has attracted hundreds of attendees and juxtaposes the biblical story of escaping slavery with contemporary efforts to end oppression and prejudice. He started the Lenny Fund, which provides support to community and social service organizations. His efforts have helped to make Boston today a much different place than the provincial, strife-torn city of the 1950s and 1960s. While there are still many problems yet to address, there has also been much overall improvement.

The Leonard Zakim Bridge is one of the most visible features of the Central Artery/Tunnel (CA/T) Project, particularly since most of the project's massive structures and engineering achievements are buried underground. The bridge crosses the Charles River between North Station and Charlestown. It will eventually replace the existing truss bridge river crossing as the connecting I-93 Central Artery tunnels are opened for traffic in 2002 and 2003. The bridge was envisioned as more than just a transportation structure, but as a link and a symbol for Boston. The preliminary and final designs included efforts not just to design a structure, but to design a bridge that would act as a bridge in its symbolic sense. The CA/T Project has been envisioned as a project that would physically knit a divided Boston back together again. It is appropriate that the structure is to be named after the late Mr. Zakim, an individual who spent his waning energy battling a fatal disease, who wanted to accomplish the same goal.

Leonard Zakim was a personal hero to me. He was a great example of a man who learned to make lemonade out of the world's lemons. He stood up strong and energetically for those who could not stand well for themselves, and he devoted much of his time and effort in attempts to bridge divides between people. Since we all have to live on this planet next to each other, Zakim's efforts provided a guideline of a way to do it besides throwing Molotov cocktails and rocks. In the strife and warfare that has greeted the start of the new millennium, Zakim's passing is a great loss for all of us. He was the personal embodiment of the concept of a bridge, which is perhaps the civil engineering feature best appreciated by the public at large. As the sleek symbol and icon of Boston of the twenty-first century, the Leonard Zakim Bridge will stand for more than a transportation highway structure; it will be a reminder of the ideal of bridging the divide between people.

Brian Brenner

Brian Brenner,
Chair, Civil Engineering Practice Editorial Board



The Role & Contributions of Hydraulic Testing Labs: Part IV, Modern Power Plant Studies

Hydraulic testing laboratories have played key roles in advancing the science, practice and teaching of fluid mechanics. One on-going laboratory has made far-reaching contributions in the field.

George E. Hecker, Albert G. Ferron & Bruce J. Pennino

With the increased size of the power plants came problems related to the intake and discharge of condenser cooling water systems. Concerns with intake and discharge structures relate to their hydraulic behavior. In terms of power plant operation, these potential problems can be categorized into three areas: outside the intake structure, inside the pump sump, and outside the discharge structure.

Flows approaching pump intake structures are sometimes the source of pump problems in intake structures. The design of a pump sump can be invalidated because of the flow patterns outside the structure. Flows going by the structure (such as river flows going by perpendicular or at an angle to the intake) can cause swirling flow in front of the intake screens that can reduce the effective area of the intake. Channels with similar approaches can also have the same effect.

Sometimes the problem lies just inside the structure, in the screen area. Contractions and/or expansions in this area may cause the start of eddies in the pump well, resulting in rough and inefficient operation of the pumps. Alden Research Laboratory (Alden) conducted one such study in 1972 for the Bridgeport Harbor Electric Station in Connecticut. The condenser cooling circulating pump had severe vi-

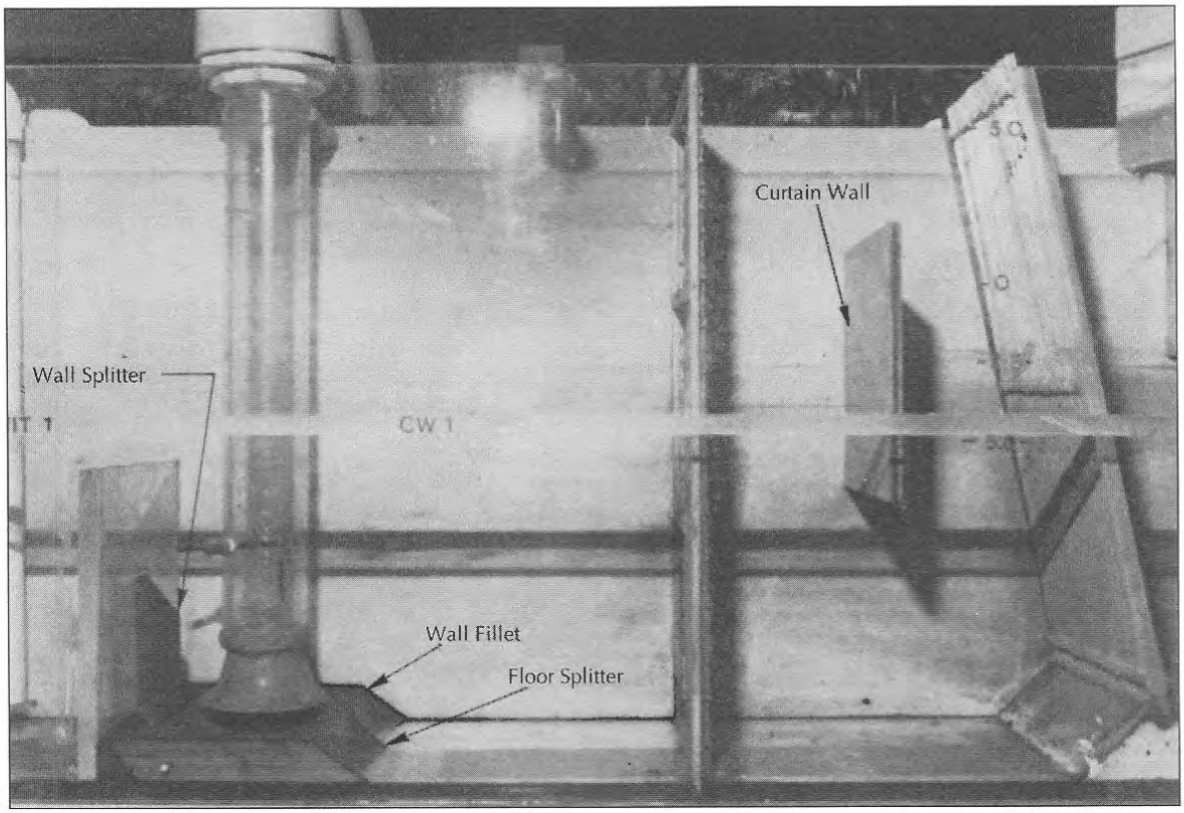


FIGURE 1. Pump intake sump with modifications.

bration problems accompanied by large suction noises. The sump was 15 feet wide. However, rotating trash screens had been installed, some of them 29 feet ahead of the pump. These screens required the flow to turn 90 degrees to go through the screen and then turn another 90 degrees after passing through the screen. As the result of this design, the flow width through the trash screen mounting structure was reduced to 3.5 feet, with velocities of around 10 feet per second approaching the pump at the center of the sump. Large, unstable vortices formed on both sides of the sump, entraining air at random intervals into the pump. The remedy was a baffle structure containing a screen with a 32 percent opening located 6 feet downstream of the screen structure. This solution was easily installed in the field, and pump vibrations ceased.

Prior to the construction of mega power plants, pumps and pump sumps were relatively small, and a number of so-called standard sump designs worked satisfactorily for most applications. However, problems began to appear with the "standard" sump designs when the pumps were made bigger, vertical pump columns increased in length and sumps were built larger. The problems were usually in the nature of large intermittent air-entraining vortices that caused poor pump performance or pump column vibration, or both. With multiple pumps in the same sump, it was not rare to also see horizontal vortices from one bellmouth to the other. In some cases, the vibrations were so severe that the pump column experienced structural failure.

In general, the sumps tended to be too large. Besides reducing the size of the sump, many other changes were sometimes necessary to solve the problems. In the case of the Indian Point (New York) Nuclear Power Station, Unit 3, cooling water intake (tested in 1985), changes to the original design included splitters under the bellmouth, corner fillets around the three sides of the pump, a curtain wall and a change in the floor configuration. These modifications have been designed into many and backfitted into others (see Figure 1). It should be pointed

out that sump changes such as these are not easily made to existing structures.

Alden found a niche in pump intake studies. A few organizations and college laboratories performed this type of study, but many other college hydraulic laboratories found the projects to be too routine or unrelated to "research." To Alden's benefit, these pump intake studies often led to the laboratory conducting other projects for utility companies. The relationships that developed continued for years and decades. Most importantly, the basic knowledge Alden gained from these studies allowed the laboratory to build an expertise for very large future projects.

Model studies were also performed on many service water pump sumps. Their problems and solutions were identical to condenser cooling water pump sumps. Therefore, model testing during the conceptual stages for any pump sump configuration was not only beneficial, engineering-wise, but also economical in the long run.

Open channel discharges from condenser cooling water systems can also be sources of problems. Recirculation, erosion and dilution are all factors that might have an effect on a project. Recirculation, in some cases, has been minimized by discharging along the shoreline of a river so that the water clung to the bank (called the Coanda effect). This setup increased the distance from the intake before the flow separated from the shoreline.

Not all cooling water systems utilized natural once-through cooling water from lakes or rivers. In some large plants, mechanical or natural draft cooling towers were used, and models of these pump intake structures were tested. In cooling tower models, the pump sump was modeled with a sufficient portion of the tower's basin. Water was distributed in the basin using a perforated plate simulating the cooling water flow at the bottom of the cooling tower. Data acquisition was similar to other pump sump studies and utilized a visual classification of vortices as well as a swirl meter (see Figure 2) to record data. One study performed by Alden had horizontal pump intakes that differed from most of the other intake studies, which tended to have vertical pumps. The solution to this problem involved the use of a vortex suppressor located above the intake. Doing many studies like this permitted Alden to develop a stockpile of many standard pump bells and model components, leading to project efficiency.

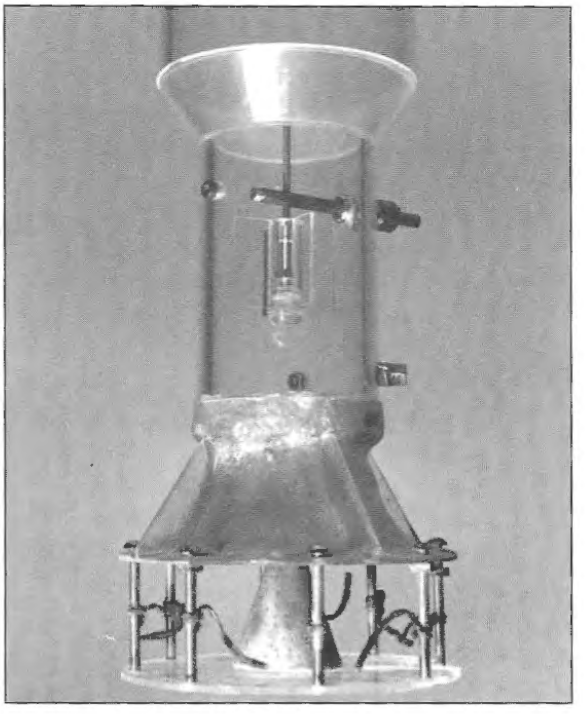


FIGURE 2. A swirl meter in a pump column, with yarn streamers used for flow visualization.

Emergency Core Cooling System Pump Sumps

Pump intake studies were not always related to circulating water-condenser systems. Some pump intake studies were on critical safety components in nuclear plants. In the mid-1970s, the Nuclear Regulatory Commission (NRC) became concerned about the reliability of the emergency core cooling system (ECCS) sumps located in the reactor containment building. In the event of a loss-of-coolant accident (LOCA), this ECCS and the containment spray systems (CSS) would be activated to supply coolant to the reactor core and vessel to dissipate the decay heat (to prevent core melt) and to the CSS to reduce containment pressure (to minimize atmospheric releases). Initially, these systems drew water from a large supply tank. Later, the water that accumulates

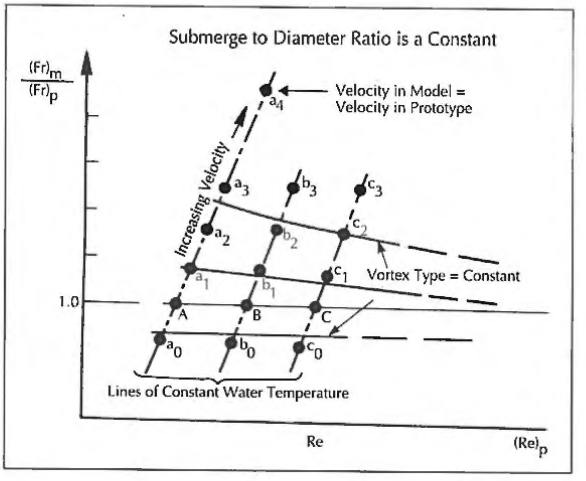


FIGURE 3. Vortex projection graph.

in a containment sump was recirculated (pumped) through the system. In effect, the ECCS sump was similar to any other pump intake, except poor performance due to excessive vortex activity could have very severe affects on reactor cool-down. There were basically two flow-related issues for the NRC:

- The ECCS sump had to be designed so that the intake was basically vortex-free with no air entrainment. (With air in the pumped flow, the ability of the flow to remove heat would be reduced.)
- There had to be sufficient submergence on the inlet to the pump so that the operational performance of the pump, particularly flow, was not impaired due to low net positive suction head (NPSH). (Sufficient head was achieved by using various combinations of blocking the approach screens to the sump.)

The crux of the matter was that these problems, with any of the various combinations of screen blockages, could not be addressed analytically. Furthermore, because many nuclear plants in the United States were site-specific designs with little standardization, the performance of one ECCS sump could not be confidently extrapolated to anther plant. Much later, the issue of debris injection would become a concern.

Trying to prove that the actual existing sump was functional was analogous to checking to see if a specific water sprinkler in a fire sprinkler system would operate during a fire. In 1977, a client visiting the laboratory on another study posed this question to George E. Hecker (who had succeeded Lawrence Neale as Alden's director on November 17, 1975). Hecker indicated that he would look into the possibility of model testing.

After a study of the existing literature, a proposal was made to use a new vortex projection technique. This vortex projection technique with a large model scale was required at the time because:

- a high margin of safety is required in nuclear-related work, compared to more routine vortex testing at pump intakes; and
- the scaling of vortex phenomena was affected by the Reynolds number, which basically considers the inertia of the flow compared to any damping due to the water viscosity.

Viscosity is affected by water temperature while the model velocity is affected by the linear scale selection. These are called "scale effects," and the NRC was concerned because physical modeling of these systems was unprecedented. For the vortex projection technique, tests were carried out at four different water temperatures with a number of different flow rates at each temperature. These flow rates simulated Froude numbers above and below that of the prototype. (The Froude number is the ratio of the inertia force to the gravity force.) A plot of Froude number versus Reynolds number was then constructed, and the vortex intensity number indicated at each test point. Loci of vortex intensity number were plotted on the curve (see Figure 3). The prototype operational Reynolds and Froude numbers were then put on the graph, and the loci lines extrapolated to obtain the vortex intensity expected in the full size prototype.

This technique was first utilized on a model of the ECCS sump for the Three Mile Island Nuclear Plant ECCS sump. The model was geometrically identical to the prototype, including all significant piping and any structural members or flow-interfering structures. Tests were conducted to study the system in the recirculat-

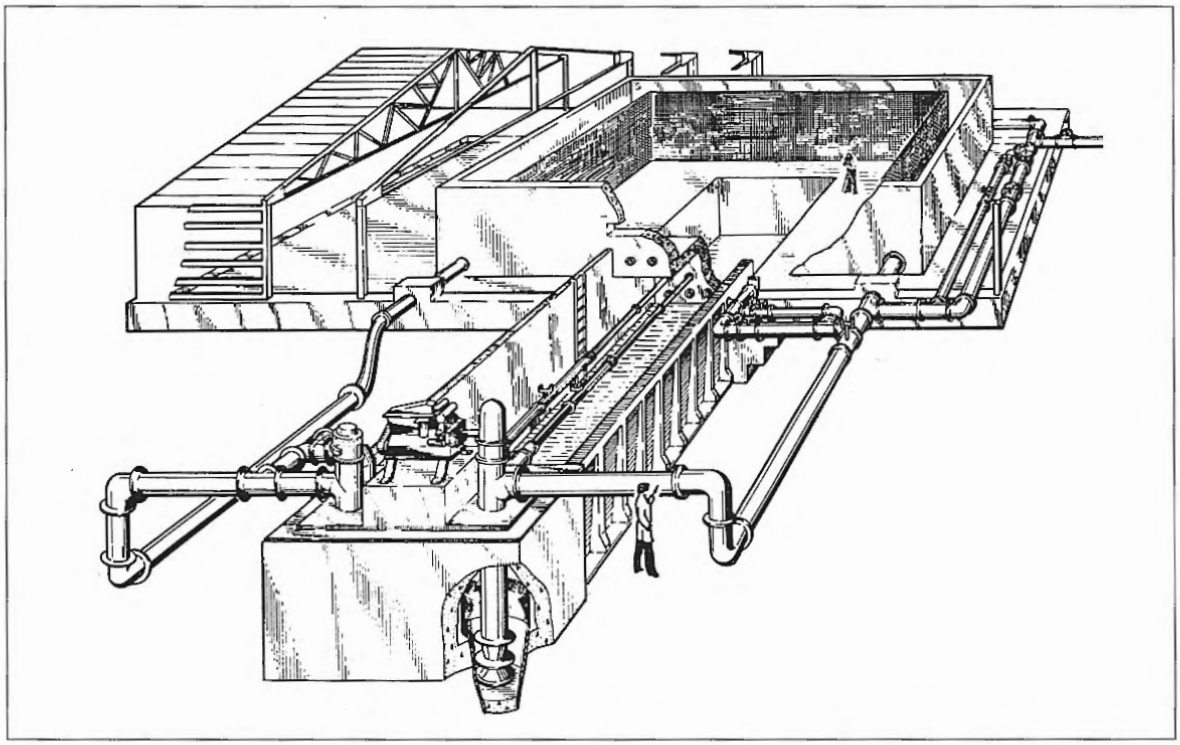


FIGURE 4. Facility for containment sump reliability studies.

ing mode. Vortices were evaluated visually, and vortimeters were used to measure prerotation in the intake piping. As in many of these studies, one or more horizontal layers of bar gratings suppressed vortex activity. The report was presented to the NRC and was accepted as a method to predict that the sump would perform as designed.

Once the word "got out" that the NRC had accepted the report, the flood gates opened and many other studies were commissioned. Because of the similitude concerns, the model scale on all tests was between 1:2 to 1:4 (one study had a full-scale model). None of the scale models duplicated the full 360 degrees of the containment vessel. Depending on the complexity of the interior of the sump, only 90 to 180 degrees of the vessel was modeled.

Full-Scale ECCS Testing

Although the models seemed able to predict emergency sump performance, there was still some question about the scale effects related to the Reynolds and Weber numbers (the latter is the ratio of inertia force to surface tension force). Laboratory staff also wanted to develop general design and review criteria that could be appplied to all plants. To resolve these NRC issues, Sandia National Laboratories contracted Alden in 1980 to build a million-dollar test facility to evaluate the vortex problem using different sump parameters in a full-scale setting. Testing continued until 1983.

The containment sump reliability (CSR) facility (see Figure 4) was designed to permit easily varying geometric and flow parameters with simple alterations of floors, walls and pipe fittings. The facility consisted of a 70- by 35- by 12.5-foot high concrete main tank and a concrete sump tank measuring 20 by 15 by 10 feet high situated within the main tank. Inflow into the main sump was through three sides of the main tank. Non-uniform flow was simulated by blocking any or all portions of the three-sided inflows. Four horizontal rows of outlet holes were located on the front wall with each row having five 25-inch diameter holes on 4-foot centers. Suction pipes ranging from 8 to 24 inches in diameter could be accommodated in each opening.

The suction pipes extended from the sump tank to a suction chamber 50 feet away.

Equipped for testing, the pipes contained a flow meter, a swirl meter, an air void fraction meter and a series of ten pressure taps located at 1-foot intervals along the pipe to measure the pressure gradient. Maximum flow for the facility was 20,000 gallons per minute (gpm). The facility was also equipped to simulate pipe breaks to investigate water jet impingement in the sump. Maximum impingement flow was 12,000 gpm. Data acquisition was by means of a mini-computer that received visual information on the vortex intensity and location information from a hand-held terminal and electrical signals from the instrumentation in the suction piping.

The main areas of concern in the studies were:

- *Entrained Air.* This air could be from surface vortices, air entrainment from pipe break jets or dissolved air coming out of solution as the result of fast swirling flow in the intake piping. (This was possibly the first time in the world that air content had been measured in vortex type studies.)
- *Prerotation.* Prerotation due to screen blockage or non-uniform approach flow patterns could have the effect of increasing the pressure loss in the inlet piping. Prerotation also can be a cause of reduced pump performance.
- *Excessive Losses in the Sump & Pump Inlet Piping.* This can lead to insufficient NPSH, thereby causing the pump to cavitate and reduce flow.

The main test program involved 66 configurations of pressurized water reactor (PWR) sumps and four configurations of boiling water reactor (BWR) sumps. Some of the configurations had horizontal pump intake pipes while others had vertical intake pipes. Tests on each configuration were made using a range of intake flows and water depths. In total, some 600 individual tests were performed.

This study also investigated different types of physical vortex suppressers. The use of horizontal gratings above the pump intake piping and grating cages with relatively wide grates were found to prevent air entrainment due to vortices. For existing sumps of all sorts, this approach could be suggested as an economical means of solving a vortex problem.

The basic results of all the scaled and unscaled testing showed that in vortex modeling, the viscous effects were negligible if the Reynolds number was greater than 70,000. This conclusion supported other findings that viscous scale effects on vortices were small when the Reynolds number exceeded 30,000. Also, no vortex problems were likely to be encountered when the inlet Froude number (based on submergence) was less than 0.23. (It is interesting to note that a similar Froude number value had been determined in 1979 based on numerous final designs for pump storage intakes that had been modeled at Alden.)

Screen Blockage Testing

Because of the potential for screen blockage due to insulation that could be blown by pipe break into the flow approaching the sumps, Alden received inquiries to study the hydraulic behavior of many types of pipe insulation used in nuclear plants. The number of such studies was increased by the intense competition between the various insulation manufacturers and the large head losses that could potentially occur at a screen.

The studies were essentially of three types:

- One question related to the pressure loss through the insulation as it accumulated on the screens. These studies were essentially performed in a closed pipe loop where shredded insulation was introduced into the system and was caught on a screen. The pressure drop across the screen was continually measured to monitor the pressure increase with time as the insulation continued to pack on the screen. The effects of temperature and water alkalinity were also investigated in this facility.
- A second type of study was conducted in a facility originally constructed to study fish behavior. In this full-scale facility, the movement of shredded insulation of different sizes was observed. Velocities required to move the insulation were also recorded.
- The third type of study involved finding the minimum velocity needed to break down the various insulation packages. In

this study, a water jet from a nozzle was aimed at the insulation, and the jet velocity was increased until the insulation showed signs of initial failure. The velocities were further increased until particles of insulation were blown away. This information of the breakdown of insulation packages was required to predict how particular insulation would stand up under jets formed by accidental pipe breaks.

Condenser Testing

Alden was involved not only in studies related to the intake and discharge of condenser cooling water systems but also in a number of different types of condenser studies. In the early 1960s, Alden modeled a horizontal split condenser. The condenser had rectangular approach piping with two right angle elbows in different planes just ahead of the condenser inlet. Observations in the field indicated that the condenser tube bellmouths in approximately the center of the two tube sheets were completely eroded. In addition, the tube sheets in those areas were highly polished.

The condenser was modeled in plexiglas to observe the flow patterns inside the condenser. Punched paper dots from computer cards were used to visualize the flow. The tube pattern was modeled with one tube in the model representing ten tubes in the prototype. The length of the tube was not modeled, but the pressure drop through the tube was modeled by inserting orifices in each model tube. To duplicate the reduced pressure in the prototype, the model was placed at the top of a 20-foot tower and the flow was supplied from a low pressure pump located at the ground level.

Initial tests immediately revealed a high velocity vortex at the center of the condenser extending from one tube sheet to the other in the same area where the damage occurred in the prototype. The vortex was so strong that it pulled air out of solution, and the model core contained only air. After numerous tests, a solution was obtained consisting of two perforated plates installed to split the condenser in the form of a tent. The holes in the 1-inch thick plate were 2 inches in diameter.

The next condenser study was another model of a horizontal split condenser (see Figure 5). The model construction was similar to the first study, but no attempt was made to model the pressure in the condenser. Visual and photographic documentation was used as before. However, three-dimensional velocity probes had become available in the decade between the two model tests. Such a probe was utilized to obtain the velocity distribution just in front of the tube sheet.

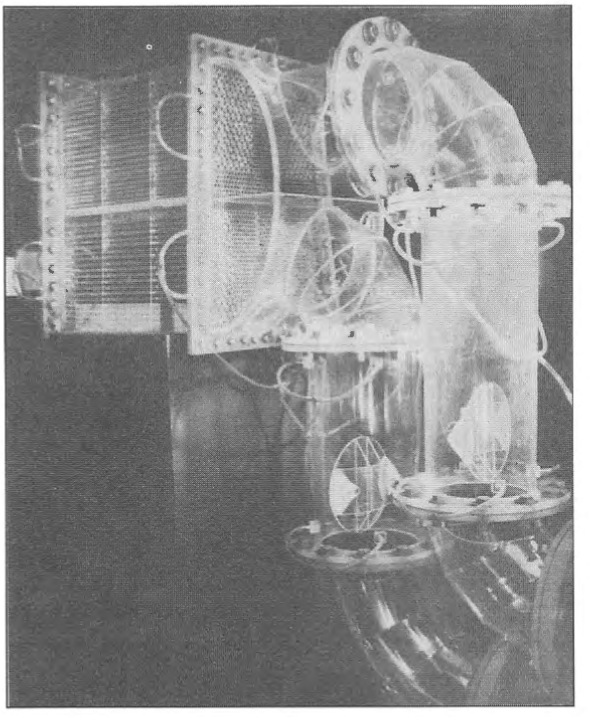


FIGURE 5. A condenser model.

A major breakthrough in velocity measuring instrumentation came in 1978 with the Hope Creek (New Jersey) model of a two-pass horizontal condenser. Tubesheet erosion and flow distribution were the concerns in this study. Flow visualization was accomplished as in the other studies. Velocity measurements, however, were obtained by use of a laser doppler anemometer (LDA) system mounted on a converted milling machine base. The LDA system works on a doppler shift principle using a helium-neon continuous wave laser. The laser beam passes through a beam splitter, creating two equal beams. The beams are then positioned to intersect at a desired focal length. At this intersection, called the measuring volume, the beams interfere constructively and destructively, producing fringe patterns in the measuring volume. Parti-

cles, no matter how small, cause the fringes to move, creating a doppler shift. The light reflected by the particles results in a shift in frequency that is a function of the velocity of the particle. A photomultiplier detects the movement of the fringe pattern and feeds that information to an automated data acquisition system.

Condensers, fouled by biological growth, have reduced heat transfer, thereby resulting in reduced power production. One method to counteract this buildup is to periodically inject the condenser with a dose of chlorine. This method kills the organisms, and they get flushed out with the condenser cooling water. In the late 1980s, with environmental concerns for the water bodies receiving these condenser cooling discharges, the U.S. Environmental Protection Agency passed a new regulation that limited the total discharge of residual chlorine to 0.2 mg/L for no more than two hours per day per plant. This regulation prompted utilities to investigate other means to prevent or remove fouling. Studies, based on a prior Salem (New Jersey) model, were conducted using fixed nozzles to selectively inject chlorine upstream of the tube sheet. This technique, called targeted chlorination, injects chlorine through a pipe with a nozzle at the end. The pipe is located far enough from the tube sheet so that the flow expansion from the discharge chlorine jet covers a portion of the tube sheet. The study was used to minimize the number of nozzles necessary to ensure that all the condenser tubes received enough chlorine to effectively remove the fouling.

Finally, evaluations of ball distribution in a sponge ball condenser tube cleaning system were conducted in the Salem model. The sponge balls were modeled with 0.125-inch-diameter colored fiber decoration balls. At the outlet to the condenser, a basket-type container (which was partitioned off in different sections) was used to catch the balls so that the percentage falling in each section of the basket could be obtained. To decrease the test time, different colored balls were used for each test, requiring the unbolting of the downstream end of the condenser only after a series of colors had been used.

Detailed studies of the inlet flow in the vicinity of the inlet of the condenser tube were made using a 3-inch inside diameter model tube to represent a 0.902-inch inside diameter prototype tube. The use of metal inserts was also studied in a similar model. In both cases, flow patterns and pressure losses were obtained.

When condenser tubes began to leak, either the inserts were installed, an epoxy coating was used temporarily, or the tube was plugged. The method of applying the coating and the behavior of the coating under vibration were studied at Alden. A wide assortment of plugs, made of different materials and varying in cost, were tested under different pressure conditions to determine their holding power. As in the past, one study would lead to another in an effort to improve the performance of power plant related equipment.

Although not related to power plants, a condenser and a hotwell on board a ship were modeled to investigate the behavior of the condensate under oscillating conditions. The model was mounted in a cradle at the end of a arm pivoted 17.5 feet away. The cradle itself was pivoted 12.48 feet from the arm. This permitted the maximum pitch to be ±15 degrees and the maximum roll to be ±50 degrees. The whole system was operated by piston cylinders controlled by an electronic control system. Data acquisition was by a video camera mounted on the cradle.

Video cameras became a great asset in model testing. Prior to videos, still photographs were taken to show customers some of the results of the tests. At regular time intervals, the client came to the laboratory and was shown a number of significant tests during the visit. These visits were always stressful times for the laboratory staff. The scenario usually went like this:

> The customer arrived early in the morning and was shown the model in one configuration. Afterwards, the director would take the client for coffee while the rest of the staff made a change to the model. The customer would return, see the model run under the new configuration and then to lunch with the director while the staff made another change. After lunch, there was a repeat of the morning's activities. By the end of the day, the customer had seen the model perform under four different configurations. In

some cases, the staff would even work that night to prepare the model for the next day. With a video camera, this extra activity was eliminated because all tests could be easily taped. In fact, a tape of a test today can be sent by special overnight courier and be viewed by many engineers the following morning in their own offices. The cost of a camera and tapes was much more economical time- and money-wise than a round trip to observe the tests in person.

Since Alden had worked almost since its inception with utilities, it was not unusual that it performed a number of studies on other fossil fuel plant equipment. Among some of these studies were:

- Hotwell outlet of a condensate polisher system;
- Water strainer comparison of pressure loss;
- Spray cooling nozzle design/efficiency;
- Air coolers;
- Steam separator drum in a condensate system;
- Boiler flow patterns investigation using water as the flow medium; and,
- Manifold pressure losses.

Stack & Three-Dimensional Air Modeling

Up to this point, all the above studies used water as the fluid. Other studies used air as the medium. Probably the first such study was a smoke stack study of the Edgar Station (Boston, Massachusetts) in 1954. There was concern about stack releases from the new Edgar Station because, surprisingly, objectionable ground-level concentrations had been detected 5 to 6 miles away. Another concern was that a south wind could direct smoke from burning high-sulfur oil toward Logan Airport. The purpose of the study was to evaluate the dispersion of smoke and test the effect of varying stack height, exit velocity, devices on the top of the stack and building streamlining. Smoke emerging from the chimneys and down-drafting locally to ground level was also to be investigated.

A 5- by 5- by 8-foot-long wind tunnel with velocity capability of 0 to 50 feet per second was built in Alden's low head laboratory for this testing. The 1:250 scale Edgar Station model duplicated every detail of the exterior features of the plant and was mounted on a 5-foot diameter rotating table to evaluate the smoke flow at every wind direction. The smoke was generated by vaporizing oil on electric heaters. Air was passed over the heaters, whose heat output could be varied, and distributed to a manifold. This manifold could distribute the smoke-filled air to the model smoke stacks and could bypass some of it to regulate the air flow to the stacks. Pitot tubes in each stack measured the air flow out of each stack. Photographs of each test were taken using strobe lights to stop the action. There was concern about the model's accuracy and applicable similitude requirements. It was concluded, based on observations of the operating prototype, that the model smoke was less buoyant than the prototype, and the model results were conservative.

By the 1960s, when very large steam-electric stations were being planned and air quality laws were coming into effect, the height of the stack became an even more important component of plant design that depended to a major extent on fuel type. Coal-burning plants particularly became concerned with local ground-level concentrations. These concentrations were reduced by stacks having heights of 800 to 1,000 feet. Mathematical modeling of stack releases became the norm. Various analytical models allowed consideration of various meteorological and atmospheric factors. To prevent downwash and local ground concentrations, some "rules of thumb" indicated that a chimney should not be less than 2.5 times the height of adjacent structures, subject to modification based on fuel type. Natural gas and low-sulfur oil plants had the least air pollution problems.

By the mid 1970s, it was realized that tall stacks, which basically put the pollutants in upper wind levels, were causing visibility and pollution problems hundreds of miles away. Restricted visibility at the Grand Canyon due to plumes from the Four Corners Power Plant, possible acidification of soils and lakes, deposition of heavy metals and other undesirable situations can result.

Three decades later, a stack-breeching section was investigated (see Figure 6). The tall stack did not have sufficient draft to discharge

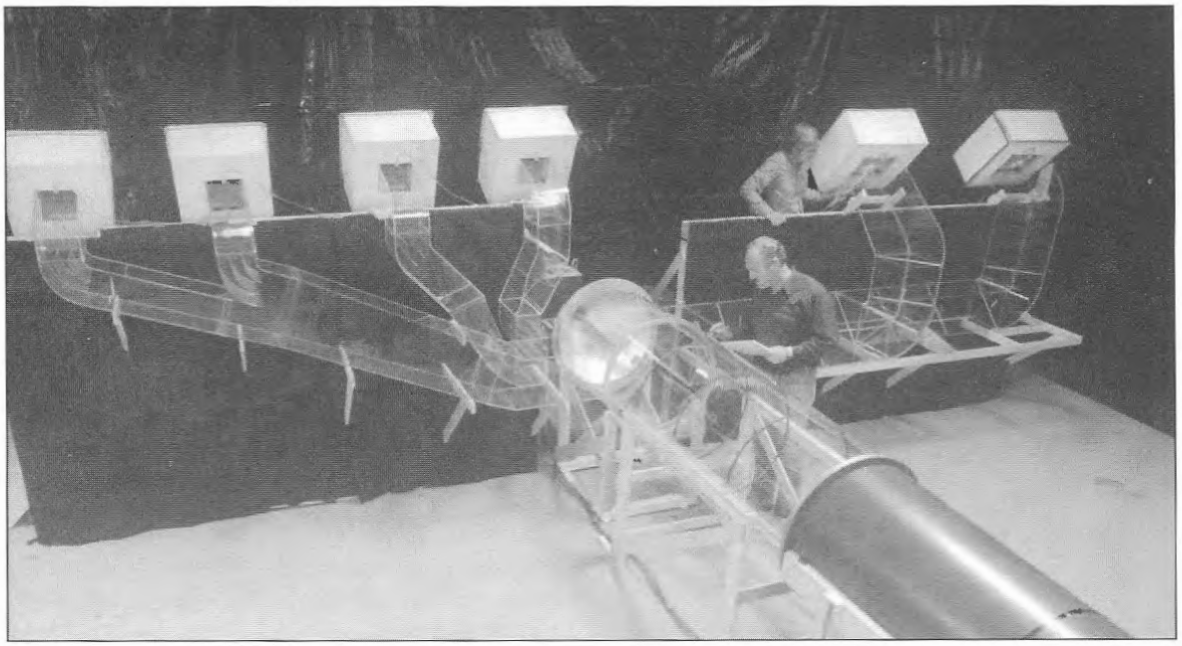


FIGURE 6. Model of ducts to a stack base.

the exhaust gases from two units. Inflow from each unit entered the stack at 180 degrees from each other. Flow into the plexiglas bottom of the model allowed the two gas streams to oppose each other and caused swirling flow to go up the stack. This swirling flow created higher pressure drop in the chimney, thereby resulting in decreased flow. The bottom of the stack was modified by putting in a splitter wall and some vaning. In addition, vaning was installed in the rectangular approach piping to further reduce the overall system pressure loss.

Three-dimensional air modeling would become a much larger activity in the 1990s when air regulations became much more stringent in order to comply with the 1990 Clean Air Act. Modeling of precipitators would also become routine. A typical air study on a wet electrostatic precipitator was performed with a model constructed entirely in plexiglas. It investigated the approach and the discharge flow patterns within the precipitator. The main objective was to better the efficiency of the precipitator by improving the flow patterns. Greater efficiency was achieved without increasing the overall length of the precipitator through vaning and using a series of perforated plates.

Smoke was considered to show the flow patterns in the stack-breeching model. However, the quantity of smoke necessary in a 3-foot-diameter model stack would have been excessive and would have caused problems in the immediate environment. Someone at Alden thought of the movies and talked with movie equipment distributors. They sold a system that produced "smoke" by means of neutral density helium bubbles. A system was acquired and used successfully to obtain photographs of the flow phenomena in the model.

The same flow visualization system was used to study a superheater manifold. The aim of the study was to equalize the flow out of each part of an existing manifold that was causing many field problems.

Warm Water: Thermal Model Studies

The first large hydrothermal models built by Alden were constructed in the early days of environmental concerns. Some were built to study recirculation while others were dictated by the loose environmental regulations of the time. In some cases, these regulations seemed to be more in the minds of the regulatory agency personnel than on paper. Many meetings were held with utility and regulatory agents to review model test results (usually surface isotherms) to see if the intent of the regulation was met. In many cases, it was a judg-

ment call based on what people thought the regulation meant.

In 1967, Alden studied the thermal discharge effects of the proposed Easton Nuclear Plant on the Hudson River. A Venturi-type discharge structure, with a throat velocity of 10 feet per second, was placed at the end of a discharge channel to maximize dilution and to prevent fish from entering the channel. This structure was called a hydraulic fish screen. By today's standards, it was a primitive surface jet discharge (thermal diffuser) where the intent was to rapidly entrain cooler river water to quickly reduce the temperature and extent of the discharge.

The Herbert A. Wagner Plant on the Patapsco River in Maryland had a similar high velocity discharge (13 feet per second at low tide) at the end of a short discharge channel. This study aimed to prevent recirculation and minimize the heated water from the Orchard Beach-Riviera Beach areas. Prior to the study, field tests were done at the site to obtain near-field velocity information. The velocities were obtained by using floats with metal drogues and photographing the floats at different time intervals from an airplane. After the floats had been deployed, Alden personnel went to the roof of the plant to observe the floats and to make notes regarding the tests. Suddenly, as if out of nowhere, a high-speed motor boat appeared, scooped up two of the floats and went on its way as people screamed from the roof. It was the second time in this type of study that floats had been damaged or stolen. Once before, on the Hudson River, floats had purposely been run down by a barge captain, who agilely guided the barge tow directly over the center of two floats as Alden staff watched helplessly.

Not all discharge structures were at the end of channels located near the shore. In 1969, Alden tested an offshore discharge structure for the D.C. Cook Plant on Lake Michigan that would minimize mixing. The idea of this scheme was to maximize the surface water temperature so that the thermal "driving force" to the atmosphere would be the greatest. This thermal transfer was accomplished by discharging through two 90-degree elbows at the bottom of the lake. The model was constructed in a small tank, and the elbow was made in two parts of fiberglas laid up on a wooden mold.

The D.C. Cook plant was caught in the changing regulatory climate. In the fall of 1970, the plant was informed that the temperature exposure for biota entering the heated plume had to be minimized. Consequently, a structure that produced maximum dilution was preferred over a discharge with a minimum dilution. Maximum dilution was best achieved with a jet-type discharge structure at the elbow locations (see Figure 7). A minimum size area enclosed by a 3°F isotherm and minimum shoreline temperature were required, which was accomplished using a 1:75 undistorted near-field/intermediate-field model. A multi-slotted discharge structure was developed, and the maximum surface and shoreline temperatures were greatly reduced.

The 1:75 D.C. Cook model built in 1971 was the first of the so-called "piggy-back" models. When many of the first large models were completed, clients requested that they be kept intact in case future studies were required. The customer would pay a small rental fee for that privilege. When contracts for other large models were signed, Alden faced the dilemma of where to build them. Since the models required enclosed buildings, and all the land near the main buildings was occupied, it was decided to build these new models above existing models in such a way that they could easily be taken out. (This scheme was never put to the test since removal of any of the top models never took place. However, in some cases, the lower model basin was used to supplement the sump in the building.)

The "piggy-back" models were those containing gently sloping topography, such as ocean and large lake sites. By the late 1960s, virtually all thermal models concerned with mixing were constructed having the same horizontal and vertical length scale (*i.e.*, an undistorted model). The platforms usually consisted of wooden 2- by 12-inch templates, set at the proper grade to represent the topography, sitting on metal pipe stilts. The D.C. Cook model used a type of cloth or screening stapled on top of the templates, which was then sprayed with chopped fiberglas and epoxy to give the model a hard surface. Unfortunately, the material sagged between the templates, causing a rippling topography. Consequently, much work was required to fill in the sags and properly re-

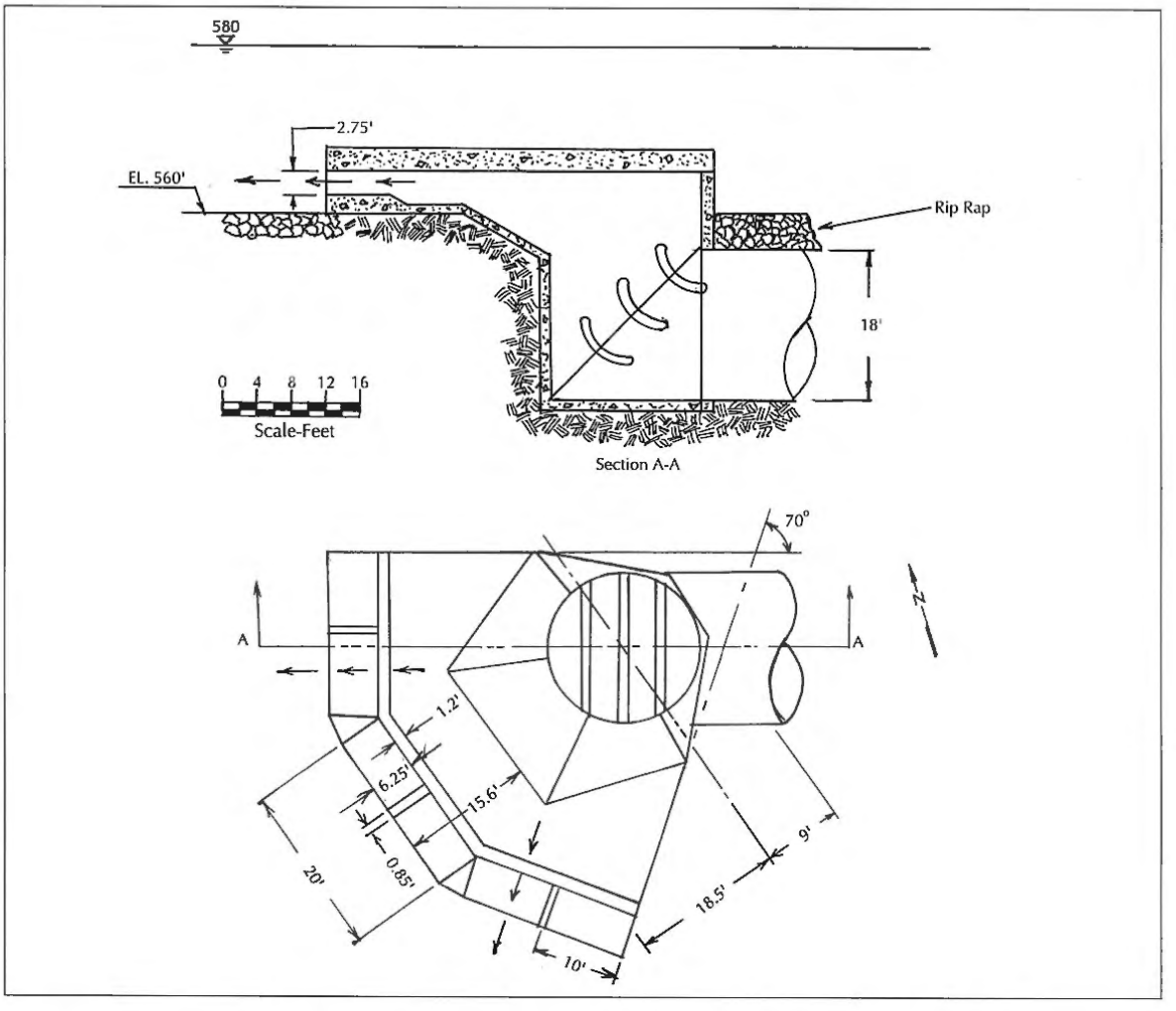


FIGURE 7. Slotted jet discharge structure for the D.C. Cook Plant hydrothermal model study.

produce site topography. A lesson was learned and, in the future, 0.25-inch-thick plywood sheets were put on top of the templates and then coated with the fiberglas. At least seven models were "piggy-backed" in the 1970s and 1980s using this technique.

Fiberglas thermal models were thermodynamically desirable compared to concrete-topped sand backfilled models. As the tests became more sophisticated, heat transfer to the model's surroundings became an issue. A fiberglas model absorbed and conducted away less heat.

Hydrothermal Regulations

The simple diffusers (as in the Wagner and D.C. Cook models) served their purposes, but things would soon change. In the late 1960s and early 1970s, many states began promulgating stricter regulations regarding thermal discharges. It was not always simple to write a regulation that would encompass all of the water bodies for a particular state. Few states had such a simple regulation. Some states had different regulations for fresh and ocean waters. In many cases, the regulations were somewhat site-specific based on the ecology of the area. For example, a river originating in the mountains and discharging into the ocean might have one regulation for the river source to the bottom of the mountains, another from that point to the upper reach of the tidal effects and a third regulation from that point to the ocean.

Tighter regulations put an end to most of the economical on-shore or near-shore condenser cooling water discharges. Instead, cooling wa-

ter system designers had to concentrate on offshore intakes and discharges, mixing zones and areas in the discharge water body beyond which certain temperature rises above ambient temperatures could not be exceeded. The industry's solutions were either off-shore, multi-port diffusers or cooling towers having no off-site thermal discharge.

The main background information for this work originated in the time period during which Alden started the major hydrothermal studies (roughly 1958 to 1964). Technical papers of that period mention mixing in tidal estuaries, turbulent jet theory, diffusion of sewage discharges, turbulent entrainment in stratified flow, discharge of warm water jet, turbulence in a diffuser boundary layer, convective currents in water and other related topics. By 1971, Dr. Donald R.F. Harleman and his colleagues at the Massachusetts Institute of Technology (MIT) were doing research and publishing technical papers specifically related to condenser cooling water discharges. The subject was of such importance to the utilities that one commissioned Drs. John E. Edinger and John C. Geyer, of Johns Hopkins University, to do a study of "Heat Exchange in the Environment."[1] The report was a textbook designed to assist engineers and scientists in computing temperatures related to the heating and cooling of natural water bodies.

By 1969, Alden had started a hydrothermal study of the Yorktown Station in Virginia that included a distorted overall model to obtain temperatures in the York River, and an undistorted model to study a multi-port diffuser for the existing and expanded plant. The overall model of the York River extended from the upper reach of the tidal effects at West Point, Virginia, to its mouth at the Chesapeake Bay. Major innovation was taking place.

Innovation in Thermal Modeling & Diffusers

It was during the mid- to late 1960s that physical thermal modeling techniques were extensively evaluated by Alden staff. A visiting mechanical engineering professor from Sweden, Peter Larsen, was a leader in this effort.

By the time the Yorktown model was built in 1971, Alden had added sophistication to these models, which in the early years were concerned with near-field mixing and far-field heat transfer. Temperature data acquisition systems using mini-computers replaced the old data recorders. Environmental controls were installed to keep the ambient air temperature in the model building constant. The idea of this system was to minimize heat transfer either into or out of the model. The goal was to control the atmospheric conditions at the equilibrium temperature — *i.e.*, the temperature at which no heat exchange occurred between the water and air. Sprinklers were mounted on the roof to reduce convective and radiant heat transfer to the model.

The intent of all hydrothermal models was to study only the plant heat discharge on a constant temperature body of water, exclusive of meteorological effects. This focus could be especially difficult to achieve in tidal conditions where numerous tidal cycles were required to establish "steady-state" thermal conditions.

The similitude of the model heat transfer with the prototype was a major concern that was extensively investigated during several studies. For one proposed project, model temperature isotherms were adjusted based on an existing plant's thermal isotherms and meteorological conditions in the field. In essence, the model was thermally calibrated. As part of the Indian Point (New York) thermal study, heat transfer pans were installed at the site and basic data were obtained. Heat transfer coefficients were also determined using pans floating on the pond adjacent to the rotating boom.

Special heat transfer pans were designed and built to monitor the actual atmospheric effect on the model and, in the early years, to determine the surface heat transfer coefficient (see Figure 8). These metal pans, which evolved over time, consisted of a continuous water labyrinth with one pan heated by coils located underneath it and an adjacent, unheated pan. Using the change in temperatures between the pans and the thermal inputs, heat transfer coefficients were calculated. The metal pan was mounted in a wooden box, with insulation between the pan and the box to reduce the heat transfer from all sides except the open top. The water in the pan was gently moved by a simple rotating paddle. Measurements of electrical energy input into the coils, water tem-

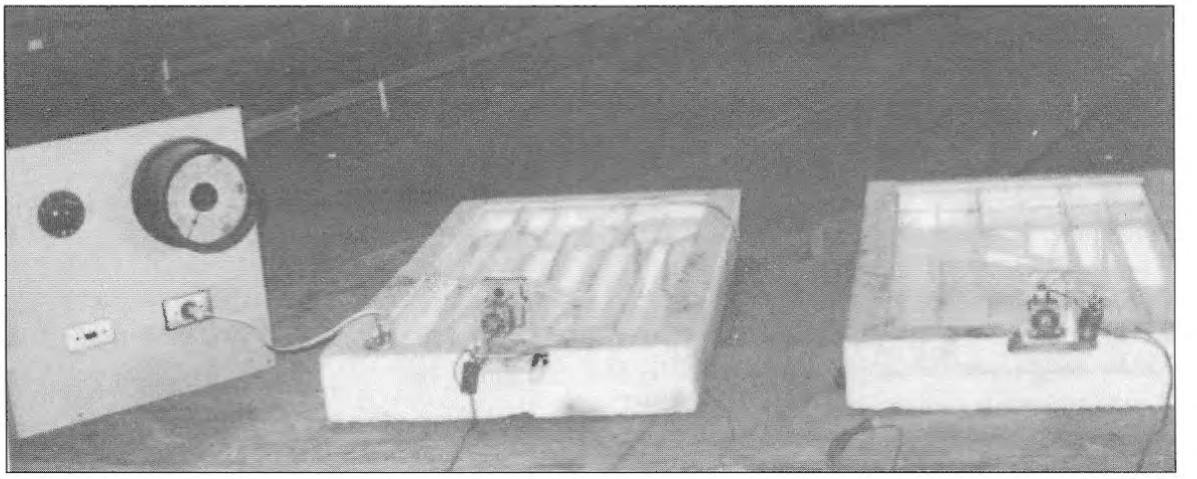


FIGURE 8. Special heat transfer pans.

perature, air temperature above the water, and wet and dry bulb temperatures were continuously monitored and recorded by the data acquisition and storage system. The wet bulb temperature was obtained by a "home-made" instrument consisting of a thermocouple inserted in a cotton wick coming out of a small bottle of water. A small fan blew over the wick with the required velocity as specified in the standards. The total effect was to continuously monitor and adjust the temperature of the ambient atmospheric conditions surrounding the thermal discharge. When steady-state conditions occurred to within about ±0.5°F, the test would begin by starting the thermal discharge.

As experience was gained in thermal testing, it was concluded that the pans could be simplified by eliminating the heating elements. The pans in effect became a monitor of atmospheric conditions in the building, and ambient water conditions were adjusted so that only the effect of the thermal discharge was measured.

The Yorktown diffuser for Units 1 and 2 was one of the early major thermal multi-port diffusers to be built in a tidal estuary where temperature would build up to a steady state over subsequent tidal cycles. The diffuser was built at a 30-degree angle from the shoreline, and contained ten 28-inch-diameter nozzles spaced 50 feet apart with a total diffuser length of 450 feet. The nozzles consisted of a simple elbow arrangement coming off the top of the diffuser and discharging flow at 14 feet per second horizontally approximately 27 feet below the water surface into the river. A second similar diffuser, which was 780 feet long with fourteen 42-inch nozzles spaced 60 feet apart, was also studied for a proposed Unit 3 at the same plant.

A cursory look at a thermal multi-port diffuser might lead one to believe that a standard design could be used for all such diffusers. However, local receiving water body conditions of currents, tides, bathymetry, plant flows, plant temperature rise above ambient, bottom materials and intake location all influence diffuser design. There are only three things that are more or less standard in a design:

- First, the water velocity at the exit of the nozzles should be in the order of 10 to 15 feet per second (preferably at the high end) to keep the momentum high enough to obtain good mixing. The upper limit is based on the economics of the size of the diffuser and nozzles and pumping costs, which increases at the rate of the square of the velocity.
- Second, the ratio of the nozzle area to the diffuser pipe or tunnel area has to be such that each nozzle discharges at approximately the same velocity.
- Finally, the optimum centerline distance of the nozzles from the bottom bed contour was determined in a 1973 Worcester Polytechnic Institute (WPI) master's thesis by one of Alden's graduate students, Robert Mattson, to be two nozzle diameters. At

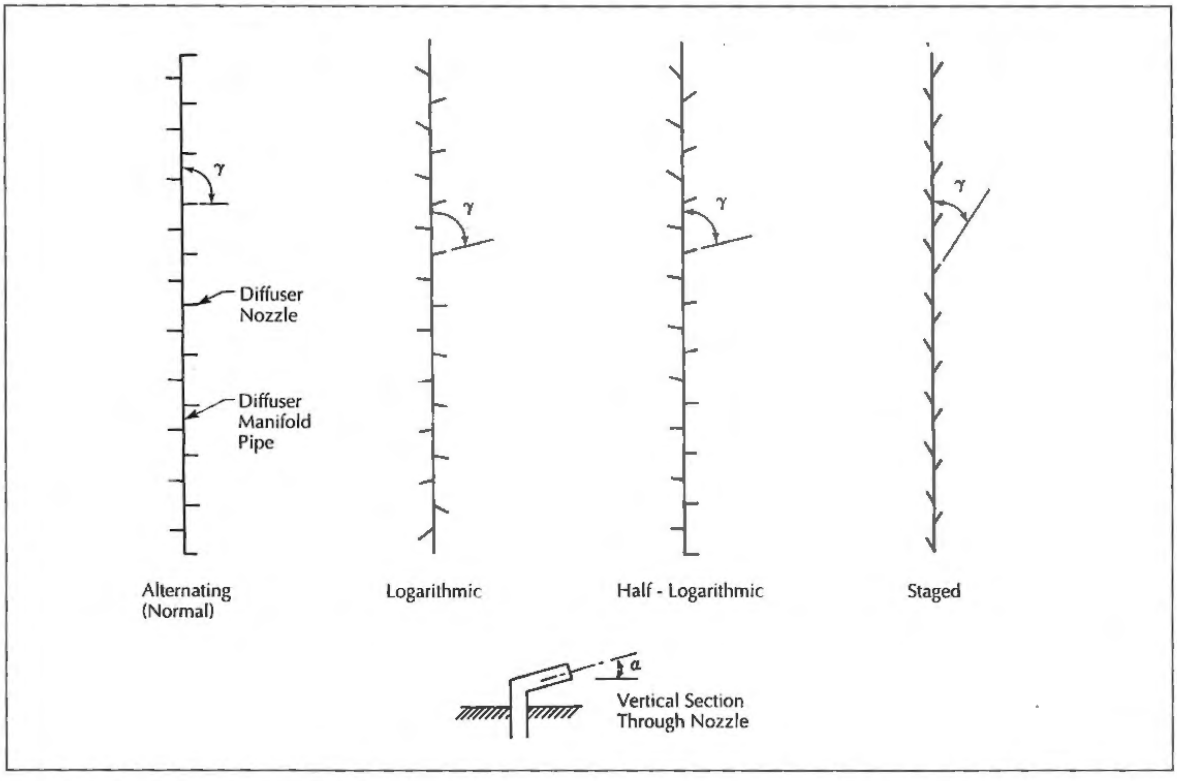


FIGURE 9. Diffuser nozzle orientations.

this optimum distance erosion cannot take place, and the jet should not cling to the bottom (causing reduced entrainment). Also, the nozzle must be oriented upward so that the high-velocity jet can entrain all the flow that its energy will permit.

Nozzles exiting from the diffusers can have many configurations. The basic ones studied at Alden were (see Figure 9):

- a 90-degree or more elbow coming off the top of the diffuser;
- horizontal or angled upward emerging from the centerline of the diffuser;
- a multiple nozzle riser off the top of the diffuser;
- staged diffusers having alternate discharging nozzles; and,
- slotted rectangular nozzles projecting from some submerged diffuser structure.
-

The latter was sometimes used to bring into compliance thermal discharges that were not "grandfathered" by new regulations.

An unusual situation occurred in 1972 with the Martins Creek Plant. The Delaware River Commission imposed thermal criteria for the plant, which was located on the Delaware River about 20 miles northeast of Allentown, Pennsylvania. Using a 1:25 model, a twelve-nozzle diffuser was developed. However, because the minimum submergence was only 2 to 3 feet, surface turbulence caused by the twelve jets was objectionable. To spread out the discharge and increase submergence, a 23-nozzle diffuser was situated perpendicular to the shoreline and followed the riverbed contour. The nozzles varied along the entire length and were also the shallowest at minimum river stage. The diameter of the nozzles varied from about 7.5 inches near the shore to 18 inches further out in the river (see Figure 10).

The first model to have a computerized printout of temperature rises above ambient was for the Indian Point Plant (Units 1 through 3). The model was tested along with the heat loads of the Bowline and Lovett plants, which are located in that portion of the Hudson River. A diffuser study for Bowline had previously been completed in 1971.

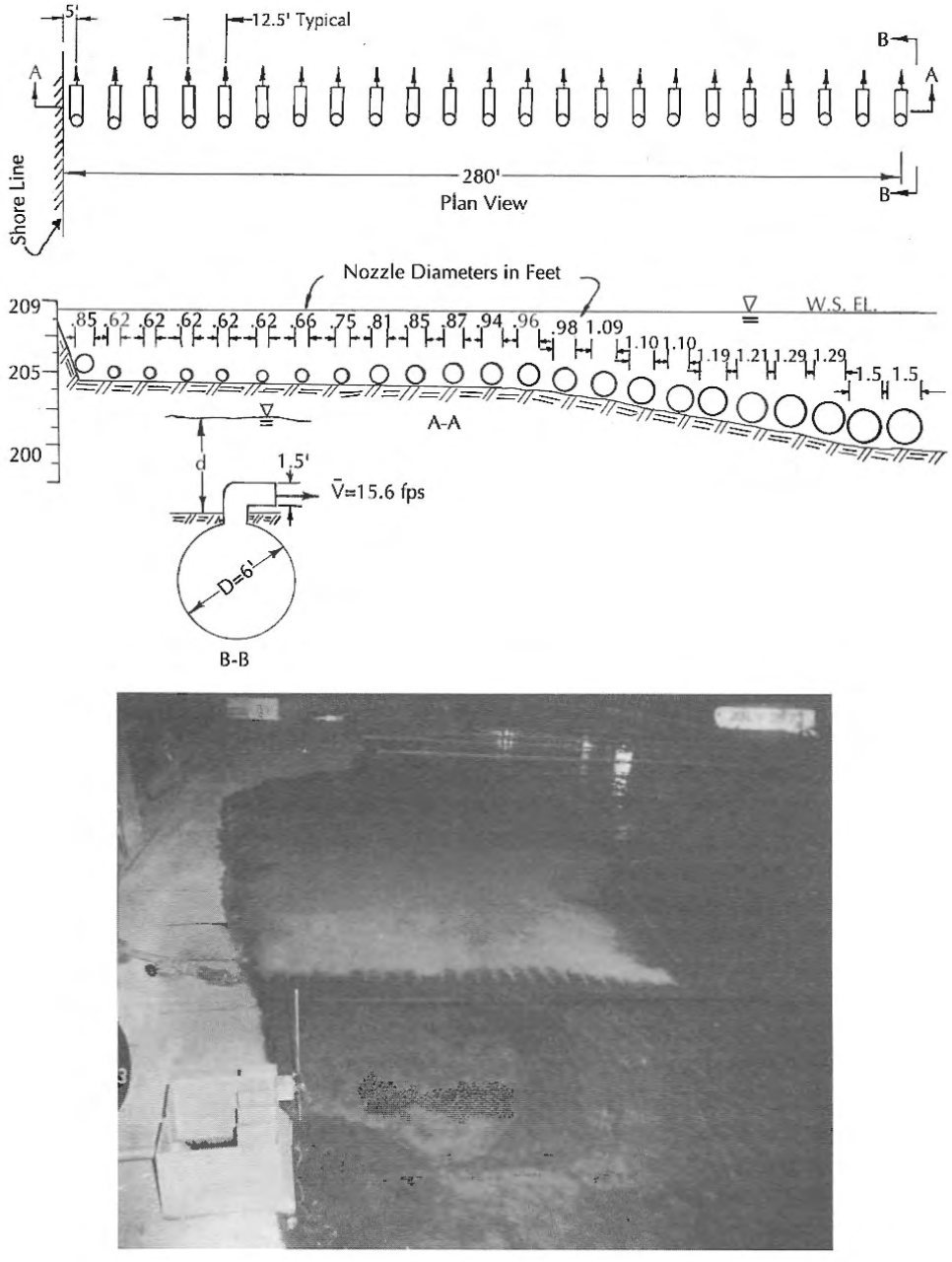


FIGURE 10. Martins Creek multiport diffuser.

A high-speed datalogger was used to scan all sensors in 17 seconds. The analog signals were converted to digital values of millivolts and were recorded either on magnetic or punched tape. Digital computer processing was used to convert the signals in degrees Fahrenheit, compute and then list temperature rises above inflow ambient temperatures. The computer also recorded temperatures throughout the model for five consecutive cycles at the same corresponding times in the tide cycle and averaged the temperatures for these five cycles. The data system had a resolution of ± 0.05°F and a repeatibility of ±0.15°F.

The New York State criterion for this section of river was that a temperature rise over 4°F should not exceed two-thirds of the surface river width nor over more than half of the river cross section. The criterion was satisfied with two diffusers — one for Units 1 and 3, and one for Unit 2.

The Units 1 and 3 diffuser was 850 feet long, had thirty-five 3-foot nozzles spaced 25 feet apart, and discharged flow at 10.5 feet per second. The Unit 3 diffuser was 600 feet long and had 25 nozzles with the same diameters and spacing as the other diffuser. Its discharge velocity was 11 feet per second. Both diffusers were situated at 45 degrees from the river centerline with the nozzles pointing downstream. What was peculiar with both diffusers was the angular variation of the nozzles to the diffuser. A logarithmic distribution of nozzle orientation along the diffuser was required to optimize lateral spreading of the effluent plume.

Studies involving typical power plants with diffusers having vertical risers containing multiple nozzles were conducted for the Fitzpatrick Plant in New York in 1969 and the Seabrook Plant in New Hampshire in 1977. Both plants had two nozzles per riser, with Fitzpatrick having a total of twelve 30-inch nozzles and Seabrook having a total of twenty-two 32-inch nozzles. One interesting aspect of the Seabrook nozzles was the original design. When a large-scale model of the riser with its two nozzles was first tested, it acted like a fluidic device. In a fluidic device, small pressure perturbations in the flow can cause the flow to shift from one port to the other. In the riser study for Seabrook, large amounts of flow would first emerge from one nozzle, then a short time later the large flow would leave the other nozzle. The alternating pattern continued periodically through the test. After it was confirmed that the inflow into the model riser duplicated that expected in the field, the riser containing the two nozzles was redesigned to eliminate the alternating phenomena.

One of the more intensive physical studies of diffusers was performed in 1978 for the proposed Jamesport Nuclear Power Plant in New York. The plant (to be located on Long Island Sound but never built) had a restrictive thermal criterion of not exceeding 1.5°F outside a 500-acre zone. The study looked at nozzle orientation, diffuser length and diffuser performance. In the orientation study, tests were run with alternating nozzles, a logarithmitic distribution, a half-logarithmitic distribution and a staged diffuser. It was concluded that a long staged diffuser resulted in a smaller mixing zone.

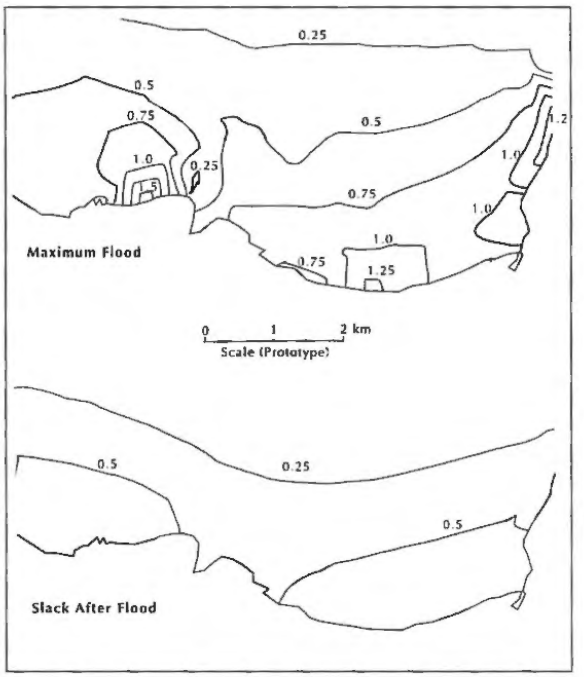


FIGURE 11. Background temperature rise isotherms.

The last very large hydrothermal physical model was built in 1986 for a power plant in Taiwan. This model had over 700 thermo-couples, and the comprehensive study also included mathematical modeling to study background temperature buildup at the site (see Figue 11). Near-field plume analyses of alternative surface discharge and diffuser designs were completed (see Figure 12). These math models utilized different discharge configurations and were aimed at minimizing the number of tests that would be required in the physical model, thereby reducing the cost of the overall study.

Analytical Thermal Modeling

The Taiwan mathematical model in 1986 for thermal discharge was not the first such analytical model created at Alden. Analytical model studies had been conducted over a decade before. The first major analytical thermal model

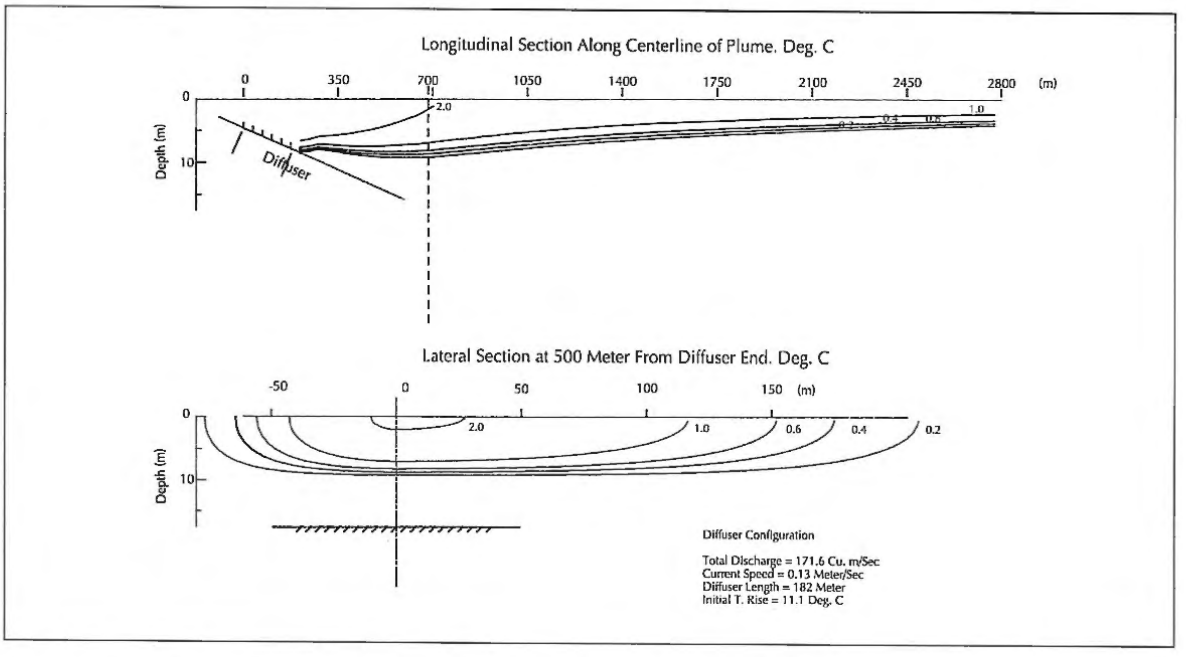


FIGURE 12. Cross-sectional isotherms from the proposed Taiwan diffuser.

study was started in 1974 for the Seabrook Station. Two types of analyses were conducted for Seabrook's thermal discharge to augment the physical hydraulic model study results. Near-field, buoyant jet diffuser analyses were used to quantify the effects of ambient vertical temperature and salinity concentrations on the plume dynamics and surface temperature rises. A second study, a far-field analysis, was used to predict the extent of the plume beyond the area covered by the physical model. The study utilized the equations of mass, momentum, thermal energy and salt in conjunction with various empirical expressions for drag, friction, entrainment, spread and surface heat transfer. The benefits of analytical modeling, which is now a standard practice at Alden and other consulting firms, are optimizing a design and determining which variables are most important. Then, the most favorable designs can be physically tested to confirm or further optimize performance.

Incidentally, the Seabrook Project led the way in having the largest number of studies performed at Alden. Thirty-one studies were recorded in the files related to Seabrook, and include analytical, waterhammer, backflushing, intake, sump, startup, thermal model, circulating water pump, diffuser, testimony, evalution, pipe and meter calibration. Part of Alden's versatility was embodied in this one project.

Soon after George E. Hecker became Director of Alden in 1975, one of Harleman's students at MIT, Dominique Brocard, came to Alden and eventually became the Assistant Director. During his stay at Alden, he used skills acquired from his mentor to conduct analytical modeling of thermal discharges. One of his first models, in 1978, was for the Mercer Generating Station on the Delaware River. A transient, one-dimensional model was used to account for the effects of the meteorology and tidal velocity fluctions in the Delaware River. An integral jet model that allowed the prediction of the three-dimensional temperature rise patterns produced by the heated plant discharge and modified to account for the effects of the opposite bank on confining the plume was superimposed to predict temperature rises in the river.

In the same year, a similar study was completed for a proposed power plant site in Charlestown, Rhode Island. The following year, a number of such models were studied analytically. The area in the East River around the Ravenswood Station in New York City was studied with another analytical model. Besides including the effect of this one station, the heat discharged by five other plants was taken into

account. Two field surveys were made, and they showed favorable agreement with the mathematical model.

In the early days of analytical models, Alden did not always have the in-house computer capability to do the complete study. Being associated with WPI became a distinct advantage at these times since the school had large computers. Alden frequently leased time from WPI to utilize their computers.

In 1980, a simple advection/diffusion model was used to predict the background heat buildup from the Egyptian Ismalia Plant discharging in Bitter Lake. The study also included a 1:64 physical scale model used to predict the degree of recirculation that would be encountered in the field.

A combination of two, two-dimensional finite element codes were used in 1985 to predict the time-varying far-field temperature rises and the indirect recirculation produced by an enlarged power station at Castle Peak Station in Hong Kong. The study utilized two codes, developed by MIT — CAFE, which calculated the tidal circulation patterns (see Figure 13) and DISPER, which calculated the distribution of a tracer such as heat. Combinations of plant loads and a range of tidal conditions and prevailing winds were analyzed.

The same programs were used in 1987 to analyze the temperature effects of cooling water discharging into a section of Maine's Saco River from the York County Waste-to-Energy Plant. The plant discharged into a pool created by the Spring, Cataract, Bradbury and West Channel dams.

The last thermal mathematical model of Alden's first century was in 1992 for the Ao Phai Power Plant in Thailand. The study involved an investigation of three discharge schemes: surface channel, near-shore pipe discharge and off-shore submerged diffusers. The analysis was conducted in two portions to predict the plume isotherms in the near field and to predict the background temperature rises induced by the far-field phenomena of dispersion, tidal flushing and heat transfer to the atmosphere. Finite element methods to study the far-field and integral-type analysis (using Alden's SBJET program) were used for the near field.

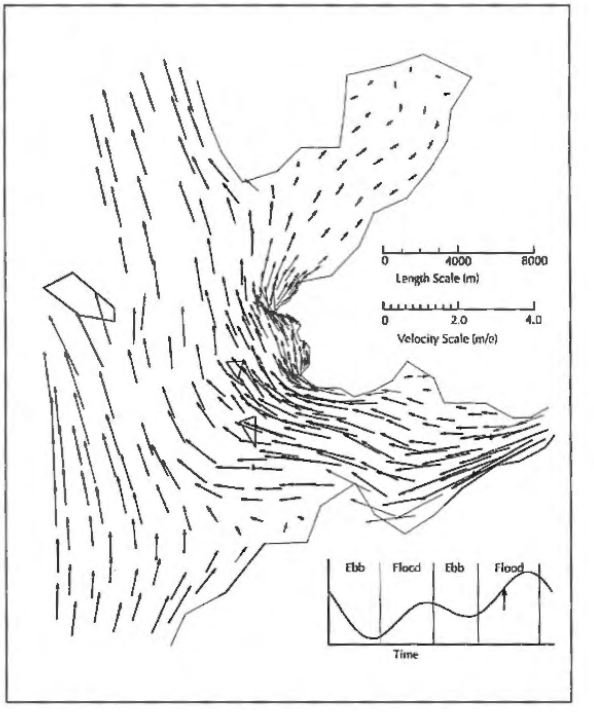


FIGURE 13. Numerical modeling of currents.

Computational fluid dynamics had reached the stage where thermal analysis using these techniques in simple near-field cases for discharges and ambient conditions was always possible. On the other hand, physical models are still required for more complex cases, such as plumes interacting with boundaries, or other plumes, reversing currents or the need for detailed thermal patterns. Costs of thermal studies with physical models can sometimes be minimized by using mathematical models to eliminate numerous schemes before testing begins or in conjunction with the testing of the physical model.

Hydroelectric Power Models: Overview

Alden has been involved with many aspects of hydraulic modeling for hydro stations since the 1920s. Models included spillways, plant alignment, intake and discharge flow patterns and gate studies. By the 1960s, most major U.S. hydro sites had been developed, and the utility companies shifted emphasis to large steam stations, and then the addition of pumped storage projects. With the latter exception, hydro power modeling diminished in the 1970s, until

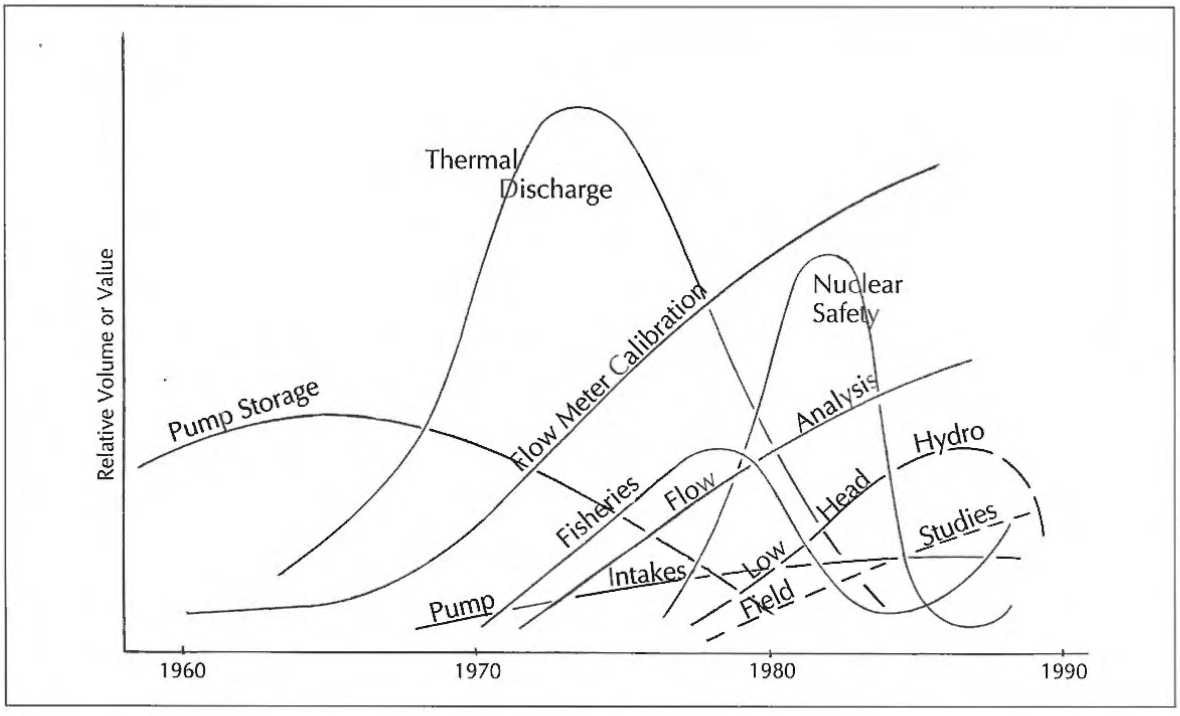


FIGURE 14. The variation of funded research at Alden.

the oil embargo and favorable legislation occurred. By the late 1970s and all through the 1980s and into the 1990s, hydro power modeling would again become a very substantial activity at Alden. The hydro modeling in the 1970s and 1980s would include new areas that had not previously been major areas of work—hydro models where navigation and sedimentation were major concerns.

As in so many other fluid-related areas, the work would come in "waves." Some of the waves would be sharp peaked and others much longer. On a relative scale, the various waves of work at Alden have included thermal modeling, which probably had the highest peak and longest base, and others such as pumped storage and field studies (see Figure 14).

Large, detailed intake models for hydro power stations became common in the 1970s because the total head at many sites was as low as 15 to 20 feet, and bulb or tube turbines were being proposed to minimize powerhouse excavation and cost. These units (not used in the United States until the late 1970s) were sensitive to approach velocity patterns and the turbine suppliers had strict requirements for approach flow distributions at the powerhouse intake. The re-emergence of hydro power would also favor Alden's traditional activity in field turbine testing. In 1970, hydro power accounted for 16.2 percent (64,000 megawatts) of the nation's generating capacity. By 1991, 10 percent of the nations's electric power came from hydro with its installed capacity reaching 92,000 megawatts.

Federal Law & Relicensing

When hydroelectric power was first conceived in the late nineteenth century, there were only a few federal or state laws regulating the construction of structures in streams and the industry itself. This policy was gradually changed through legislation that tried to preserve the public's interest in public lands as well as navigable rivers.

The Federal Water Power Act of 1901 applied to public lands, and the Act of 1906, as amended in 1910, applied to navigable rivers. The Act of 1901 limited the rights on public lands to mere revocable licenses. Licenses obtained under this act reflected the views of administrative officers and were subject to change at discretion. The license for power plants on navigable rivers, under the Act of 1910, required a special act of Congress

in each instance and was limited to fifty years. Plans were subject to Federal authority, and locks or other navigable facilities might be required at the time of the original construction or at some later date. The acts contained no expression of policy nor did they specify what would happen to the projects at the end of fifty years. It is easy to see that these acts made it difficult to obtain financing to construct hydroelectric projects.

The legislation that changed the picture was the Federal Water Power Act of 1920. Under this act, which was a contract between the government and the licensee, an applicant could secure a license for up to fifty years. The license specifically spelled out the conditions that the licensee had to fulfill, and these could not be changed unless the licensee breached the contract. When the license expired, the government could take over the site for its own use, could permit the site to be taken over by others, or could issue another license to the original licensee.

If the properties were taken over by the government or by others before the license expired or at the termination of the license, a just compensation had to be paid to the licensee. This compensation, called a net return, was based on the licensee keeping good and accurate financial and technical records. If the site was not taken over by the government or others, the licensee was entitled to a new license on such terms as was reasonable in view of the conditions existing at the time of renewal.

When this Act of 1920 passed, it favored municipalities over private interests in issuing licenses. Regardless, it was so much better than the previous legislation that it promoted the financing and construction of many of the river hydroelectric plants. By 1922, 39 licenses had been issued under this act, for a total of 2,040,000 horsepower. In addition, the Federal Power Commission still had 309 applications for licenses. For the preceding 19 years, only 863,000 horsepower had been developed.

The 1920 Federal Water Power Act resulted in model work being performed at Alden related to dams, spillways and powerhouses starting in the early 1920s. However, more work was generated by 50-year relicensing than by the original act. The Federal Energy Regulatory Commission (FERC), which replaced the Federal Power Commission by the Department of Energy Organization Act of October 1977, listed from two to 18 projects each year from 1970 to 2001 up for relicensing (except for 1993 with 173 projects and 2000 and 2001, with 36 and 33 projects, respectively). From 2002 to 2033 16 projects per year are scheduled to come up for relicensing.

Besides the Act of 1920, other legislation affected the hydroelectric industry in the twentieth century. In 1978, the OPEC oil cartel was in control of the world's oil supply and caused fuel oil and gasoline shortages in the United States. Home heating oil cost rose over 40 percent within two years. As a result, Congress passed the five-part National Energy Act of 1978. One section of this act was the Public Utility Regulatory Policies Act of 1978 (PURPA), which was designed to reduce U.S. dependence on foreign oil and encourage the development of small power production facilites, particularly small hydro power. PURPA established the rules and regulations regarding the purchase and sale of electricity from small power producers and cogenerators. The act mandated that utilities must buy the power produced by small hydrogenerators at a rate not to exceed the cost to produce the power itself or purchase it from another (*i.e.*, the avoided cost). In essence, utilities had to buy high-cost power from these small, relatively expensive (because of their small size) units. In the 1990s, these contracts would cause financial difficulty for some utilities because competitors could supply cheaper power. Also, the utilities themselves were generating less expensive power because of over capacity, but the contracts had to be honored or "bought out."

As a further tax incentive to encourage small power development, Congress passed the Crude Oil Windfall Profit Tax Act in 1980 (COWPTA), which was modified in 1981 and 1982 by the Economic Recovery Tax Act and the Tax Equity and Fiscal Responsibility Act. Significant tax benefits became available to public and private developers. Lastly, PURPA allowed the FERC to exempt small hydro facilities from most federal and state utility regulations for facilities up to 30 megawatts.

Many projects that qualified under PURPA were very small, had limited funding and were not as tightly regulated as larger hydro projects.

As the result, they usually refurbished old existing sites and did not require the services of Alden. Other projects, which required special optimization or interacted with navigation facilities, were model tested and became a major source of new work for Alden. As a direct result of the legislation and the greater cost of fuel, the "gold rush" for low-head hydro began. There was voracious competition to get FERC licenses at existing dam sites. Many of the small sites (from 5 to 10 megawatts) that had operated from the 1920s to the 1960s, but had been abandoned in favor of big steam units, suddenly had value to utilities. Numerous old stations were brought back on line. Many stations had their capacities increased because of the greater value of the site and the water being spilled. An industry for low-head, mini-head and micro-hydro plants developed, with many major engineering firms and new small ones getting into the field.

When hydro project licensees neared license renewal time, they inquired about some of the ambiguous parts of the Act of 1920. To clarify this act, Congress passed the Electric Consumers Protection Act of 1986 (ECPA). Congress clearly gave the FERC the authority to regulate the hydroelectric industry. Under the ECPA, the FERC has the ability to revoke licenses for violations of the act or to impose fines up to $10,000 per day per violation.

The ECPA requires existing licensees to notify the FERC at least five years in advance of the license expiration date on existing licenses for which relicensing will be sought. The ECPA also requires those applying for relicensing to make any technical information they may have available to competitors for the site. In spite of this, the relicensee is favored by the act. Also, contrary to the 1920 Act, municipalities are not favored by the ECPA.

Environmental Considerations & Hydropower

If the early environmentalists were happy about the fifty-year license limitations they sought under the 1920 Act, they had to be ecstatic by the provisions of the ECPA. Relicensing required the FERC to consider state recreational planning as outlined in the Statewide Comprehensive Outdoor Recreation Plan (SCORP) required under the Land and Water Conservation Fund Act. In addition, recommendations made by the National Marine Fisheries Service, the U.S. Fish and Wildlife Service and the state's fish and wildlife agency had to be factored into the FERC's decision-making process. To illustrate the complications, the U.S. Fish and Wildlife Service has to take into account the Endangered Species Act, the Fish and Wildlife Coordination Act, the National Environmental Policy Act and the Clean Water Act of 1972 when making their recommendations to the FERC. Even the recommendations of Native American tribes have to be factored into the FERC's decision.

Under licensing or relicensing, the FERC is required to investigate whether the sites are being, or will be, efficiently utilized and properly managed. Utilization may require changes in the geometry of the structures or the site, or may necessitate updates or changes to the powerhouse turbines. Alden did a number of studies from the late 1970s to 1994 that involved such modifications.

Hydro Studies

One of the early (1980) hydro intake models during this period was for a small hydro project proposed in Lawrence, Massachusetts. A company wanted to develop the power potential at the existing Essex Dam on the Merrimack River. Optimizing inlet flow conditions and maximizing net head were the priorities.

From 1981 to 1987, Alden conducted four studies relating to modifications to existing low-head hydroelectric projects. The first project (located in Brunswick, Maine) was an intake structure to two tube turbines being used to reduce the spillage over the flashboards on the dam during periods of high flow. The intake had a square penstock with two elbows in the same plane, with the second elbow ending in a transition from square to round. Proper flow conditions were obtained at the turbine entrances and were verified in the model using velocity traverses at the model turbine inlet and a swirl meter at the turbine location.

The second project was for the Worumbo Hydroelectric Project located in Lisbon, Maine. This model study was conducted in two phases. The first phase called for a study of a proposed powerhouse with three tubular turbines, having a total flow capacity of 7,500 cu-

bic feet per second (cfs), to replace an existing plant having a total flow of 900 cfs. The model results indicated that the canal and headgate structure had to be modified and the bottom of the head pond had to be excavated to accommodate the substantial flow increase. In the second phase, the powerhouse location replaced a portion of an existing dam. During this study, rating tests were conducted for the spillway, and various headrace evacuation schemes were evaluated.

The Lockwood Hydroelectric Project on the Kennebec River at Waterville, Maine, was studied to investigate flow patterns at the intake and head loss from the head pond to the tailrace. The six old units in the existing plant were to be overhauled and a new unit, with twice the flow of each existing units, was proposed for construction next to the plant.

Finally, the fourth project was the Lewiston Falls Hydro Project on the Androscoggin River in Maine. This study of a new plant involved the evaluation of flow patterns approaching the intake and losses in the approach channel. Cofferdams used to keep water out of the project during construction were also evaluated for various flow conditions.

Another low-head project involved replacing five turbines with a total flow of 1,100 cfs with three units having a total flow of 2,130 cfs. The proposed units (near Paterson, New Jersey) would occupy three of the five existing intakes. Besides studying the intake flow patterns, the model was used to evaluate the head losses. Intake flow patterns are very critical in all low-head hydroelectric power plants; improper approach flow conditions can be the source of a high percentage of the head loss through the structure. Poor flow patterns outside the structure can also be the cause of reduced turbine performance.

For the Emporia Hydroelectric Projects on the Meherrin River in Virginia, a vertical shaft Kaplan turbine was proposed to replace a wet pit turbine. The model evaluated the modifications to the existing structure and measured velocity distribution and swirl immediately upstream of a vaned elbow leading to the turbine. During the study, vortex activity was observed in the forebay and was eliminated using vortex-suppression devices.

In 1986, a model study for the new Hydro Kennebec Station in Winslow, Maine, focused on inlet flow patterns for the turbines, crest gate performance, flow conditions in the tailrace and other related issues. The pit turbines had a flow capacity of 4,000 cfs at a head of 27 feet.

One power company proposed to abandon ten units, with a total capacity of 8,000 cfs, and replace them with two pit turbines with a capacity of 15,200 cfs at the Swan Falls Project on the Snake River in Idaho. The new powerhouse would be built on the east side of the river where the existing bypass sluiceway was located. The model showed a large flow along the face of the ten abandoned units toward the new powerhouse, causing poor flow patterns at the entrance to the new units. Guide walls were used to remedy the problem. In addition to this problem, the model showed that the excavation planned in front of the new powerhouse was excessive and could be minimized without affecting the turbine performance. This factor alone saved the utility more than the cost of the model.

Sometimes model studies conducted for the same client and on the same river were performed simultaneously, which was the case for the Crescent and Vischer Ferry Hydroelectric Projects on the Mohawk River near Schenectady, New York. Both powerhouses underwent structural renovations and the study was conducted to evaluate the entrance to the turbine to determine if better flow patterns could be achieved. The Vischer Ferry study also included the development of a new flow-regulating structure to bypass flow by the powerhouse when necessary and the development of an ice boom.

Navigation Models

Some utilities were increasing their electrical production capabilities utilizing features of the 1920 Act that were not previously used. American Electric Power Corporation was the first private utility company to build a hydro power project at a federal lock and dam facility. In 1978, Alden evaluated intake velocities and the navigational impact of building a hydro power project on an Army Corps of Engineers (COE)

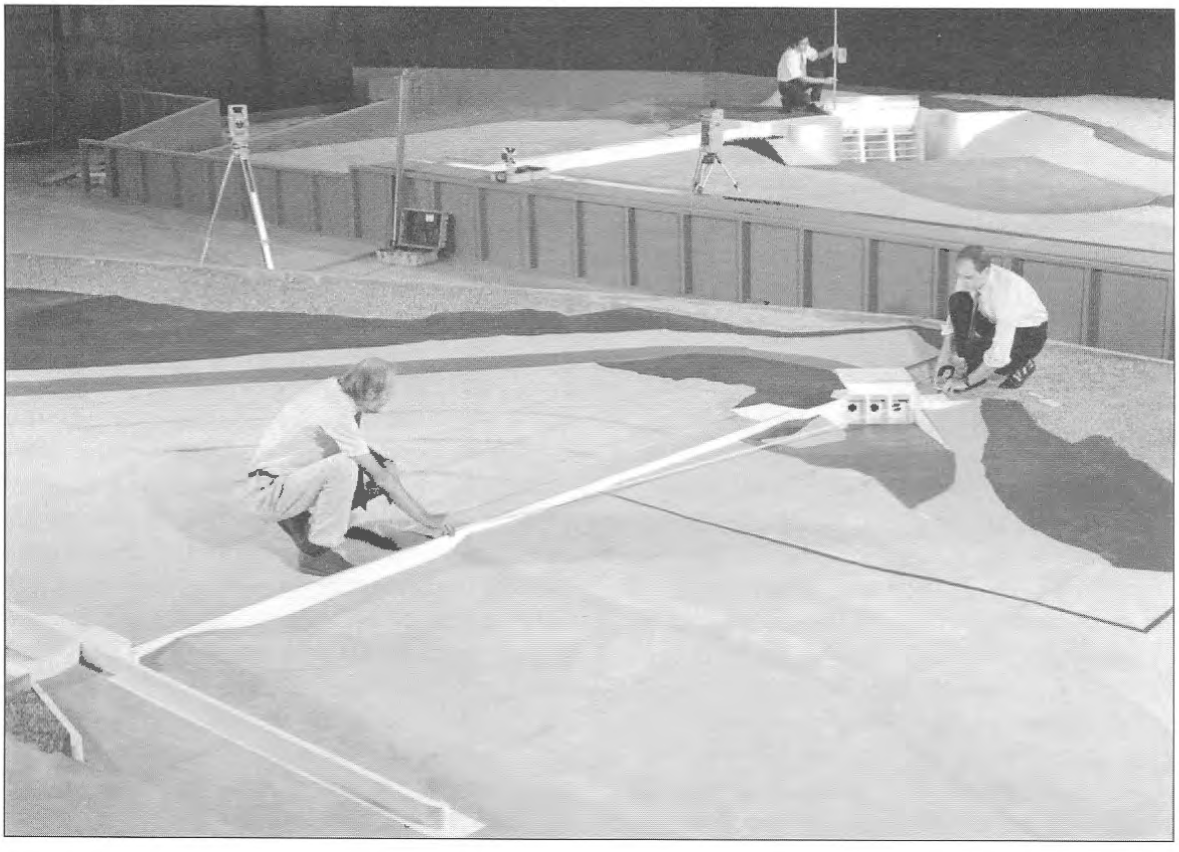


FIGURE 15. Overall navigation model (foreground) with separate detailed powerhouse model (background) for the Demopolis Project.

operated lock and dam on the Ohio River, 40 miles downstream of Parkersburg, West Virginia. The run-of-the-river power plant was to be situated on the right bank of the river and would be called Racine. In general, these types of studies required two or three models to address issues raised by the COE and turbine suppliers. A detailed large-scale intake model was used to evaluate velocity distributions. A second overall model (typically 1:100 to 1:150 linear scale) was used to evaluate currents and surges that could affect navigation. In some cases, a third distorted scale model addressed sedimentation.

The overall type of model became known as a navigation model. Three other navigation models, Demopolis (see Figure 15) on the Tombigbee River in Alabama, and Lock and Dam Nos. 13 and 9 on the Arkansas River in Arkansas, were tested at Alden in later years. Because of the large size of these models, technical considerations and experience dealing with the COE, Alden was one of only two or three laboratories in this country interested in doing this type of work.

Since Racine was the first such private project on a COE site, the COE had no written specifications or formulated review procedures at that time. As the result, positive interaction between Alden, COE and the project owner was necessary to see the project through. The 1:150 model showed that the power plant discharge proposed in the original design created a large eddy at the lower lock approach during low river flow conditions. This eddy impeded barge traffic into the lock. A relatively small excavation in the tailrace that confined the plant discharge to the adjacent river bank remedied the problem. Other navigational concerns were studied by using time-lapse photography of a model non-powered barge train coasting into the lock and by running the model with and without the power plant in operation. Later models would have a scaled motorized tug and barge.

In the Racine navigation model, velocities were measured using floats with candles and a

rotating segmented plate in front of an overhead camera. The same method had been developed 30 or more years earlier (see Figure 16). Reduction of these data is time consuming and requires a photographer. When it was realized that more of this work would be coming, Alden developed a computerized float-tracking system. This method allowed flow patterns to be determined from video recordings and plotted using computer-aided design (CAD) software. The output was velocity vectors that were then superimposed on project drawings.

All concerns of the COE were addressed, and Racine was built. It is currently producing power and has not encountered any difficulties. The major general requirements of the COE included:

- no increase in upstream flood levels;
- no impact on lockage cycle time; and,
- no impact on dredging.

By the time the Demopolis Hydro Project model study was commissioned in 1983, the COE had written specifications relating to model studies. The concerns on the Demopolis site related to the modifications of river currents and surges imposed by the power plant discharge. Very long excavated intake and discharge channels were needed for this proposed plant. The model showed local downstream cross-currents from the hydro plant discharge in the lower lock approach area where the push tows would cut power and subsequently lose maneuverability. The problem was remedied by selective discharge channel excavation in the "Selma chalk," which comprised the river bank flood plain, and directing the plant discharge flow toward the bank opposite the navigation lock. A second, larger model of only the power plant intake area was built to study the flow patterns at the entrance to the turbines. Flow patterns approaching this area were obtained from the overall model and imposed at the inlet to the powerhouse model. Towards the end of the study, basic COE approval had been secured for the construction of the project, but, unfortunately, the project ran into financial difficulties and was canceled.

In 1984, a detailed powerhouse and an overall navigation model was commissioned for a

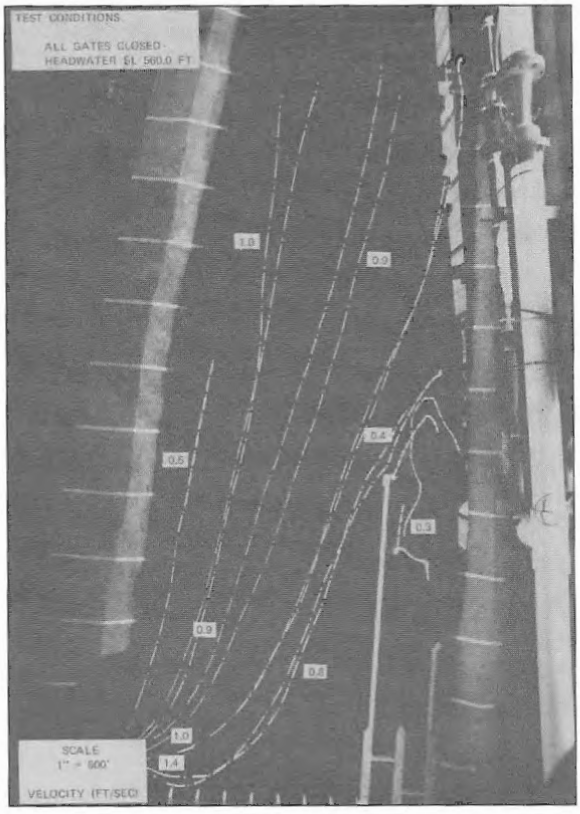


FIGURE 16. Streak lines and velocities in a navigational model.

proposed 33-megawatt hydropower station to be located at Lock and Dam No. 13 on the Arkansas River near Fort Smith, Arkansas. The results from the model studies showed acceptable flow conditions at the lower lock approach and minimized head losses through the powerhouse. The tailrace configuration was also improved through the use of the model, effectively widening the river downstream of the dam and permitting a more unrestrictive passage of flood flows. Despite the encroachment of the overflow embankment by the powerhouse, the tailrace allowed project flood flows to be passed without increasing the upstream water elevations. Proper lock performance with the added powerhouse was verified through the use of a COE remote-controlled scale model of a barge tow entering the lock both upstream and downstream. The tow model indicated forces acting on the tow with and without the power plant in operation.

For this navigation model, a new construction method was developed and utilized. Con-

tour maps of the prototype were digitized using a linear resistance gauge for position and a manual entry key pad for elevation. These data were stored in a computer and used to print out a continuous sheet of the model elevations versus model position along a transect in the model. This sheet was then pasted on a thin aluminum film and cut along the elevation line using a bandsaw. A rotating laser leveling beam was used to install the templates using spot levels printed by the computer on the template paper.

Sediment Transport Modeling

Because the Arkansas River transports a tremendous amount of sediment during flood stages and is highly developed from a navigation standpoint, the COE required that the proposed hydro plant's impact on river morphology be studied with a movable bed model. A separate model for Lock and Dam No. 13 was constructed for this purpose. The model was operated in the transient mode simulating a few years of typical river flows. Because of the COE's experience in this type of modeling, it required Alden to use coal as the movable material in the model, and the coal had to be crushed and graded relative to size and density.

The movable bed transport models required innovation in many aspects of modeling in order to quickly construct, operate, calibrate and provide a model that met the COE's requirements while producing reproducible model data. The first challenge was obtaining a stockpile of clean coal having the required density and gradation. Various suppliers were investigated, and handling techniques were developed. Aluminum templates were built and placed in their proper positions in the model. The model was then filled by hand with coal to the correct elevation on the templates and the templates were gently removed to prevent movement of the coal. The model was also equipped so that it could be stopped at any time in the flow cycle and be drained from the bottom to prevent the coal from moving during this process. The procedure was reversed for restarting. The difference in the transport of bed material around the site both with and without a power plant in place was determined. Of special importance was the required maintenance dredging in the entrance areas of the lock.

The Lock and Dam No. 13 model also used a computerized system, developed on some of the earlier large tidal models, that allowed automatic regulation of inflow, outflow, spillway gates and powerhouse flow. Such automation of data was especially important in the repeatability of the hydrograph for all the different series of tests. These systems became so developed that the model could operate around the clock with little or no continuous staffing, allowing a much quicker turnaround for evaluating the numerous tests required by the COE.

A technique to reduce hydrographic data was also developed for this model. At different times in the river flow cycle and at the end of the cycle it was necessary to obtain the river contours. In the past, an engineer used a level and rod to obtain contours. In a model of this size, this method would take two or three men many days to survey and get accurate results. Level I beams with a traveling platform incorporating an x-y position system were used to locate an elevation rod at the point of interest. An operator moved the rod tip to the coal bed and pressed a button to record the position and elevation in the computer. A few hours after completing the survey, the operator could see the contour map of the river bottom.

After construction permits were issued, building of the powerhouse at Lock and Dam No. 13 was started while the model study was still ongoing. It was decided that the position of the plant had been fixed by the model study and that further changes developed from the study would be only in the plant approach or discharge areas. In fact, the dedication of the plant took place before the final model test, to verify some previous data, had been completed at Alden.

In 1988, Alden performed similar studies for a proposed 42-megawatt hydro station at Lock and Dam No. 9, located on the Arkansas River near Morrilton, Arkansas. The study was also to include a navigation model, a movable bed model and an intake model. Rather than having three distinct models, as had been the case for Lock and Dam No. 13, it was decided to construct the movable bed model in the same area as the navigation model. Keep in mind that the

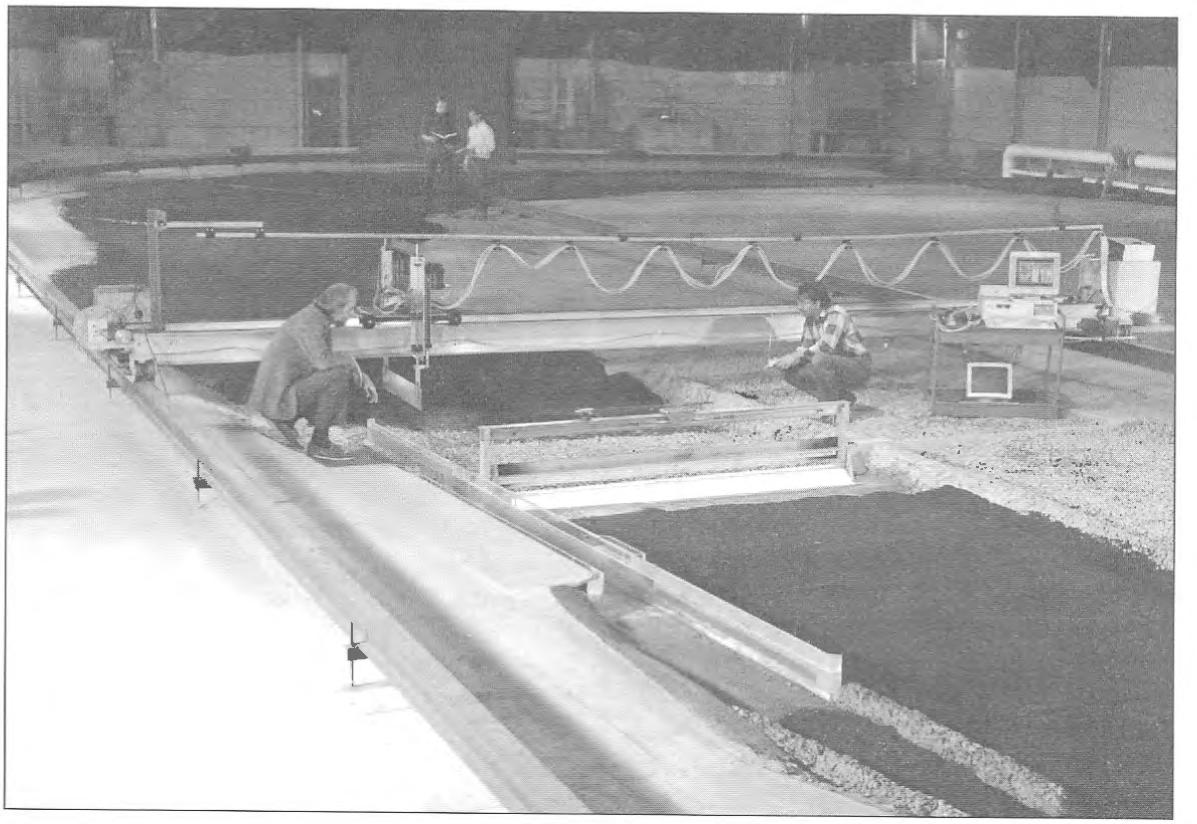


FIGURE 17. Computer-controlled screeding device for moveable bed model.

navigation model was constructed to facilitate removal of those model parts that would not be common to both models. To be cost effective, this method required that all navigation model studies be completed before starting the movable bed studies. If successful, the method would save space as well as construction time.

To reduce the labor in preparing the movable bed, another Alden innovation was developed. The reproduction of the river bed topography in the earlier Lock and Dam No. 13 model had been labor intensive and time consuming. A computer-driven screeding system to accurately form the river bottom was designed based on the same type of platform used to gather the river topography information in the Lock and Dam No. 13 model. The platform contained a stepper motor driving the screeding apparatus through a chain drive. The screeding apparatus was a flat blade with each end operated by chain driven stepper motors (see Figure 17). The system was operated by a personal computer containing the topographic information and utilizing x-y-z coordinate feedback from the platform. Manual labor was only necessary to maintain a small excess amount of coal in front of the flat blade. This device was a tremendous success and demonstrated its reliability and advantages over the hand molding of the river bed by repeatedly and accurately forming the river bed for the many diverse studies conducted in the movable bed. There certainly was no lack of praise for the system, especially from those who had shoveled and placed the coal in the Lock and Dam No. 13 model.

Sediment transport models, or the so-called movable bed models, were not new to Alden. The first such model tests were conducted in 1957 for the Elrama and Dickerson projects. Tracer studies, where movable material indicates movement tendency, were done much earlier. The materials used to simulate the movable bed varied depending on the fall velocity and the specific gravity of the prototype material, the model scale ratio, field data on erosion patterns and calibration tests.

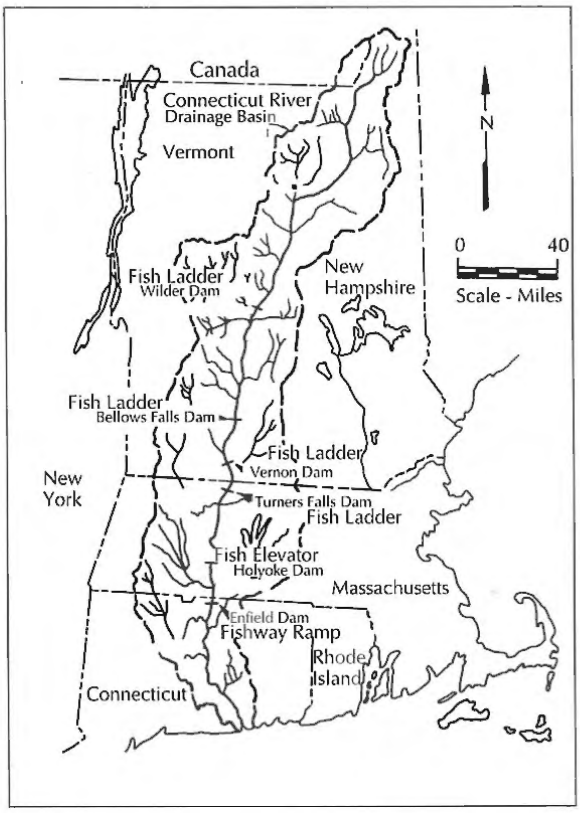


FIGURE 18. Connecticut River drainage area.

Relicensing & Fish Passage

Perhaps the most difficult parts of licensing or relicensing hydroelectric power plants since the passage of the ECPA of 1986 have been recreational and environmental issues, especially as they related to fish, wildlife and botanical resources. Before this act, the utility put up a dam, spillway and powerhouse, and began producing power. After the ECPA, these same structures, which were impediments to fish, became a source of frustration to the owners or potential owners who tried to foresee the FERC's demands on environmental issues. Utilities were perfectly at ease with the technical nature of power production, but few had the necessary staff to study and recommend changes or additions to existing facilities to satisfy the FERC's environmental requirements. Utilities had a choice of staffing or hiring outside consultants to satisfy the ECPA. Both paths were followed, and Alden worked with both utilities and consultants on fishways, fish screens and fish behavior.

With the building of numerous dams in the 1800s and 1900s, people had become increasing concerned about the cessation of fish migration, and limited studies on fishways for upstream passage were begun. Many early dam builders had made some effort to provide upstream passage. By the 1940s, successful passage of shad had been demonstrated at the Bonneville Dam on the Columbia River in Washington state, which prompted regulatory and other personnel to investigate shad fishways for other rivers in the United States.

One river that benefited from Alden's studies was the Connecticut River (see Figure 18). From the time that the Holyoke Dam was built to supply water to mills, shad and Atlantic salmon were prevented from accessing their spawning grounds in the upper reaches of the river. Shad had at one time gone as far as Bellows Falls, but by 1849, they could go no farther than Holyoke. Salmon, on the other hand, had spawned as far as Beecher Falls, Vermont, on the Canadian border. By 1798, they were limited to only as far north as Turners Falls. Sixteen years later, the salmon spawning migration had ceased.

Starting in the early 1950s, with a study for the Holyoke fish elevator, Alden has been involved in developing fish ladders or other passage facilities for four other dams on the Connecticut River, finally opening up the entire river to the spawning fish. In 1955, the first year of operation, the Holyoke elevator passed a disappointing number of shad, only 4,899. Today, it is not rare to see over a half million shad pass this elevator, mostly in the month of May.

Successful fish ladders need to satisfy a number of criteria. First, the fish have to find the entrance way, which requires attraction water that is fast enough to be sensed by the fish but not so strong that the fish cannot swim against its current. If the attraction water is from a different source than ladder water, the melding of the two waters near the entrance needs to be smooth, with a velocity distribution as uniform as possible across the cross section. To achieve this, a floor diffuser is used where internal hydraulics must be optimized. The gradient down the ladder cannot be too large, nor can the velocity be too high to prevent the fish from going from one pool to the next. Also, at every pool or drop in

the ladder, there has to be a relatively calm area where the fish can rest, if necessary, and not be carried down the ladder.

Hydraulic models are useful in the design of fish facilities because detailed velocity distributions and overall flow patterns can be measured at critical locations and evaluated. Based on previous experience, the internal hydraulics, entrances, weirs or other components can be quickly optimized.

Fish ladder types also depend on the species of fish. West Coast ladders for salmon consist of a series of pools created by small weirs. The salmon go up the ladder by jumping over each weir into the next pool. In the Connecticut River, the predominant fish is shad, normally a non-jumper. The fish ladders in this case are usually of the vertically slotted type.

In 1973, approximately 20 years after the effort at Holyoke, Alden studied the Turners Falls fish passage, the first dam upstream of Holyoke. The fishway was designed to handle 40,000 Atlantic salmon and 850,000 shad. At this location, fish have two ways to get over the dam. At the upstream part of the dam, there is a gatehouse that controls water to a long power canal. The Cabot Station is at the far end of the canal and discharges into the Connecticut River. Fish have an option of using a 66-pool ladder at the Cabot Station or a 42-pool ladder at the spillway. All fish, whether they went up the Cabot Station or the spillway ladder, pass through the gatehouse fishway, which contains viewing, counting and diverting areas. Fish can be captured at this location for study or transport to other areas. With the 1973 Turners Falls model study, Alden began a series of model studies on fish passage that continued to the present, and will continue in the future as long as there are fish-related relicensing issues.

The Turners Fall fish passage model was used to investigate the location of the fishway entrance relative to the dam and flow coming from the overflow gates. The model contained a portion of the dam, two bascule gates and a portion of the downstream channel topography. The topography was extremely rough, and locating the fishway entrance was challenging to say the least, since fish need to sense the attraction water in order to find the ladder. The gatehouse slotted fishway was also evaluated and optimized. In 1986, the gatehouse ladder entrance and a portion of the Cabot Canal were studied when increased flows in the canal were being considered. The Cabot Station and spillway ladder consisted of a series of pools, each 1 foot above the other, separated by baffles having open ports such that the fish could swim over the baffle or through the port to reach the next pool. Finally, in 1991, due to fish mortality in the Cabot Station log sluice fish sampler, Alden was asked to look at different geometries for the opening in the sluice bulkhead gate slot, the inclination and the length of the sampling screens, the arrangement of the lateral guide walls and the screen porosity.

Alden studied the fish ladders for the next three upstream fish ladders on the Connecticut River. The first study was for the Vernon Dam, located between Vernon, Vermont, and Hinsdale, New Hampshire. The fish ladder was located on the west bank of the river, adjacent to the powerhouse. The studies of the slot type ladder, containing 25 pools, included the measurements of the pool flow patterns, the head drop between pools, the slot velocity distribution and the pool power dissipation.

Downstream Fish Migration

Although it was important to get fish upstream, regulatory agencies also became concerned with the plight of fish going downstream, either through the turbine or over the spillway. There had been wide ranging estimates of high fish mortality due to turbines. Some experience indicated that mortality through the slowly turning hydroturbines varied with the species, type of machine and head. With Kaplan and bulb turbines, the mortality of some species can be as low as about 5 to 10 percent. Depending on site specifics, Francis turbines may produce higher mortalities.

Downstream fish migration methods needed to be investigated and, in 1991 as part of a proposed capacity increase, a downstream fish migration scheme for Vernon Dam was studied at Alden. Based on velocity data, it was determined that modification of an existing trash boom with a vertical 15-foot skirt would be effective in directing fish to a trash sluice. When the plant expansion did not take place, an overall model was used to evaluate flow

patterns approaching a fish pipe that would take downstream migrants around the hydro station. A detailed model of the fish pipe was used to evaluate internal hydraulics.

The fish ladder at Bellows Falls, located between Rockland, Vermont, and North Walpole, New Hampshire, was studied in 1980. Three different methods of supplying attraction water were studied:

- water from an existing sluice;
- a diversion conduit attached to a proposed tube turbine draft tube; and,
- a closed conduit from the head pond.

The downstream migration studies at Bellows Falls were started in 1989. The method to guide the fish was similar to the Vernon Dam study in that it used a boom with a 15-foot skirt to guide fish to an existing ice and trash sluice in the Bellows Falls power canal. Two alternate schemes were also studied. One called for a bypass at the southeast corner of the forebay, and the other bypassed along the powerhouse face to the west end of each trashrack bay.

The fish ladder studies for the Wilder Station, located between Hartford, Vermont, and Lebanon, New Hampshire, commenced in 1982. The study included a portion of the main entrance weir collection channel and the attraction water diffuser, including the first two pools. An investigation was also made relative to the supply of the attraction water. This water would be supplied by a proposed turbine to be installed in an empty south bay of the powerhouse.

A downstream fish migration scheme consisting of a floating boom with a 10-foot skirt leading to a bascule gate was investigated for the Holyoke site. This study also evaluated the partial removal of a submerged dam to determine its effect on velcoitiy patterns and downstream migration.

Fish Passage in the Susquehanna River Basin

The predominant fish on another river, the Susquehanna River in Pennsylvania, is also shad, and four dams block their passage to 300 miles of their spawning grounds. By agreement between the four utilities owning the dams and the supervising governmental agencies, all four dams were to have fish passage facilities by the year 2000, thus allowing shad, river herring and other migratory fish access to the upper reaches of the river.

Fish passage model studies have been conducted at Alden for three dam sites on the Susquehanna River: Conowingo, Holtwood and Safe Harbor. A proposal for a study at the fourth dam site, York Haven, has been submitted and is currently awaiting approval. (Some of these dams were modeled approximately 70 years earlier by Alden.)

All three proposed upstream fish passage facilities for the Susquehanna River were fish elevators. When the Alden studies began, only one small lift facility was operating. This lift had been in operation since 1972, and was located on the west bank at the Conowingo Dam. Two Alden studies investigated east bank fishway entrance schemes at this site. The first study looked at a two-entrance concept. The concept for the second study involved three fish entrances, two floor diffusers and an energy dissipater in the flip bucket of the spillway gates used to supply water to the elevator. Part of the study involved minimizing air entrainment. For all three entrances, the velocity distribution was measured and adjusted to meet the requirements of fish biologists monitoring the tests.

At Holtwood, two fish elevators were studied. One elevator was located at the powerhouse and had two tailrace entrances. The other elevator, located at the spillway, had a single entrance.

For the Safe Harbor Dam, a number of models were tested in order to provide acceptable velocity distributions in the tailrace near the entrances. Another model was used to develop favorable internal flow hydraulics leading from the entrance to the fish elevator hopper, including three attraction water diffusers.

An interesting aspect of shad "tagging" was initiated by the Pennsylvania Fish Commission in 1985. The otolith, or earstone, of the shad grows a ring a day, similar to the rings on trees. It was found that when shad fry were immersed in a solution of tetracycline, the growth ring for that day absorbed the tetracycline. Using ultraviolet light and viewing the otolith under a fluorescent microscope, the tetracycline produced a yellow glow. By immersing the shad fry on different days, they could form

a code to identify the shad's origin. The commission has been doing this at their Van Dyke Hatchery for all shad since 1985.

Collection Systems, Barriers & Diversion

In additional to fish passage by means of fish ladders or elevators at hydro plants and other dams, there are fish protection requirements for all types of projects located on salt or fresh water sites. Protection systems can be classified into four categories: behavioral barriers, physical barriers, collection systems and diversion systems.

One of Alden's first fish protection studies was performed in the early 1960s. The model was approximately 80 feet long by 35 feet wide. The model river water entered the model at one end and was regulated by a gate at the downstream end. The intake structure being studied was located between the two ends of the model. For some unknown reason, at some point in the study the client decided to see if fish would be attracted to the intake structure. The Alden Director at the time, Professor Hooper, tried to convince him that you could not model fish, and that most certainly with a fixed-end model, the fish behavior in the model would not represent its actions in the prototype. Nevertheless, the client insisted in going ahead with the fish test. A staging was erected in the rafters to quietly observe the fish behavior. At the appointed time, 500 small fish were trucked to the model and released. They all immediately moved to the upstream end and proceeded to dine on the incoming food that must have been in the water. For hours, the technician laid on his stomach on the staging, recording that nary a fish had left the group at the model inlet. The results were probably reported as inconclusive.

In contrast to this early study, Alden has been involved in numerous successful fish-related studies, starting with the Easton hydraulic fish screen in 1968, up to the current Eicher screen study. Alden's facilities and flexibility enabled many varied studies to be performed (see Figure 19). It is necessary to perform full-scale tests with various species of live fish under controlled conditions because fish behavior cannot be predicted. When basic technology is being developed using a variety of species, full-scale tests in the field are not appropriate. Site-specific studies, however, are now successfully done. With the environmental concerns that developed in the 1960s and 1970s, the power industry became interested in all aspects of fish barriers and handling.

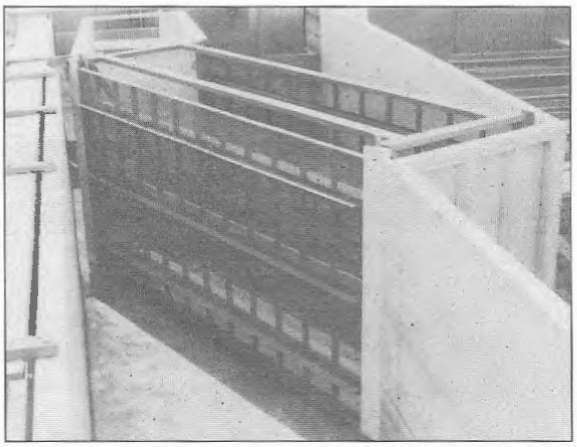


FIGURE 19. Angled screens and fish bypass in an outside basin.

The first study that used live fish was started in 1973 for the Nine Mile Point Nuclear Plant, Unit 2. Located on Lake Ontario, Unit 1 has a cooling water circulating system with an offshore intake and a near-shore discharge. For Unit 2, a Water Quality Certification required biological (fish) studies. With the possible exception of salmon studies in the Pacific Northwest, it is believed that this was the first large-scale systematic effort by a utility to investigate barriers, handling and diversion of fish away from an intake. The purpose of the initial study was to investigate fish behavior related to different aspects of the project. Hydraulic and biologic (fish) studies were conducted to investigate the performance of full-scale devices that would be incorporated into the intake. In some cases, a scale model was first tested to evaluate head losses with the proposed device. At the same time, for the most promising devices, a section would be evaluated at full scale with fish. Louvers, chains, baffle screens, as well as other devices were tested using alewife and smelt. Entrapment problems had occurred previously with these species. (These extensive studies continued until 1977.)

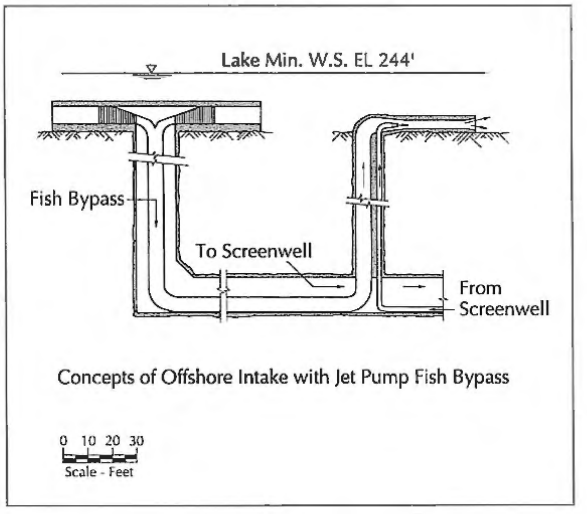


FIGURE 20. Concept of an off-shore intake with peripheral jet pump fish bypass for the Nine Mile Point studies.

At first, a holding facility had to be built. Since control fish would be used in each experiment, duplicate systems were required. It was decided to use 190-gallon galvanized oval horse troughs and 500-gallon round plastic swimming pools. The troughs were coated with a non-toxic epoxy paint, and each tank was piped with copper pipe to a sand filter to remove any large particles. Aquarium air pumps bubbled air through a stone discharge to supply oxygen to the fish. The first large group of fish arrived at Alden on a Friday afternoon and the fish were placed in the holding tanks. The next morning, the technician who arrived to feed and check the fish found them in a peculiar position, belly up. Investigation revealed that they had been poisoned by the various elements in the metal piping. The systems were immediately re-piped using only non-toxic plastic piping. The next batch of fish fared better, but after a while the multitude of fish in each tank began to kill themselves due to their own wastes. Including a biological filter in the system solved that problem. However, there was still some unexplained mortality. Finally, it was determined that fish need some current to swim against. The discharge water from the filters was then directed tangential to the side of the tanks, causing a rotating motion in the tanks — with the higher velocity water near the edge of the tank and nearly still water at the center. The fish were now happy exercising in their physically and biologically clean water.

Fresh and salt water fish were used in different studies. These fish were trucked to Alden from different sources. Once on the site, the biologists were responsible for an accounting of all fish. The freshwater pools were readily converted to saltwater pools by using a granulated aquarium product mixed with water.

Fish Transporting

Fish entering the offshore vertical intake pipe at the Nine Mile Point Station would be subjected to sudden pressure changes (see Figure 20). To study fish behavior under these conditions, a 1-foot square tank with a viewing window was constructed. The tank was filled three-quarters full of water, and fish were introduced into the tank. The alewife, smelt or coho salmon were allowed to become acclimated before the pressure inside the tank was quickly increased from 0 to 36 pounds per square inch (psi), to simulate their decent into the intake tunnel. They were held at 36 psi for 15 minutes, simulating the horizontal passage in the intake tunnel. Finally, the 36 psi pressure was released in a 2-minute period. When initially subjected to the 36 psi, the fish's air bladder was compressed, causing the fish to become disoriented and swim with their tails lower than their heads. They quickly recovered, however, and then swam normally. When the pressure was released, no ill effects were observed. After each test, the test fish were returned to a holding tank and observed for a few days. Comparison of these fish with control fish indicated no difference in mortality between the two samples.

Pumping is one method to transport diverted fish. Two methods to accomplish this were investigated. In the first method, a special mechanical pump designed to pump fish was acquired from Chile. This pump was a centrifugal pump with few vanes and a low rotation speed. Tests showed satisfactory results, but the feeling was that another type of pump — a jet pump — would be more advantageous. A literature search was conducted on core and peripheral type jet pumps, after which physical models of each were constructed and tested (see Figure 21). Although the peripheral jet pump required more input energy than the core jet pump to induce a given suction flow,

the peripheral jet pump had better fish pumping characteristics. Jet pumping tests of smelts and coho salmon showed no noticeable undue stress on these fish. Alewive fish, on the other hand, displayed some fragile tendencies during the same testing.

As a further study on the jet pump, fish were introduced directly into the shear zone of the jet pump. Since the alewives had been shown to be the most fragile, they were used in these tests. Velocities in the shear zone were varied, and the fish were introduced first all head first and then all tail first. Careful inspection of all parts of each fish were made in addition to observing their behavior in the holding tanks after the tests.

Another part of the collection system was the piping. A 10-inch piping system was built containing many transparent acrylic plastic sections for viewing purposes. Fish were introduced in the pipe, and the velocity was increased sufficiently to cause downstream fish movement. In the first section of straight pipe, the fish were observed to be swimming facing the flow. After passing through a 90-degree elbow, the fish became disoriented due to the complex flow pattern in the elbow. After the elbow, the fish toppled through the pipe. After emerging from the pipe, the fish swam in strange ways in the holding tanks until they had regained their equilibrium. Their performance was analogous to the behavior of humans after they become dizzy.

Screens & Louvers

As part of the overall study, screen and louvered types of guidance structures were first studied in the 3- by 3-foot flume in the basement of Alden Building 2. Tests were performed using all three species of fish, which were collected after going through a bypass at the end of the structure. After the success of the small flume, an outdoor facility using vertical axial pumps and a large indoor flume using a ship bow thruster were built to study full-scale guidance systems at prototype velocities. Systems for the Nine Mile Point Project, as well as the Indian Point Project, were investigated in these facilities. In the outside facility, a screen, angled at 25 degrees to the flow, led to a 6-inch vertical bypass opening. From here, the fish entered a 12-inch pipe to a collection area. A weir type gate in this area controlled the flow, and a lift basket collected the test fish. In any one test, roughly a few hundred to a thousand fish were introduced upstream of the screen and went through the bypass area.

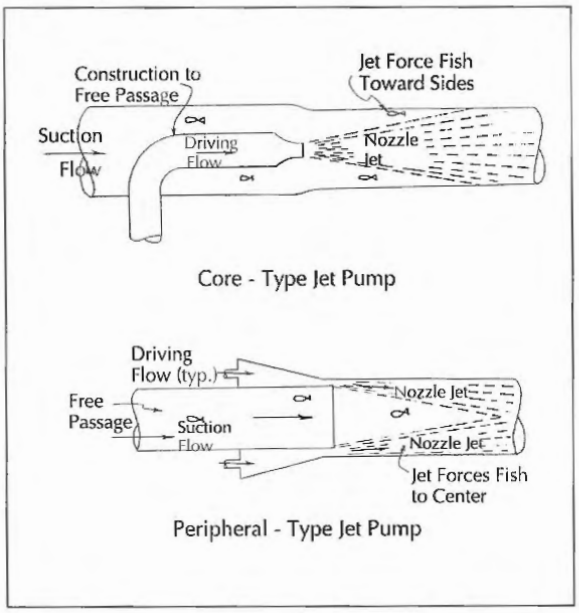


FIGURE 21. A comparison of core-type and peripheral-type jet pumps in relation to potential injury to fish.

There was concern about trash accumulation on the screen guidance systems. Studies using real trash were performed to investigate this question. Trash was measured by weighing, after which it was introduced upstream of the angled screen. After reaching steady state, the trash that remained on the screen and that went through the bypass was weighed and recorded.

After studying the various components of a fish guidance system, they were assembled into the complete system and tested. The system consisted of the angled screen, the bypass, the jet pump, the piping and the collection area. These tests revealed nothing different than what was found when the components were tested alone. However, a major general outcome of all the studies was the effectiveness of the angled, flush-mounted traveling screen with a number of species. A bypass pipe with a jet pump was the preferred means to transport the fish back to their environment.

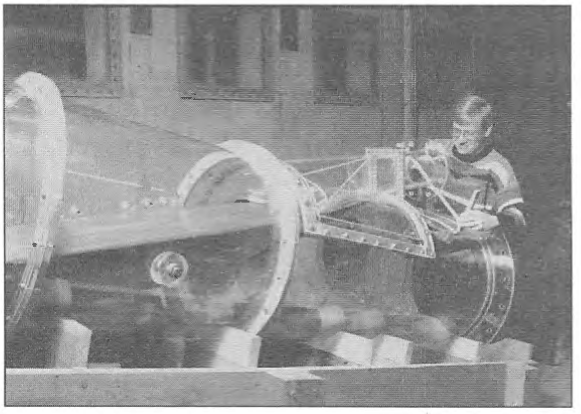


FIGURE 22. Model for screen bypass tests.

Eicher Type Screen & Intake

While some of these studies were being conducted at Alden, George Eicher was designing a fish bypass system for downstream fish migrants at the T.W. Sullivan Plant on the Willamette River in Oregon. Lacking room in the forebay for a traveling screen, Eicher decided that his only alternative was to place an inclined screen in the 11-foot diameter penstock. The screen was made of wedgewire and was inclined at a slope of 19 degrees to the penstock axis. A bypass pipe was installed at the downstream end of the screen to collect fish and any debris, such as trash. The design proved to be a success, and Eicher patented the system. In 1992, Alden conducted a model study to improve the screen geometry near the bypass and to streamline the bypass itself. A photo of the model with the improvements is shown in Figure 22.

The original prototype screen was installed in Washington State at the Elwha Hydroelectric Project. This screen had undergone model testing at another laboratory prior to being installed in a 9-foot penstock in Elwha. In the spring of 1989, the screen was 99 percent effective in bypassing downstream migrating coho salmon smolt. The purpose of the Alden study was to reduce the peak value of the velocity component perpendicular to the screen and to minimize the head losses through the screen, by means of a possible re-design.

The Elwha Eicher screen was inclined at 16.5 degrees to the penstock and contained what is called a screen break at the upper 20 percent of the diameter. At this point, the screen had a –3-degree slope that ended in a bypass pipe. The approach penstock to the screen was not straight. It had a 16-degree bend approximately 3.8 diameters upstream of the screen, causing the approach flow to be asymmetrical. The Elwha screen, made of wedge wire, had three different porosities along its length. The porosity of the upstream portion was 63 percent open. At approximately 60 percent of its length, the porosity was 32 percent open, and after the screen break, the porosity was 8 percent open. The porosity was varied with the intent to improve the velocity distribution along the screen and thereby reduce fish impingement. The results of the model study indicated that a flat, uniform 50 percent porosity screen, with a re-designed simplified straight geometry bypass, satisfied all the requirements with less head loss and an improved velocity distribution.

During the model study, an error was uncovered in the as-built bypass compared to the drawings. The as-built bypass entrance was larger than the drawings called for, resulting in a larger bypass flow. Since the model was constructed from the drawings, model studies were done to see the effect of the change in the entrance.

Modular Inclined Screen

The Eicher screen was designed primarily to be used in an existing penstock at sites where there are no other physical means to bypass downstream migrating fish. In locations where there are no penstocks, a device called a modular inclined screen (MIS) can be utilized to bypass fish. An MIS consists of a streamlined entrance, a wedge wire screen set at a shallow angle to the flow (basically an angled screen) and a bypass to divert fish to a transport pipe or to a holding facility. The screen is set on an eccentric pivot shaft so that it can be rotated for cleaning by backflushing (see Figure 23). The MIS would be installed as a pre-assembled module directly in front of one intake bay, and fish would be guided to this intake.

MIS studies were performed at Alden in 1993. Three different test facilities were used to study various features of the MIS. A small flume was used to measure upstream and downstream velocities adjacent to the screen inclined at angles of 10, 15 and 20 degrees to the

flow and with porosity of 30 and 50 percent open. A 1:6.6 hydraulic model was used to study entrance effects, overall head losses and bypass configurations. A third facility, with a 1:3.3 scale of the entire MIS system, was used to evaluate fish reaction, diversion efficiency, immediate fish damage and possible delayed mortality. The three models showed that with relatively uniform approach velocities of 2 to 10 feet per second, the MIS had favorable hydraulic characteristics for fish.

Protection of Fish Larvae

Smolts and larger fish undergoing upstream and downstream migrations at power sites were not the only concerns of environmentalists and regulatory agencies. Fish larvae were studied at Alden from 1978 to about 1980. The purpose of the studies was to investigate several components of fine-mesh screening systems to determine their potential for collecting, diverting or transporting fish larvae with resulting low mortality. The larvae used in these studies were striped bass, alewife, winter flounder and yellow perch. Walleye, channel catfish and bluegill larvae were also used in 1979. The holding facilities were similar to those used in the smolt studies but of smaller size. The holding tanks were either small aquariums or large beakers set in styrofoam.

In all, some 1,500 studies were conducted to determine the mortality of larvae in different systems. The systems were classified into three groups:

- Modified, traveling water screens with fine-mesh screening, lifting buckets and low pressure sprays;
- Angled, traveling water screens with fine-mesh material and a bypass; and,
- Pumping units, jet and mechanical pumps used to return collected or diverted larvae to their natural environment.

Screen retention studies were conducted with flow velocities ranging from 0.5 to 2.0 feet per second and with mesh sizes from 0.014 to 0.079 inches. Larvae size measurements were made on a sample of 25 larvae for each of these tests. It was determined that screen retention was a function of mesh size relative to larval length and body depth. For some species, a mesh size of 0.02 inches or less might be required to retain all larval stages.

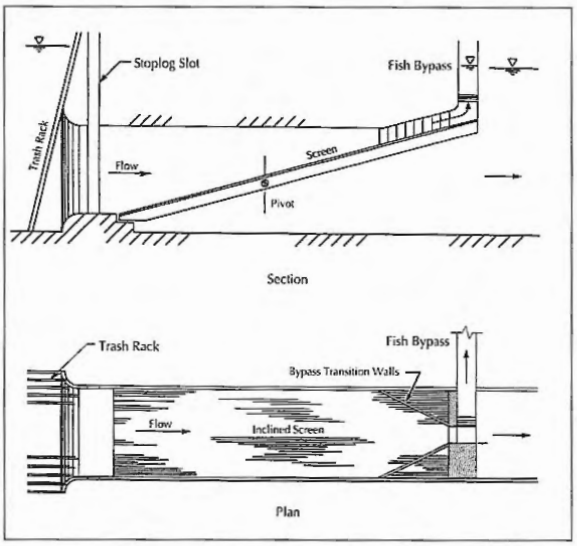


FIGURE 23. Plan and section views of a modular inclined screen.

Larvae were also subjected to air exposures, and their mortality evaluated from one to four days. These studies indicated that exposure time was extremely significant in larval mortality, and that air exposures of larvae for any reason should be minimized at all facilities.

As far as diverting larvae, angled fine-mesh screens have the potential to divert older larvae to bypasses, providing the proper mesh size and flow velocity are incorporated in the system. However, it was recommended that studies be conducted for each site prior to selecting the method of handling the larvae.

The jet and screw impeller centrifugal pumps acted differently with the various larvae species. The jet pump was very effective in pumping the striped bass larvae, while the centrifugal pump did a better job with alewife and yellow perch larvae. Once again, the conclusion was to conduct pretests before specifying a pumping system.

Divestiture of Alden Laboratory by WPI

Starting in the 1960s, the United States became a more litigation-minded nation. At all levels in government and private life, lawsuits became a major concern, particularly for those with

"deep pockets." By the 1980s, some communities were closing pools and playgrounds due to liability/litigation concerns. In this atmosphere, because Alden's projects involved mega projects, WPI became acutely concerned with liability, even though Alden had never been sued.

On October 17, 1985, while the large Lock and Dam No. 13 navigation model and others were in "full swing," the WPI errors and omission insurance on the Alden Research Laboratory and the Manufacturing Engineering Application Center (MEAC) terminated. WPI chose not to renew the Alden policy due to its high cost, even with a large deductible. Later, directed by the WPI trustees, the school's president and the vice president of business affairs negotiated with the Alden director and four Alden senior engineers to divest the world-renowned facility even though it had the largest WPI research budget.

In April 1986, the final agreements were signed by both sides, making Alden a private for-profit company. The five new owners who bought the business and the equipment were Albert G. Ferron, George E. Hecker, Johannes Larsen, James B. Nystrom and Mahadevan Padmanabhan. Hecker was voted president of the new corporation. One of the sale conditions was the retention of the Alden name by the new company. On May 12, 1986, the laboratory officially began as a new private company under the name of Alden Research Laboratory, Inc. — the fifth name for the laboratory since its inception. The names and the year they changed were as follows:

Hydraulic Testing Laboratory (1894)
Alden Hydraulic Laboratory (1915)
Alden Research Laboratories (1965)
Alden Research Laboratory (1977)
Alden Research Laboratory, Inc. (1986)

GEORGE E. HECKER *was appointed Director of the Alden Research Laboratory in 1975, when it was part of WPI, and became President in 1986 when Alden was separately incorporated. Prior to joining Alden in 1971, he worked for Stone & Webster in Boston and for the Tennessee Valley Authority before that. With more than 35 years of experience in solving flow problems using physical models, analyses and field studies, he has published widely and has served on many national professional committees. He has degrees from Yale and the Massachusetts Institute of Technology.*

ALBERT G. FERRON *was employed at Alden for 35 years. He also was an Adjunct Associate Professor of Mechanical Engineering at WPI. Upon his retirement from Alden in 1992, he was Vice President of the Flow Meter Calibration Section. Currently, he is employed at the University of Massachusetts Medical School in Worcester, continues as an Adjunct Associate Professor in WPI's Department of Civil & Environmental Engineering and is active in many community projects.*

BRUCE J. PENNINO *is Professor of Civil Engineering Technology at Springfield Technical Community College. Formerly, he was a Research Engineer at Alden for many years. He has a B.S.C.E. from Bucknell University and a M.S.C.E. from Colorado State University. He has over 30 years of civil and hydraulic engineering experience, and is a registered professional engineer in Massachusetts.*

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Lower Merrimack River Bridges

A review of the bridges along a fairly industrialized stretch of a river provides a unique way to portray the development and range of modern bridge design and construction.

LOLA BENNETT & RICHARD KAMINSKI

This article is a photo essay describing 14 of the present 24 bridges that cross the Merrimack River in Massachusetts, starting in Tyngsborough and continuing to Newburyport. The bridges span not just the river, but over a hundred years of bridge design and construction history. They provide a visual record of the different methods and approaches that have been used by bridge engineers over the decades.

Originating in the White Mountains of New Hampshire, the Merrimack River flows 110 miles on its way to the sea. The last 50 miles, known as the Lower Merrimack, begins at the Massachusetts state border at Tyngsborough and winds through a dozen communities in northeastern Massachusetts before reaching the Atlantic Ocean at Newburyport.

J.W. Meader, in his book, *The Merrimack River, Its Source and Tributaries* (1869), provided a vivid description of the river:

> *The Merrimack, dotted here and there with a variety of craft, from the light and trembling skiff to the heavy gondola, and the still more imposing and majestic moving ocean craft, with their broad, white sails and tall masts overshadowing the water, and, spanned with its bridges, flows calmly at its base, not in straight, monotonous course, but with a gentle meandering, of which the eye can never tire.*

Early settlers in the Merrimack Valley relied on this major waterway for sustenance and access to the interior from the sea, but the river remained an impediment to overland travel throughout the Colonial period, with ferries being the only means of crossing the river. By the close of the War of Independence, there were at least nine ferries between Tyngsborough and Newburyport. However, in an age of increasing industrialization, the need for a more permanent and reliable means of crossing the river grew stronger.

In 1792, the Massachusetts General Court authorized construction of the first bridge across the Merrimack River. By 1800, there were five bridges; by 1900, fifteen; today, there are twenty-four. At least 55 different bridges have spanned the Lower Merrimack River over the last 200 years, and half of the present bridges are the third or fourth generation to span their respective locations. The present structures range in age from 17 to 117 years old, incorporate a wide variety of materials, represent nearly every major bridge type and reflect the evolution

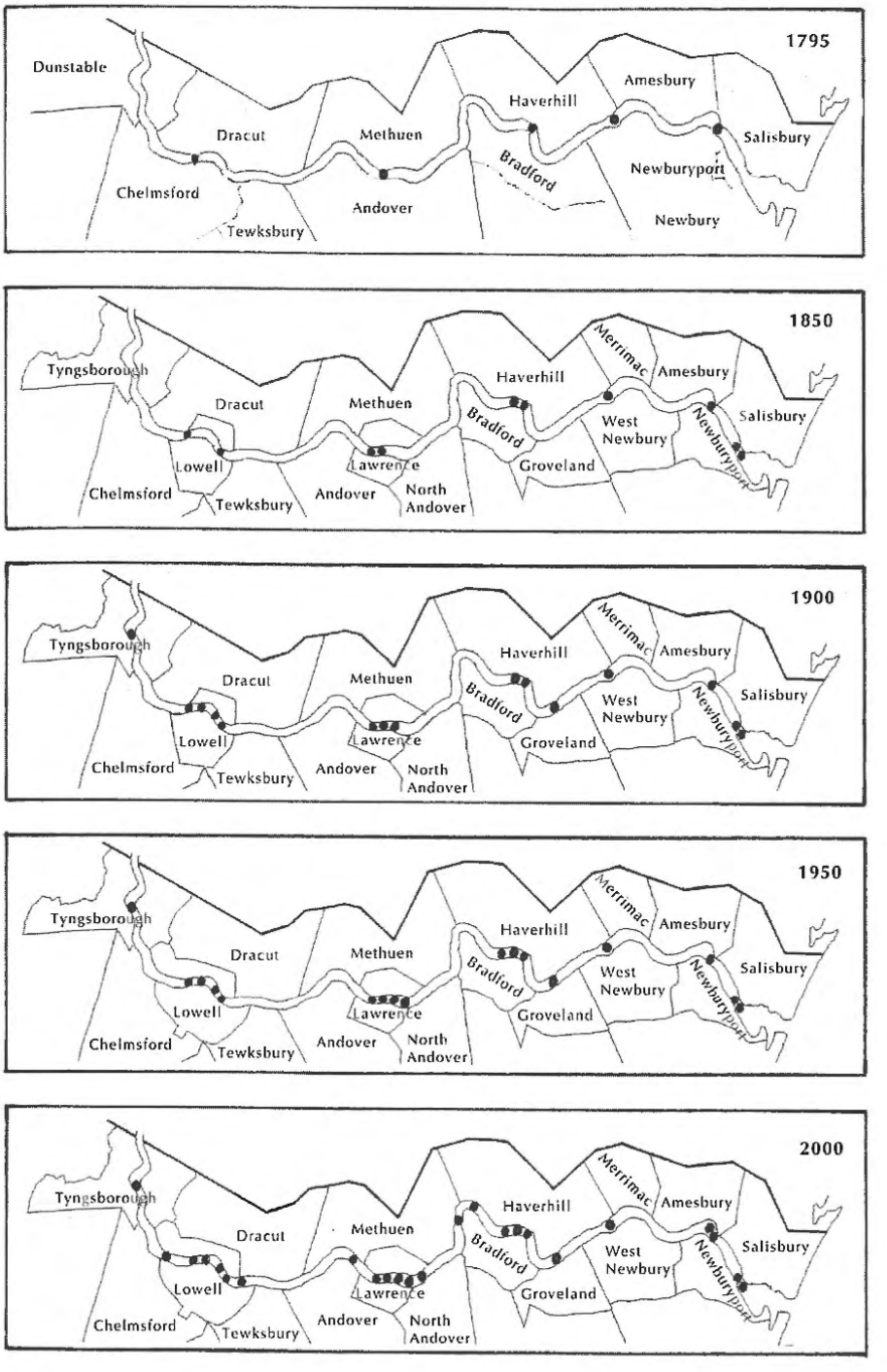
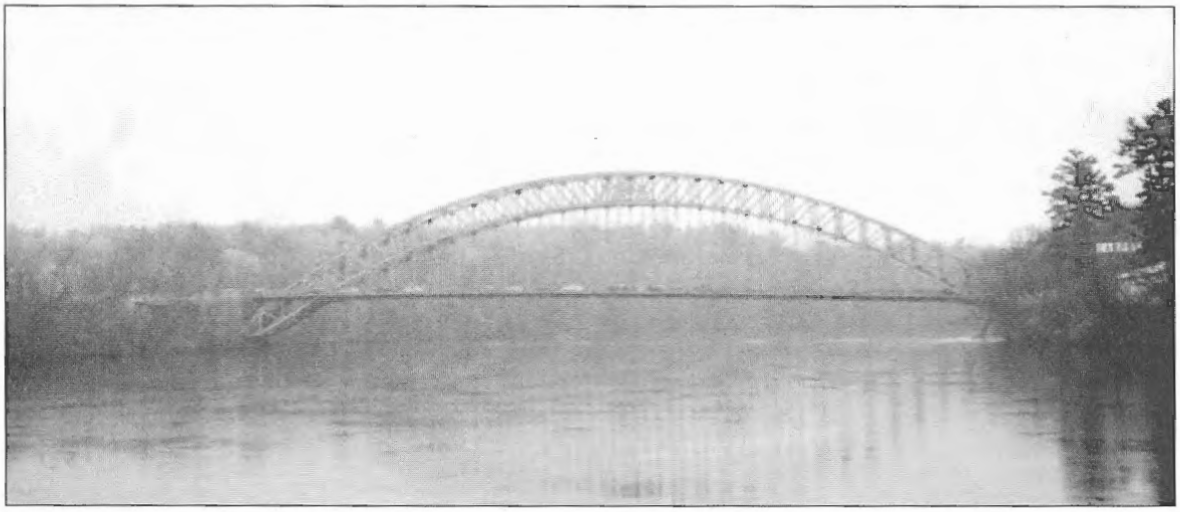


FIGURE 1. Chronology of bridge locations on the Lower Merrimac River.

of engineering technology over the past two centuries. Although most have undergone repairs and rehabilitation over the years, nearly all retain their original appearance, and all but one remain in active use. Figure 1 presents a chronology of bridge locations on the river.



Tyngsborough Bridge, Route 113, Tyngsborough, 1930

For nearly 150 years, beginning in 1728, the only way to cross the Merrimack River between Nashua and Lowell was by ferry. In 1871, the Massachusetts State Legislature authorized the Middlesex County Commissioners to lay out a highway and build a bridge across the Merrimack at Tyngsborough. The highway was constructed in 1872, and a four-span, iron Whipple truss was erected at this location in 1873. That bridge carried heavy vehicular and pedestrian traffic for 57 years until the County Commissioners declared the aging structure unsafe in 1929. In the following year, the Massachusetts Department of Public Works constructed the present 547-foot steel trussed-rib through arch. The bridge was designed and erected as a statically determinate three-hinged steel arch, and made more rigid upon completion by fixing the crown hinge. This design became a popular one for highway bridges around the turn of the century.

The history of metal arch bridges began, however, in 1779, when the world's first cast iron arch was erected near Coalbrookdale, England. A half-century later, the metal arch was tested on a modest scale in the United States, with Captain Richard Delafield's (1798-1873) Dunlap's Creek Bridge (1839) on the National Road at Brownsville, Pennsylvania, and General Montgomery Meigs's (1816-1892) Rock Creek aqueduct-bridge (1858) in Washington, D.C. Although both structures were successful, metal arch bridges remained a novelty, in part because of the difficulty of analyzing stresses in both fixed and two-hinged arches. The development of the three-hinged arch in 1870, along with the introduction of structural steel and its pioneering use in James Eads's (1820-1887) magnificent arch bridge (1874) across the Mississippi at St. Louis, stimulated the metal arch bridge's rise in popularity in the late nineteenth century. After the beginning of the twentieth century, this graceful and compelling arch form was often chosen for both its structural qualities and aesthetic appeal, particularly for long-span bridges at highly visible locations.

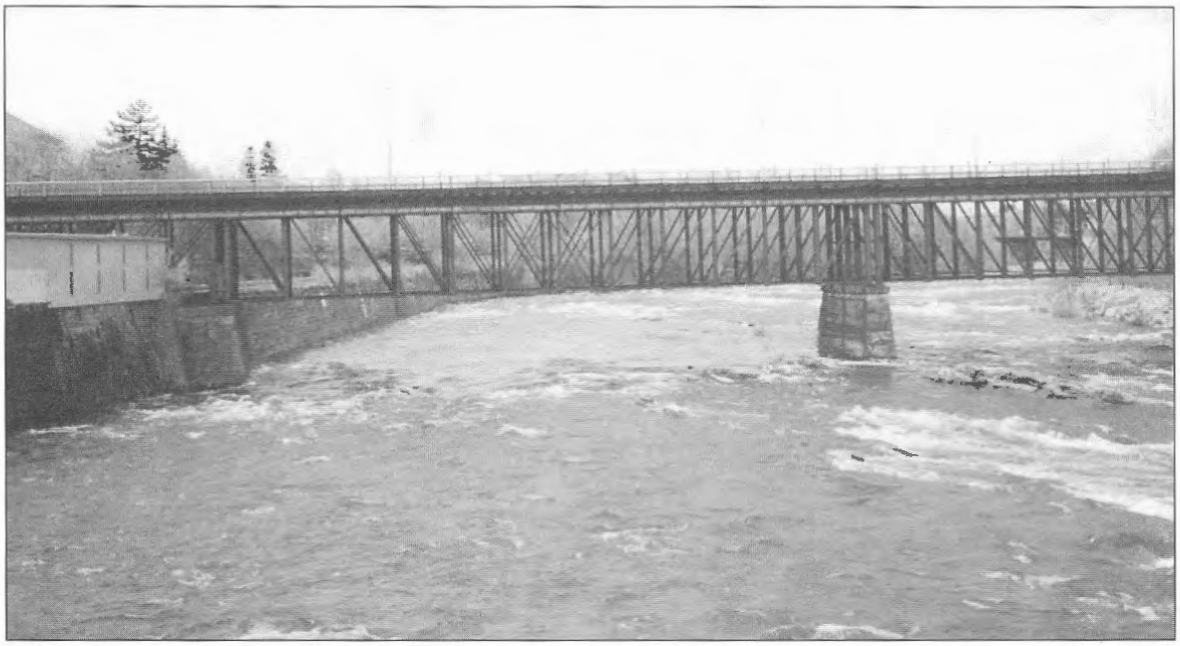
The Tyngsborough Bridge was contemporaneous with the great steel arch bridges at Bayonne (1931), Sydney (1932) and Pittsburgh (1932), but its design and scale most closely resemble the 540-foot Bellows Falls Arch Bridge (1905-1982), which was the longest steel through arch bridge in the United States when it was completed. Nationally renowned structural engineer J.R. Worcester (1860-1943) designed the latter bridge and was the consulting engineer who ultimately recommended replacement of the old Whipple truss structure at Tyngsborough. The Tyngsborough Bridge is the only single-span bridge on the Lower Merrimack, and has the fifth-longest main span of all bridges listed in the MassHighway statewide bridge database. Plans are currently being studied for the rehabilitation or replacement of this structure to accommodate an increasing flow of highway traffic to and from Route 3.

Rourke Bridge, Wood Street Extension, Lowell, 1982

The Massachusetts Department of Public Works erected this modular-panel steel bridge in 1982 to relieve traffic congestion along the Route 3 corridor at Lowell. The seven-span, two-lane temporary bridge was intended to remain in place for six years while a permanent four-lane bridge was designed and constructed, but the project has been repeatedly delayed by political and budgetary concerns.

Since its construction, the bridge has been heavily used by both vehicles and pedestrians. A sidewalk was added in 1986 and the deck was replaced in 1996. This type of structure, comprised of standardized 5- by 10-foot galvanized steel truss panels, is a modern version of the World War II "Bailey Bridge," introduced in 1941 by British engineer Sir Donald Bailey (1901-1985). Two hundred miles of temporary bridging were built by Allied forces during the war to move troops and equipment, and it has been said that General Dwight D. Eisenhower considered the Bailey Bridge one of the three most important technological developments of the war effort, along with radar and the heavy bomber.

Since World War II, modular panel bridges have become commonplace for both military and civilian applications, particularly for emergency installations or for temporarily detouring traffic while permanent bridges are undergoing repairs or reconstruction. They can be shipped in sections, assembled rapidly, require little maintenance and are reusable. In addition, the span length and carrying capacity requirements for a given span can be easily adjusted by adding or subtracting panels. While sometimes derided as resembling a "giant erector set," panel bridges are intended to be efficient, cost effective and temporary — thus, aesthetic considerations are incidental to their design. Under Chapter 204 of the Acts of 1982, this bridge was named for former Assistant Secretary of Transportation Raymond F. Rourke, and his late son Rep. Timothy M. Rourke, "in recognition of the numerous and outstanding contributions [they made] made to the Commonwealth and the Lowell Area."



Moody Street Bridge (Textile Avenue Bridge), Textile Avenue, Lowell, 1896

This three-span, pin-connected, steel Pratt deck truss was erected by the City of Lowell in 1896, when Moody Street was extended across the river from the city's thriving industrial center to the growing residential neighborhood of Pawtucketville. The bridge was designed by consulting engineer John E. Cheney (1847-1906), who is best known for his career as a bridge engineer for the City of Boston from 1885 to 1906. The deck truss configuration was likely chosen for this site because of its ability to support an exceptionally wide deck without requiring a substantial increase in the size of the floor beams and lateral bracing. This bridge was designed to carry two vehicular lanes, two pedestrian sidewalks and a street railway line.

Although the trusses bear evidence of periodic repairs, the Moody Street Bridge retains much of its structural integrity. However, its most distinguishing aesthetic feature — ornate wrought-iron railings — have been replaced with modern guardrails. According to the MassHighway Historic Bridge Inventory, the Moody Street Bridge is the oldest of five known Pratt deck truss highway bridges in Massachusetts, and also contains two of the longest (180 feet) nineteenth-century metal truss spans in the state. The Pratt truss was patented in 1844 by railroad engineer Thomas W. Pratt (1812-1875) and featured vertical wooden compression members and diagonal iron tension members.

Developed at a time when railroads were placing new demands on bridges and the structural action of trusses was just beginning to be understood, the Pratt truss (along with the Howe and Whipple trusses) heralded the transformation from empirical to scientific bridge design. While the type was not immediately popular, it became so after the Civil War, when, along with other truss types, it was built with all-iron members. The Pratt truss was favored for its straightforward design, strength and adaptability, and by 1870 it had become the standard American truss for moderate spans on both railroads and highways, and remained so well into the twentieth century.

One of the largest and most famous Pratt deck truss bridges is George S. Morison's (1842-1903) Portage Viaduct (1875) across the Genesee River at Portageville, New York. The outstanding qualities of the basic configuration spawned many modifications and subtypes, including the Pennsylvania, Parker, Kellogg, Baltimore, Lenticular and Camelback trusses. At least one-third of the bridges presently spanning the Lower Merrimack utilize some variation of the Pratt truss in their design.

Aiken Street Bridge (Joseph R. Ouellette Bridge), Aiken Street, Lowell, 1883

In 1883, the City of Lowell erected the Aiken Street Bridge to provide a second Merrimack River crossing (the other being Central Bridge) near the city center. This five-span, pin-connected, wrought iron, lenticular through truss bridge features a polygonal upper chord whose outward thrust is perfectly balanced by the inner pull of the lower tension chord. The distinctive double-convex curvilinear (lens-shaped) profile has precedents dating back at least to the early nineteenth century, when British railroad pioneer George Stephenson (1781-1848) built the Gaunless Railway Bridge (1824) with four wrought iron lenticular trussed girders. The form was further developed in the 1840s and 1850s in France, Germany and Britain, where it was used for a variety of innovative spans, including Isambard Kingdom Brunel's (1806-1859) impressive Royal Albert Bridge (1859) at Saltash.

In the United States, patents were granted for lenticular trusses in 1851 and 1855, but the form did not become popular until 1878, when William O. Douglas (b. 1841), of Binghamton, New York, patented an "elliptical bridge truss," and became associated with the Corrugated Metal Company (later the Berlin Iron Bridge Company) of East Berlin, Connecticut. After gaining the exclusive rights to Douglas's patent, the company aggressively marketed the design and by the 1890s had fabricated and erected hundreds of lenticular bridges in the United States. The Aiken Street Bridge is one of about 50 of the company's bridges that have survived to the present.

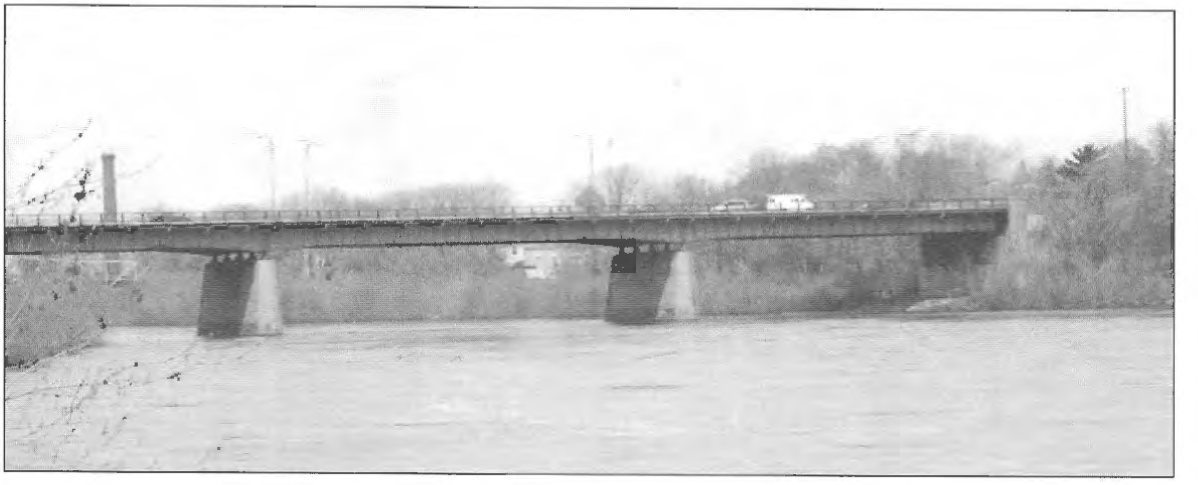
One of the most famous lenticular bridges, Gustav Lindenthal's (1850-1895) Smithfield Street Bridge (1883) at Pittsburgh, was based on the German "Pauli" truss, and is the only known example of the lenticular type *not* built by the Berlin Iron Bridge Company. The Aiken Street Bridge and the Smithfield Street Bridge, both erected in 1883, are among the oldest surviving lenticular truss bridges in the United States. The Aiken Street Bridge has the distinction of being the longest of this type and the only example having more than three spans. In 1954, the Aiken Street Bridge was named in memory of Joseph R. Ouellette, a Medal of Honor recipient killed in the Korean War. The Massachusetts Highway Department rehabilitated the bridge in 1998.

Central Bridge (John E. Cox Memorial Bridge), Bridge Street, Lowell, 1937

Historically known as Bradley's Ferry, this site has been utilized as a major river crossing since the early eighteenth century. In 1825, shortly after the founding of Lowell, the Massachusetts State Legislature authorized the Proprietors of the Central Bridge to erect a toll bridge between the Merrimack Company's newly established textile mills and the growing village of Centralville. In 1826, Luke S. Rand, a Vermont contractor working on the construction of the mill complex, built the four-span wooden truss structure known as Central Bridge. After being rebuilt in 1843, covered in 1849 and rebuilt again in 1862, the bridge was destroyed by fire in 1882. Its successor, a three-span iron Whipple truss, carried traffic for over half a century, until it, too, met with disaster during the floods of 1936.

The present 474-foot steel cantilever through truss bridge was erected in 1937. It is the oldest of only a few cantilever through truss bridges in Massachusetts, one other being the Tobin Memorial Bridge (1949) in Boston. Cantilever bridges consist of a pair of anchor arms extending from the abutments to the piers, which counterbalance a pair of cantilever arms extending from the piers over the remaining void to support a simple, suspended span. This type of bridge is advantageous for long spans because of the material savings inherent to continuous structures, and because it can be built out from both ends without erecting falsework.

The cantilever design was used for rudimentary wooden plank bridges in ancient China, but was not utilized for major spans until the mid-nineteenth century, when Heinrich Gerber constructed the first modern cantilever bridge, the Hassfurt Bridge (1867), over the Main River in Germany. A decade later, in 1878, Charles Shaler Smith (1836-1886) erected the first cantilever truss bridge in the United States, across the Kentucky River at Dixville, Kentucky. In the late nineteenth century, the potential of the cantilever form was demonstrated with the construction of the immense Forth Rail Bridge (1890), after which the type became popular for railroad bridges, largely because of its rigidity under moving loads. The subsequent construction disasters involving the Quebec Bridge (1907/1914) curtailed this enthusiasm, but also prompted a period of scientific structural analysis and re-evaluation of engineering practices from which the cantilever form emerged with renewed acceptance. The type came into common use for both railroad and highway spans in the United States around the 1930s, when many trusses of this type were erected, including spans of the Carquinez Strait Bridge (1927), Pulaski Skyway (1932) and Oakland Bay Bridge (1936). Under Chapter 586 of the Acts of 1985, Central Bridge was renamed in memory of former Lowell City Councillor John E. Cox.



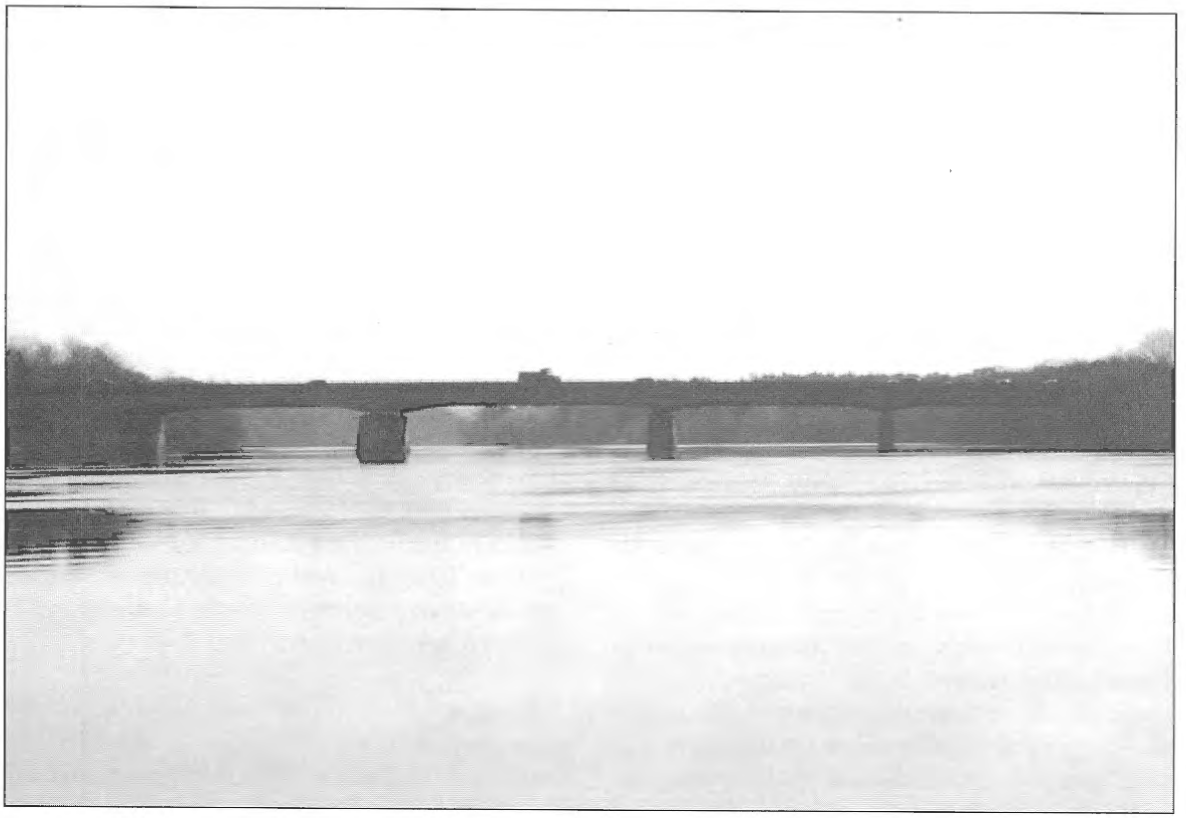
Hunts Falls Bridge (Quinn-Holmes Bridge), Route 38, Lowell, 1952

The Hunts Falls Bridge was designed and constructed by the Massachusetts Department of Public Works in 1952 when Route 38 was developed as part of the state highway system. Prior to that date, the nearest crossing of the Merrimack was at Central Bridge, access to which required driving through the congestion of downtown Lowell. The 450-foot long structure consists of three continuous spans of variable-depth, riveted steel girders.

Girder bridges are the simplest type of bridge, based on the ancient principle of the trabeated structural system (*i.e.*, a horizontal beam supported at both ends). Primitive types of girder bridges, such as a tree trunk or stone slab laid across a stream, have been used since the beginning of time, but metal girder bridges were unknown until the nineteenth century. Beginning in 1784 in England, wrought iron beams and rails were produced by rolling, but for many decades the cross-sectional size of beams that could be produced by this process was modest and not adequate for bridge-building. The earliest prototypes for metal girder bridges were built in the 1840s, using the same techniques employed in the construction of locomotives and steam boilers.

In 1845, Robert Stephenson (1803-1859) and William Fairborn (1789-1874) designed a unique, wrought iron box girder for the Britannia Bridge (1850), and at nearly the same time, American railroad mechanic James Milholland (b. 1812) constructed the first plate girder bridge (1847) for the Baltimore & Susquehanna Railroad at Bolton Station, Maryland. These early designs developed out of the mid-nineteenth-century tradition of riveting plates and angles together to create larger structural members capable of spanning greater distances than could be produced by the rolling process alone. By the end of the nineteenth century, girders were still considered uneconomical for long spans because they required more structural material than truss bridges of comparable length, but their popularity grew as their lower fabrication, erection and maintenance costs became evident.

At the end of the nineteenth century, continuous girders, such as those used in the Hunts Falls Bridge, were developed. Continuous girders extend over more than two supports, thereby allowing loads on one span to be resisted, in part, by adjacent spans. This redistribution of stresses results in lower maximum bending moments compared with simple spans, which saves material by allowing the use of lighter girder sections, or by allowing the same size sections to extend over a greater span. The haunches supply a greater web depth over the piers, where reverse or negative bending forces occur and, in addition, introduce an aesthetically pleasing curvilinear form to an otherwise rectilinear design. Riveted girders were a popular type for railroad bridges beginning in the late nineteenth century, and became popular for highway bridges in the 1930s. Under Chapter 348 of the Acts of 1960, the Hunts Falls Bridge was renamed in honor of World War II veterans T.J. Quinn and Richard Holmes.



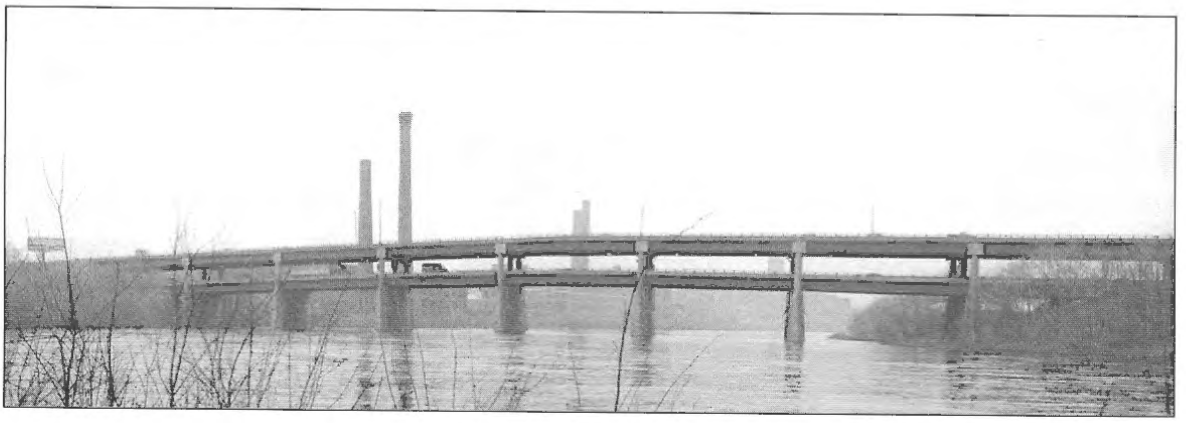
Interstate 93 Bridge (General Edward D. Sirois Bridge), I-93, Andover-Methuen, 1959/1971

This five-span, riveted steel, deck girder bridge was erected by the Massachusetts Department of Public Works in 1959 as part of the construction of Interstate 93. In 1971, the bridge was widened from four lanes to eight lanes by the addition of two lines of continuous welded girders on either side. This combination of riveted and welded construction reflects a transition in metal member fabrication that occurred slowly but steadily after World War II as the process of arc welding was refined.

In the nineteenth century, fusing two pieces of metal together by the hammering (or forge welding) method was an awkward, time-consuming process requiring the use of a forge, and was, thus, unsuited to the task of fabricating large structural members. The arc welding process, initially employed for shipbuilding, uses intense heat from a high-amperage electrical arc to create a strong metal bond quickly, thereby making fabrication of structural members more economical by reducing labor and eliminating much of the material waste associated with riveting.

Welded girders, which weigh about 15 percent less than riveted girders of the same capacity, were commonly used in buildings after World War II, but their use in bridges was delayed by a lack of information about the impact resistance of welded metal. When experiments proved that welded girders were as reliable as their riveted counterparts, welded girders became an attractive option for the hundreds of bridges required in the construction of public highways from the late 1950s on.

For mid-twentieth-century highway officials, an added attraction of welded girder bridges was their efficient, streamlined appearance — a look that was in keeping with the "modern" design aesthetic of the post-war era. Today, both riveted and welded girder bridges are commonplace, but often unnoticed features on highways across the nation. Under Chapter 784 of the Acts of 1968, the Interstate 93 Bridge was named in memory of General Edward D. Sirois, a former member of the Massachusetts General Court and vetern of three wars.



Interstate 495 Bridge (Rev. James T. O'Reilly Bridge), I-495, Lawrence, 1962

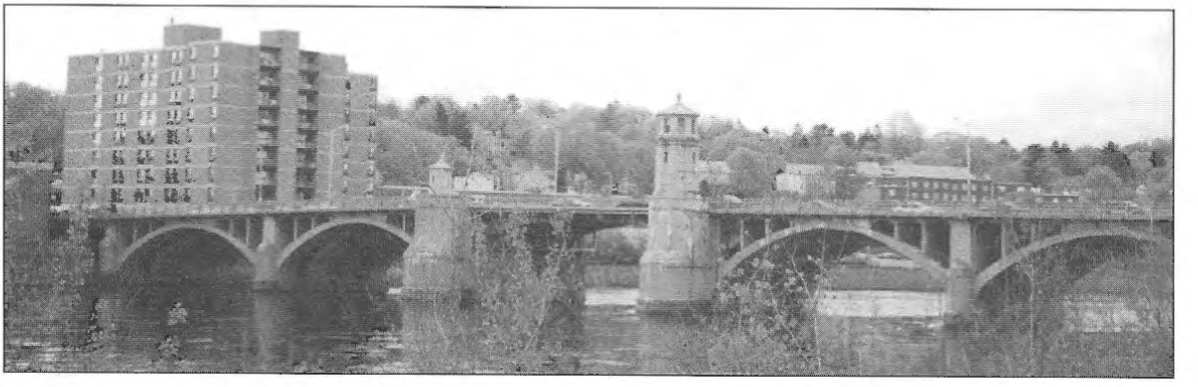
In 1944, the federal government initiated the National System of Interstate and Defense Highways to create an integrated network of 41,000 miles of highways connecting the major metropolitan regions in the United States. This program, described by President Eisenhower as, "the greatest public works program in history," was launched full-scale in 1956, when the Federal Aid Highway Act authorized 41 billion dollars in funding over a period of thirteen years for interstate highway construction. Under this program, Interstate 495 was designed as an 88-mile controlled access circumferential superhighway forming an "outer beltway" around metropolitan Boston and through the Merrimack River Valley, a region of Massachusetts seen as having potential for industrial and economic growth. At Lawrence, where a number of existing routes would converge with the interstate both north and south of the Merrimack River, highway engineers designed this 1,200-foot, eight-span, double-deck, welded steel girder bridge in accordance with federally prescribed standards, to promote safe and efficient traffic flow patterns in an area of anticipated heavy traffic congestion. The bridge was described in the *Master Plan of Highways for Lowell-Lawrence-Haverhill*, as follows:

> "The new Merrimack River Bridge will have two highway decks; the upper one for through traffic and the lower one for traffic which will utilize either one or both of the adjacent traffic interchanges for local distribution. This plan will provide needed local traffic service between North Andover and areas of northern Lawrence."

Bridges with multiple levels have been built since ancient times. For example, the Pont du Gard (ca. 14 A.D.) in southern France carried an aqueduct on one level and a roadway on another level; the Newburyport Railroad Bridge (1840-1865) had separate levels for horse-drawn carriages and steam-powered trains; and the Brooklyn Bridge (1883) has a pedestrian walkway above the deck carrying automobile traffic.

The first modern double-deck highway bridge was built on Wacker Drive in Chicago in 1925, and the design increased in popularity during the mid-twentieth century as numerous expressways were built in metropolitan areas across the United States. Today, double-deck highway bridges are frequently encountered in urban settings where there is a large volume of high-speed traffic and many interchanges occur in a confined space with limited right-of-way.

Under Chapter 379 of the Acts of 1963, the Interstate 495 Bridge was named in memory of the Reverend James T. O'Reilly, who served as pastor of Saint Mary's Catholic Church in Lawrence from 1886 to 1925, in remembrance of his public spiritedness and service to the community.



Haverhill Bridge (Lower County Bridge/Pfc. Ralph T. Basiliere Bridge), Main Street, Haverhill, 1925

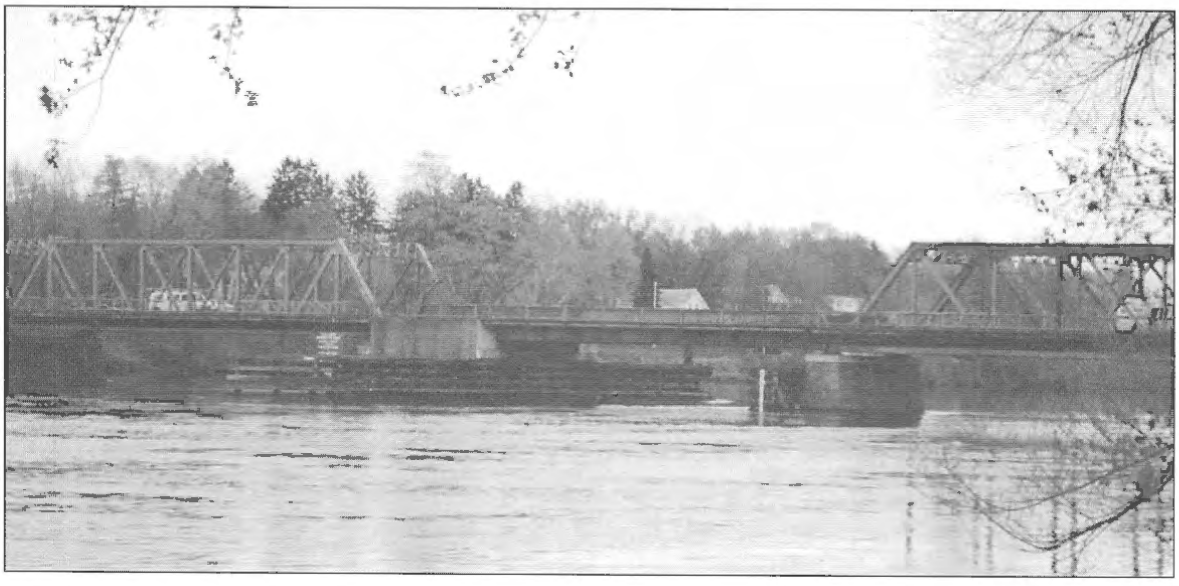
In 1793, the Proprietors of the Haverhill Bridge Company erected a wooden toll bridge at this site. The bridge was a three-span arched timber truss structure, designed by local builder Timothy Palmer, who had recently completed the first bridge across the Merrimack at Newburyport. After being rebuilt in 1808 and covered in 1825, the Haverhill Bridge carried traffic for half a century, until it was replaced with an iron Whipple truss structure in 1873.

The present six-span, open-spandrel, reinforced concrete deck arch bridge was erected to replace the iron bridge in 1925. This bridge was designed by George F. Swain (1857-1932), a civil engineer of national renown who consulted on the construction of a number of important Massachusetts bridges (including several on the Merrimack). The Strauss Bascule Bridge Company designed the bridge's trunnion bascule draw span, but the operating machinery was never installed, as plans to make the Merrimack navigable to Lowell were never implemented.

The Haverhill Bridge is one of two reinforced concrete open-spandrel arch bridges on the Merrimack, the other being the Central Bridge (1918) at Lawrence. (A third reinforced concrete open-spandrel arch bridge spanned the Merrimack at Pawtucket Falls in Lowell from 1916 to 1968.) Although similar in design, the Haverhill Bridge is more aesthetically successful in that its bascule span was an integral part of the overall design, while the proportions of the Central Bridge were compromised when it was redesigned during construction to accommodate a proposed bascule span that was later eliminated from the design.

According to the MassHighway Historic Bridge Inventory, the Haverhill Bridge is one of only six open-spandrel rib arch bridges in the state, and it is noteworthy for its unusually ornate neo-classical details. The practice of combining the compressive strength of concrete with the tensile strength of metal reinforcing began the 1860s with experiments by French gardener Joseph Monier (1823-1906), who used iron netting to reinforce flower pots and planters. In the 1870s and 1880s, others extended these experiments to larger structures such as beams, slabs and arches. The first reinforced concrete arch bridge in the United States, the Alvord Lake Bridge in Golden Gate Park, was built in 1889 by Ernest Ransome (1844-1917), who patented the twisted-bar reinforcing system that became the basis for most common systems in use today. Ransome's structure emulated the traditional design of masonry bridges through its use of the solid-barrel arch form and surface treatments that purposely imitated the appearance of stone.

As engineers gained experience with the structural properties of reinforced concrete in the early twentieth century, they sought to use the material more efficiently. By relying on the tensile strength of the reinforcing steel, instead of the compressive strength of a ponderous mass of concrete alone, they were able to attenuate the arch ribs and open up the spandrel walls, thereby creating increasingly lighter structures of more elegant proportions. Under Chapter 231 of the Acts of 1972, the Haverhill Bridge was renamed in honor of U.S. Marine Pfc. Ralph T. Basiliere, Haverhill's first casualty of the Vietnam War.



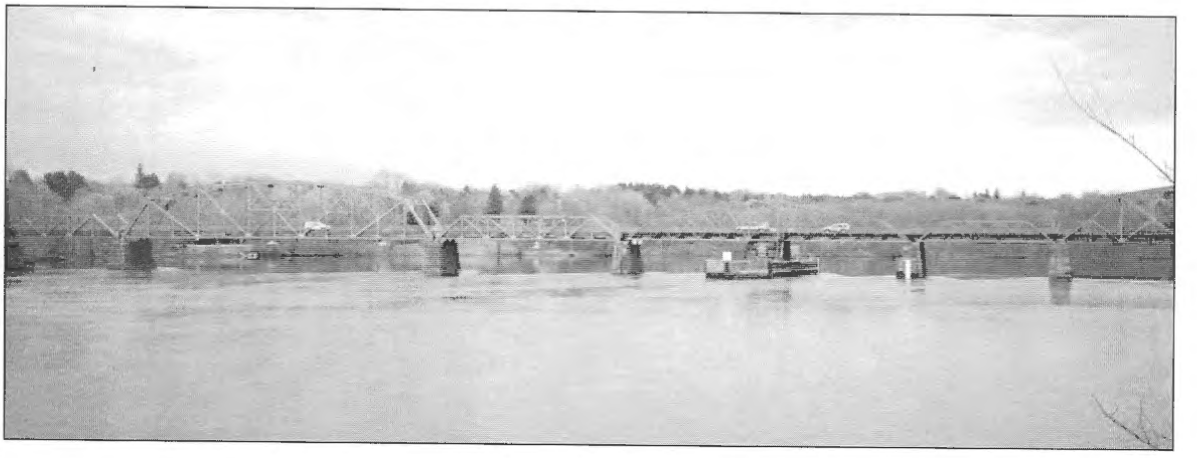
Groveland Bridge (Congressman William H. Bates Bridge), Groveland Street-Main Street, Haverhill-Groveland, 1913/1950

A chain ferry operated at this location for nearly 140 years until the first bridge was built here in 1872. It was a five-span, wrought iron, tubular arch bridge, patented by Zenas King (1818-1892) in 1861. Proclaimed "a perfect model of symmetry and beauty," this bridge carried traffic for ten years until it collapsed under the weight of snow and an overloaded lumber wagon. The damaged structure was replaced in 1882 with an iron structure consisting of five Warren through truss spans and a central swing span.

In 1913, the southern spans of the bridge were consumed in a fire caused by faulty street railway wiring. The three burned trusses were replaced with the present three 123-foot, riveted Pratt through truss spans at the southerly end of the bridge, and the structure was put back in service the following year. In 1950, the bridge's two northerly spans were replaced with two 140-foot, riveted Pratt through trusses (of slightly heavier construction than the southern spans), and the draw was replaced with the present bascule girder span.

A bascule is one of the three major types of movable bridges, along with swing and lift spans. The term *bascule* comes from the French word meaning *see-saw* since the principle of counter-balance is fundamental to its design. A bascule bridge has one or two counterbalanced leaves that pivot about a horizontal axis at one end and lift at the other end to provide a clear passage for boats. Unlike swing bridges, which pivot on a central pier, and lift bridges, which are raised and lowered horizontally between vertical towers, bascule bridges can be rapidly opened and closed, allow unlimited vertical clearance in the open position and do not restrict the navigation channel. The bascule concept was originally used in medieval times for drawbridges over castle moats and, in the sixteenth century, Leonardo da Vinci (1452-1519) made sketches of a rudimentary bascule bridge with a counterweight. However, the modern bascule bridge was not fully developed until 1890 when an efficient method of counterbalancing the weight of the span had been found and electric motor technology refined.

The first modern bascule bridges were the Van Buren Street Bridge (1893) in Chicago and the Tower Bridge (1894) in London. Bascule bridges are now the most common type of movable highway bridge found in Massachusetts. Twenty-nine examples of this type have been identified among the 44 movable bridges listed in the MassHighway statewide bridge database. The Groveland Bridge contains one of two operable bascule spans on the Lower Merrimack. Under Chapter 621 of the Acts of 1970, the Groveland Bridge was named in memory of William H. Bates, a local congressional representative.



Merrimac Bridge (Rocks Bridge), Haverhill-West Newbury, 1883/1895/1914

In 1794, the Massachusetts General Court authorized the Proprietors of the Merrimac Bridge to construct a toll bridge at Rocks Village in Haverhill. Completed in 1795, it was the fifth bridge to span the Merrimack River, replacing a ferry system that had been in operation since the late seventeenth century. The bridge had pile and beam approaches, a 30-foot draw and a 140-foot arched timber truss, based on Timothy Palmer's design.

The bridge was washed away in a freshet in 1818, and rebuilt on the old foundations ten years later as a 900-foot, four-span covered wooden Town lattice truss (patented in 1820 by Connecticut architect Ithiel Town [1784-1844]) with a central drawspan, which held the record as the longest covered bridge in New England for many years. With the passage of time, sections of the bridge were replaced with new spans of various types and sizes, resulting in the somewhat irregular appearance of the present bridge. The draw was replaced in 1873 and again in 1883, when the present iron swing span and adjacent pony truss span were erected. In 1895, the wooden span at the Haverhill end was replaced with a Pennsylvania through truss designed by Edward S. Shaw.

The bridge reached its present 812-foot, six-span configuration in 1914, when the Essex County Commissioners replaced the remaining wooden spans east of the draw. Initial plans to replace the wooden trusses with reinforced concrete arches were eventually discarded in favor of the present three steel Pratt trusses designed by George F. Swain (1857-1932). The trusses were adopted as a cost-saving measure in order to utilize some of the old piers and resulted in spans of unequal length and different design, which fortunately provided visual balance to the truss configuration at the Haverhill end, and were, thus, in the County Engineer's estimation "not as objectionable as might be expected."

Of the bridge's six spans, the rim-bearing swing span is the most technologically significant since it is the oldest movable highway span in Massachusetts. Swing bridges, which rotate about a vertical axis, were first built in the United States in the early nineteenth century across the Erie Canal and eventually became the dominant type of movable bridge by the end of the century. The Rock Island Bridge (1896) across the Mississippi and the Center Street Bridge (1901) at Cleveland are two famous examples.

The Rocks Bridge is one of two operable swing bridges on the Merrimack, the other being the Deer Island Drawbridge. Comparison of the two operable swing spans reflects the changes in swing bridge construction that occurred around the turn of the century: from rim-bearing to center-bearing systems; from through truss to deck plate girder superstructure; and, from manual to electrical operation. According to the MassHighway Historic Bridge Inventory, the Rocks Bridge swing span is one of the oldest riveted metal trusses in the state and the earliest surviving work of the Boston Bridge Works, one of the largest independent bridge manufacturers in New England from the 1870s through the 1930s whose work is well-represented on the Lower Merrimack.

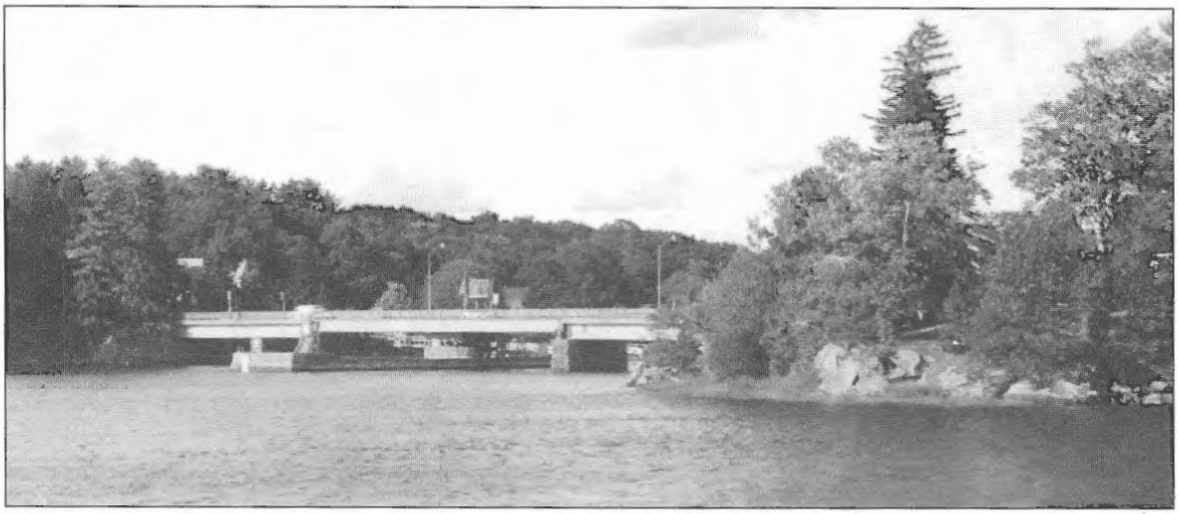
John Greenleaf Whittier Bridge, I-95, Amesbury-Newburyport, 1952

This 1,346-foot, three-span, riveted steel, continuous through truss bridge was completed in 1952 as part of the construction of Interstate 95, a limited-access route that parallels Route 1 from Maine to Florida. The bridge was designed by Massachusetts Department of Public Works engineers. Although the main span of this bridge forms an arch over the navigation channel, the bridge's structural action is that of a continuous truss rather than an arch. The continuous truss acts as a unit over more than two supports. Therefore, stresses from live loads are transferred to members in adjacent spans. The advantages of a continuous bridge over a simple span bridge are economy of material, convenience of erection and increased rigidity under traffic.

Continuous bridges were built in England as early as the 1850s, and in the United States in the 1870s, but because continuous bridges are subject to secondary stresses from pier settlement, this type of structure was initially used only for short spans. After the beginning of the twentieth century, longer spans of this type were gradually introduced, first for railroads in the 1910s, and then for highways in the 1920s. The first major bridge of this type in the United States was Gustav Lindenthal's 775-foot Sciotoville Bridge (1917) spanning the Ohio River at Fullerton, Kentucky.

The continuous through truss type came into common use in the 1930s, when monumental spans were erected, including the Bourne (1934) and Sagamore (1935) bridges over the Cape Cod Canal. These nearly identical bridges each received a design award from the American Institute of Steel Construction.

According to the MassHighway Historic Bridge Inventory, the John Greenleaf Whittier Bridge is essentially a half-scale, double-barrel version of the Bourne and Sagamore bridges, but is somewhat less successful aesthetically due to the elongated approach spans. Under Chapter 460 of the Acts of 1953, the bridge was named in honor of distinguished Haverhill native John Greenleaf Whittier (1807-1892), whose essays and poetry include many references to the Merrimack River and its bridges.



Essex-Merrimac Bridge, Northern Span (Deer Island Drawbridge), Main Street, Amesbury, 1966

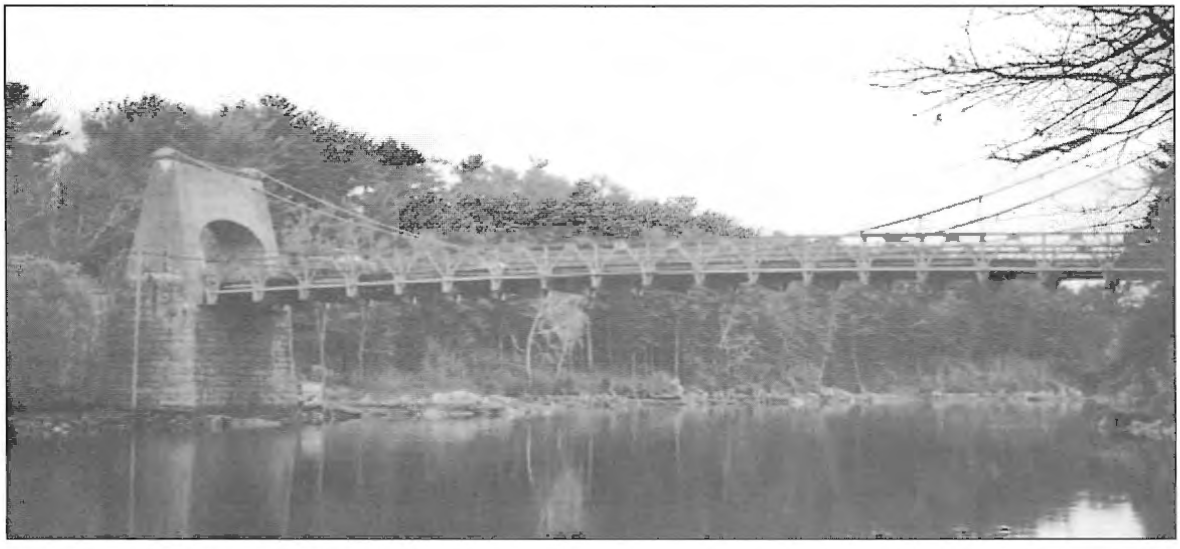
At Deer Island, the Merrimack is spanned by separate bridges over the north and south river channels: a swing bridge connecting the island to Amesbury on the north side of the river; and a suspension bridge connecting the island to Newburyport on the south side of the river. This place was the site of the first bridge across the Lower Merrimack, erected in 1792 by Timothy Palmer (1751-1821), a builder from Newburyport. The bridge approaches were simple pile and beam structures with a draw over the northern channel, while the main spans (measuring 113 feet and 160 feet) were arched timber trusses, which resembled (or may have been directly adapted from) the sixteenth-century designs of the Italian architect Andrea Palladio (1518-1580), who was the first to illustrate and build a truss bridge.

The timber truss covered bridge was developed in Europe by Swiss carpenter Hans Ulrich Grubenmann (1709-1783), whose most famous span was the arched timber Schaffhausen Bridge (1757) across the Rhine, but wooden bridges were not commonly built until the end of the eighteenth century, when the idea found its way to America. Several contemporary references suggest that a 1764 "geometry work" bridge at Norwich, Connecticut, may have been the first use of the truss principle in American bridge design, but Palmer's Essex-Merrimac Bridge is generally considered the first long-span timber truss bridge in the United States.

Palmer's most famous bridge, the Permanent Bridge (1806) over the Schuylkill River in Philadelphia, was his longest (194 feet) arched timber truss span to that date and one of the first covered bridges in America. The preservation measure of roofing and weatherboarding wooden trusses was so effective that it soon became conventional practice, and was applied to all wooden bridges on the Merrimack by the 1820s.

Palmer's successors, Louis Wernwag (1769-1843) and Theodore Burr (1771-1822), continued to use empirically-designed arch-truss combinations for timber bridges, until they were eventually superceded in the 1820s and 1830s with designs of Ithiel Town (1784-1844) and Stephen Long (1784-1864), which dispensed with the arch form entirely, and thus heralded the many scientifically designed truss forms to follow.

After being covered around 1810, and possibly rebuilt in the mid-nineteenth century, the northern timber span of Palmer's Essex-Merrimac Bridge lasted until 1882, when it was replaced with an iron, three-span, double-intersection Warren through truss with a rim-bearing swing span. When that bridge was destroyed by fire in 1965, the present rolled steel girder, center-bearing swing span was erected in its place. The operating machinery was motorized in 1971. The Deer Island site is historically significant for its association with Palmer's pioneering work in timber truss bridge design.



Essex-Merrimac Bridge, Southern Span (Chain Bridge), Main Street, Amesbury-Newburyport, 1909

At the time of its construction, the Essex-Merrimac Bridge was considered a great engineering achievement, but was denounced by boatmen as a hindrance to navigation. Thus, in 1810, the timber truss spanning the southern channel was replaced with a then-novel type of structure to allow easier passage for boats. Designed by Judge James Finley (1756-1828) of Pennsylvania, the bridge featured a deck suspended from wrought iron chains that hung between wooden towers spaced 243 feet apart.

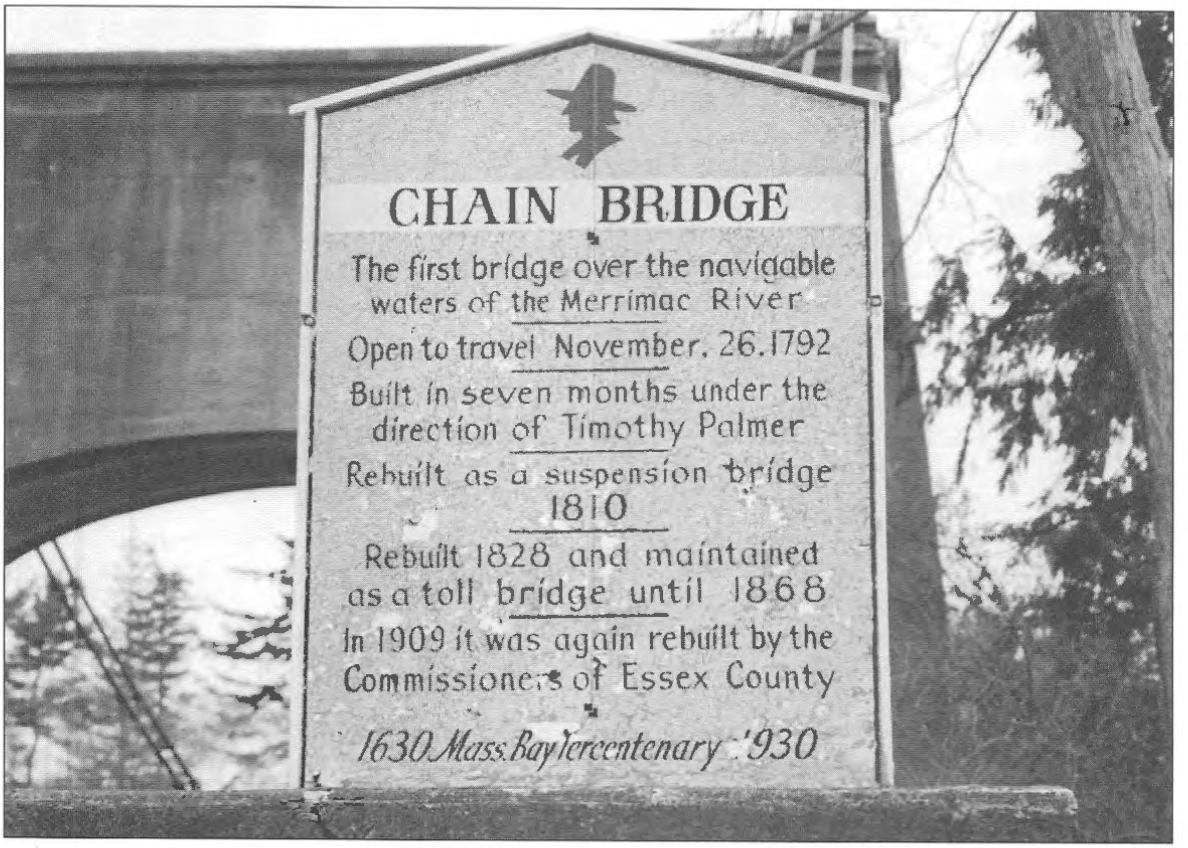
Unlike primitive suspension bridges, which deflected significantly under loads, Finley's bridge combined a catenary suspension cable of wrought iron links with a stiffening truss for the roadway, to create a suspension bridge with a rigid, level deck. In 1801, he built the first suspension bridge in America, a 70-foot span, at Uniontown, Pennsylvania, and in 1808 he was granted a patent for his design. About 40 bridges were built based on this patent. Finley's longest span was a 308-foot pedestrian bridge (1809) in Philadelphia, but he confidently asserted that much longer spans were possible, and thus foreshadowed the work of his successors:

> "There is no reasonable doubt that in some extraordinary case this kind of bridge will be extended to one thousand feet once the subject shall be fully understood . . . Let us pursue the idea of extension to 15,000 feet without any middle pier."

Descriptions of Finley's work were published in the United States and abroad, and may have influenced the design of other iron chain suspension bridges built in the next two decades, including Capt. Samuel Brown's (1776-1852) Union Bridge (1820) between England and Scotland, a five-span iron chain suspension bridge across the Merrimack at Newburyport (1826) and Thomas Telford's (1757-1834) Menai Strait Bridge in Wales (1826). The major drawback of the design was that the chains were vulnerable to manufacturing flaws and corrosion: if one link failed, the whole bridge could collapse. This problem was illustrated when the Essex-Merrimac Chain Bridge failed under a heavy load in 1827.

Although the bridge was rebuilt following Finley's original design and continued to carry traffic for 80 years, wrought iron chain bridges were losing popularity. During that period, wire cable technology — which promised greater reliability and roughly two times the tensile strength of forged bars for a given sectional area (and thus the possibility of truly long spans) — began replacing wrought iron chain designs in France and the United States.

In 1841, Colonel Charles Ellet (1810-1862) erected the first successful wire cable suspen-



Commemoration plaque on the Chain Bridge.

sion bridge across the Schuylkill in Philadelphia, and just eight years later, he built a record-breaking 1,010-foot wire cable span at Wheeling, West Virginia. Simultaneously, another pioneer in the field, John A. Roebling (1806-1869), was working on suspension aqueducts for canals. In the 1840s, he established his own wire mill and became a proponent of wire cable suspension spans. In a letter concerning the proposed suspension railroad bridge at Niagara Falls (1855) he stated:

> "It cannot be questioned that wire cables, when well made, offer the safest and most economical means for the support of heavy weights . . . There is not one good suspension bridge in Great Britain, nor will they ever succeed as long as they remain attached to their chains."

With Roebling's vision and the steady development of wire cable technology, suspension bridges grew increasingly longer. At the end of the century, Roebling's monumental, 1,595-foot Brooklyn Bridge (1883) held the world record for suspension bridge spans. While diminutive in comparison to the Brooklyn Bridge, the Chain Bridge had became a cherished historic and visual landmark by 1909, when it was turned over to the county for rebuilding. In deference to local sentiment that wanted to retain the appearance of the 80-year old structure, consulting engineer George F. Swain (1857-1932) designed the present bridge to echo the form and massing of the former structure, but utilized state-of-the-art materials, including reinforced concrete towers and steel wire cables, for strength and durability.

Although rebuilt in essentially modern form, the Chain Bridge is historically significant for its association with Finley's pioneering suspension bridge that formerly spanned this location, and is structurally significant as the only highway suspension bridge in Massachusetts. The Massachusetts Highway Department is currently developing plans to rehabilitate this picturesque historic landmark.

NOTES & ACKNOWLEDGMENTS — *The photographs and descriptions presented here were prepared as part of a project for a graduate bridge engineering course at Tufts University. Richard Kaminski took all of the photographs except for the photo of the Deer Island Drawbridge. Lola Bennett wrote the descriptions of the bridges and took the photo of the Deer Island Drawbridge. The authors wish to thank Steven J. Roper, MassHighway's Historic Bridge Specialist, for his encouragement and helpful comments.*

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Pumping Test Program for the Central Artery/Tunnel Project in Downtown Boston

A well conceived and implemented pumping test program for a large-scale excavation project offered greater protection to structures adjacent to the project area.

ABDELMADJID M. LAHLAF,
IQBAL AHMED & FRANCIS D. LEATHERS

Six full-scale pumping tests were conducted to obtain field data on the permeabilities of the soil and rock strata likely to require groundwater control during Interstate 93 (I-93) highway tunnel and ventilation building construction for the proposed Central Artery in Area Geotechnical Consultant (AGC) Area 04 (Central Area) of the Central Artery/Tunnel (CA/T) Project. The pumping tests were supplemented by extensive borehole permeability testing in the soil and rock during the AGC phase geotechnical investigations.

The results of the six pumping tests performed show that the rock is frequently more permeable than the overlying soils. Four of the tests were conducted in bedrock, with three tests indicating permeabilities in the range of 0.5 to 2×10^{-3} cm/sec and the fourth test indicating a permeability of about 5×10^{-5} cm/sec. These results are in the same range as indicated by the borehole permeability tests in rock. One pumping test was performed in glacial till and indicated a permeability of about 5×10^{-5} cm/sec, which is near the middle of the range measured by borehole permeability tests in glacial till. One pumping test was performed in glacial till and sand and gravel deposits and indicated a permeability of about 5×10^{-4} cm/sec. The radii of influence estimated from the four pumping tests in rock ranged from 350 to 2,200 feet for drawdowns in the pumping test wells that ranged from 42 to 81 feet.

Groundwater control in the deep excavations for the Central Area is expected to consist primarily of relieving piezometric pressure in the rock and glacial soils below the excavation. The pumping tests provided direct indications that during pressure relief operations high

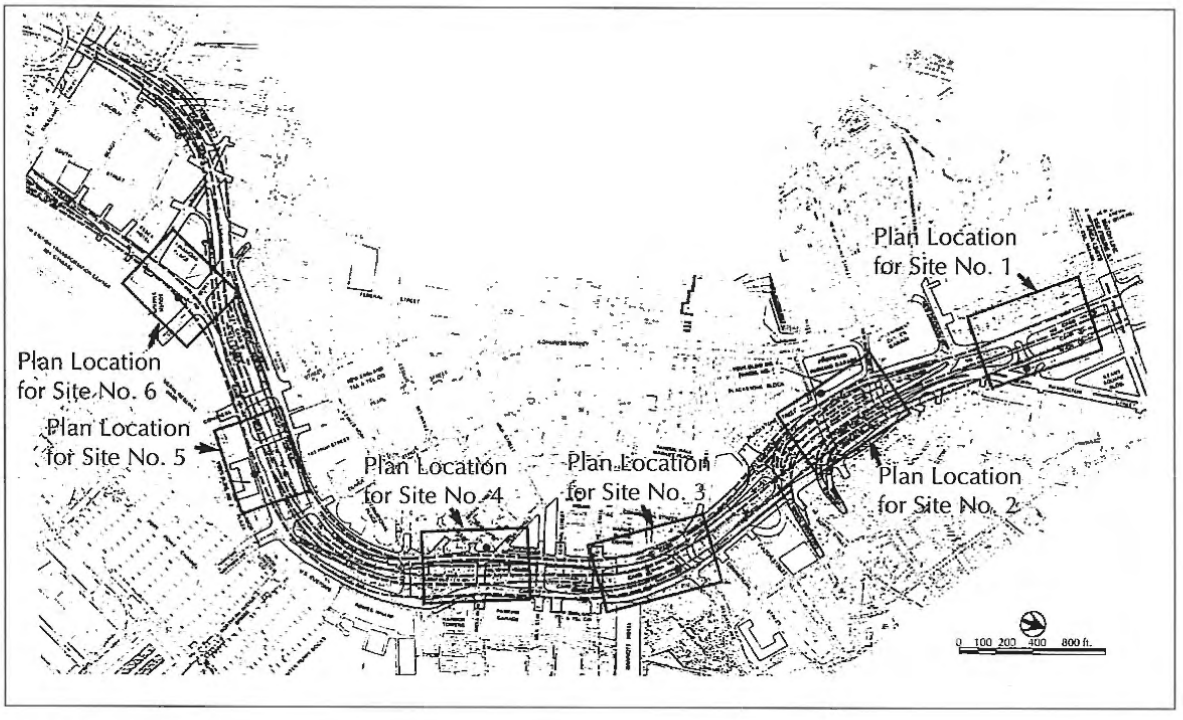

FIGURE 1. Location of the pumping test sites.

pumping rates and large zones of influence should be anticipated. Also, the borehole permeability tests indicated that large zones of permeable rock may be present below the proposed bottoms of the excavation support walls. The presence of such permeable zones could result in substantial drawdowns outside the excavation walls and, therefore, may necessitate the use of grout curtains and/or recharge wells to control drawdowns outside the excavation walls.

Introduction

The pumping test program was conducted in conjunction with a geotechnical investigation that was performed in the Central Area to determine the subsurface conditions and engineering properties of the subsoils and rock for use in the design of the proposed depressed Central Artery. The six different sites were selected to investigate a range of soil and rock conditions that are likely to require groundwater control during I-93 highway tunnel and ventilation building construction in the Central Area. Four of these tests (Tests 2, 3, 5 and 6) were performed in bedrock. Pumping Test 1 was performed in glacial till and a sand and gravel stratum. Pumping Test 4 was performed in glacial till. The results of each test reflect the specific site and test stratum conditions within the zone of influence of the particular test location.

Figure 1 presents a key plan showing the locations of the pumping test sites in the Central Area. The installations at each of the pumping test sites consisted of a pumping test well, piezometers and observation wells. Installations of test wells and instrumentation at the pumping test sites were performed between July 8 and October 15, 1991. The pumping tests were conducted between September 3 and November 6, 1991.

Project Background

The CA/T Project in the Central Area involves the construction of a widened underground I-93, most of it to be constructed beneath the existing elevated Central Artery. The Central Area project site is located along the existing Central Artery from just north of Causeway Street to just south of Kneeland Street in downtown Boston. South of Congress Street the proposed artery alignment bifurcates into the northbound tunnel that will follow Atlantic

Avenue from Kneeland Street to Congress Street and the southbound tunnel that will follow the existing Dewey Square Tunnel. The proposed artery alignment passes over the Massachusetts Bay Transportation Authority (MBTA) Blue Line subway tunnel at State Street and underneath the MBTA Red Line station and tunnel at South Station on Summer Street. The project in the Central Area also includes two ventilation buildings (Nos. 3 and 4), with the former located on Atlantic Avenue adjacent to the Russia Wharf Building on Congress Street and the latter located at Blackstone and New Chardon streets adjacent to the Haymarket area.

The topography of the site is fairly level, with the ground surface elevations typically ranging from el. 108 to 116. Areas fronting the site are occupied by typical urban buildings including low- to high-rise residential, commercial and office structures, hotels, retail establishments and parking lots. Many of the structures adjacent to the site are relatively old brick buildings. Most of these buildings are concentrated in the northern part of the Central Area. The older structures in the Central Area are typically founded on granite block footings and timber piles, while foundations for the relatively new structures range from spread footings and mat foundations, to caissons and piles. Most of these foundations are susceptible to being affected by drawdowns outside the slurry walls. Timber piles are susceptible to dry-rotting when subjected to cycles of drying and wetting; while both shallow and deep foundations can experience settlements caused by the consolidation of the clay layer due to drainage from the base of the clay layer.

The process of depressing the Central Artery in the Central Area involves a major cut-and-cover construction effort. The depths of excavations required for the tunnels range from 40 to 110 feet. The widths of excavations range from about 55 to 235 feet. The depths of excavations for the ventilation buildings range from about 90 to 140 feet. Excavation support is to be provided generally by soldier pile and tremie concrete (SPTC) walls constructed using slurry techniques, although in many cases conventional slurry walls with reinforcement cages may be used. Excavation will be performed between cross-braced perimeter slurry walls, with an additional center slurry wall in the wider sections.

The slurry walls, apart from providing temporary excavation support, are to be incorporated as part of the permanent tunnel structure. In addition, the existing elevated Central Artery, which is to remain in service during construction, will be supported on the slurry walls through a series of transfer beams and girders at the existing footing or pile cap level. The slurry walls generally penetrate at least 10 feet into the bedrock.

Subsurface Conditions

The subsurface conditions at each of the pumping test well locations are summarized in the schematic shown in Figure 2. The typical subsurface profile in the Central Area consists of a surface layer of urban fill, under which a layer of organic soil is generally encountered. The organic deposits are underlain by Boston Blue Clay, which is underlain by glacial till. Sand layers are not unusual above, below or within the clay stratum. The glacial till is underlain by bedrock, the upper layers of which typically consist of completely to severely weathered argillite. Following are descriptions of the strata in which the pumping tests were conducted.

Glacial Till. Glacial till encountered in the Central Area typically consists of a heterogeneous mixture of sand, silt, clay and gravel, with cobbles and boulders. Glacial till with a fines content (material passing the No. 200 sieve) of greater than about 30 percent is considered here to be cohesive till and is designated T_1 and T_2. Till consisting primarily of granular soil with less than 30 percent fines is described as granular till and is designated as T_3.

Standard penetration test (SPT) N-values for the till are typically greater than 50 blows/foot, reflecting a very dense soil. However, some zones with N-values less than 50 blows/foot are also present in the till, indicating looser materials, especially toward the top of the cohesive till stratum. The cohesive till with SPT values less than 50 blows/foot is designated as T_1, while the very dense cohesive till with SPT values greater than 50 blows/foot is designated as T_2. The granular till, T_3, generally has SPT values greater than 50 blows/foot.

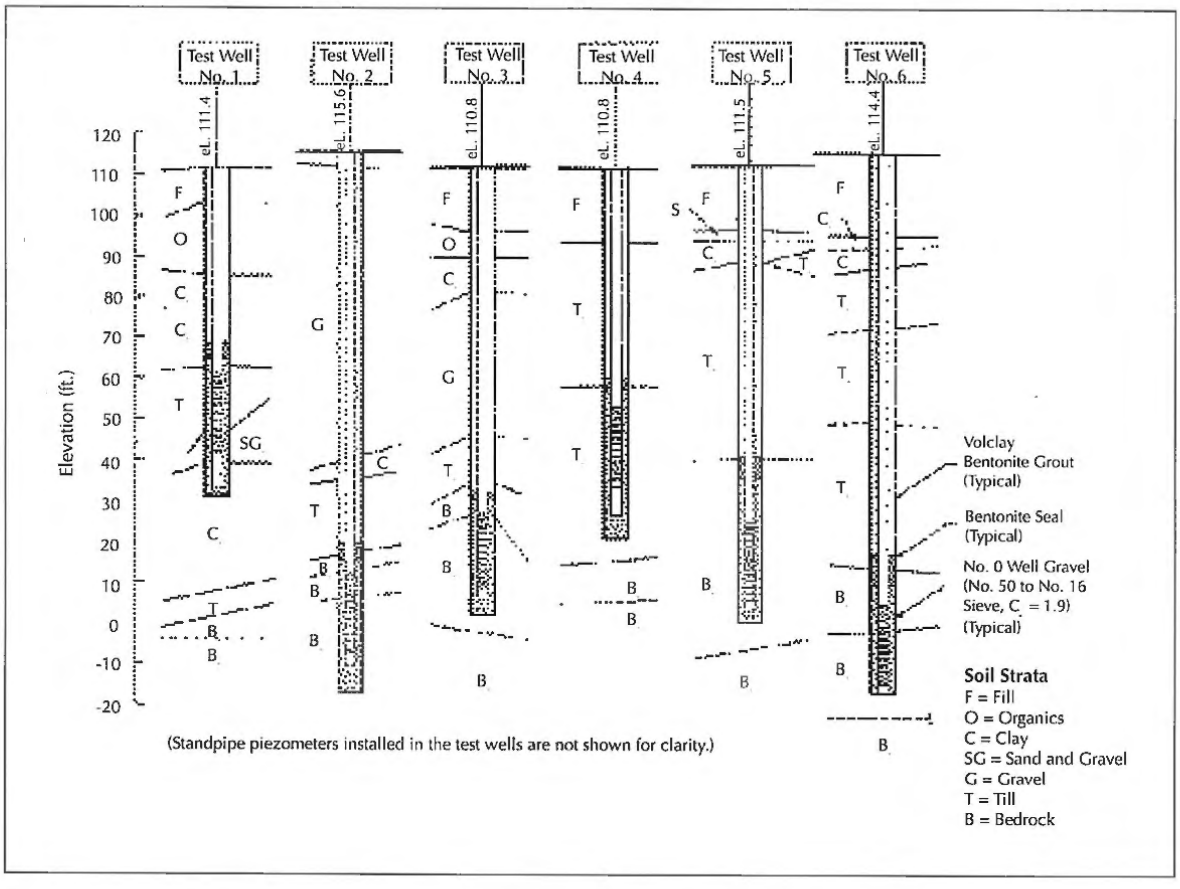

FIGURE 2. Subsurface conditions at the pumping test well sites.

Borehole permeability tests indicate that the permeability of the T_1 and T_2 till deposits ranged from 1×10^{-6} to 4×10^{-4} cm/sec and that of the T_3 till ranged from 1×10^{-4} to 3×10^{-3} cm/sec.

Sand & Gravel. Localized natural deposits of sand (S) and sand and gravel (SG) were encountered within the clay, glaciomarine and glacial till deposits in the northern part of the Central Area. The thickness of the localized granular strata were typically less than 10 feet. The gradation of the S and SG deposits ranged from fine sand with silt to widely graded sand and gravel.

SPT N-values in the S and SG deposits ranged from 8 to over 100 blows/foot, with most of the values greater than 30 blows/foot, indicating a dense to very dense condition. Sand and sand and gravel deposits with SPT values less than 50 blows/foot have been designated S_1 and SG_1, respectively, and deposits with SPT values greater than 50 blows/foot have been designated S_2 and SG_2, respectively. Part of the pumping stratum in Pumping Test No. 1 was located in this deposit.

Bedrock. Top of bedrock in the Central Area was observed to vary between depths of 65 and 130 feet (el. 57 to el. –15) in the borings drilled along the proposed artery alignment. Bedrock was sampled by coring or, in some cases where the rock was sufficiently weathered, by split spoon. Depths of coring ranged from 5 to 103 feet. Rock recoveries in the coring runs were generally between 30 and 70 percent, although some were as high as 100 percent. The bedrock encountered in the Central Area is part of the Cambridge formation of the Boston Basin. The predominant rock type recovered in the borings was Cambridge argillite. Other rock types found include interbedded medium-grained sandstone and conglomerate, tuffaceous argillite and intrusive rocks such as basalt, andesite and diabase.

The argillite encountered in the borings is fine grained, very thinly bedded and is characterized by alternating dark and light grey beds of silt- and clay-sized particles. The rock is regionally folded, and locally the dip of the bedding is at a generally high angle (*i.e.*, 55 to 85 degrees) from the north end of the Central Area to just north of Summer Street, although it varies locally from horizontal to vertical. South of Summer Street the bedding is generally moderately dipping (*i.e.*, 35 to 55 degrees).

The top of the rock is typically moderately to extremely fractured, with close to very close joint spacing.

The argillite has been grouped into three categories based on weathering:

- B_1 rock, consisting of completely weathered or very severely weathered argillite;
- B_2 rock, consisting of severely or severely to moderately weathered argillite; and,
- B_3 rock, consisting of moderately weathered, slightly weathered, and unweathered argillite.

Groundwater. The groundwater regime in the Central Area is typically distributed into two distinct aquifers: an unconfined shallow aquifer above the clay that acts as an aquitard, and a confined aquifer below the clay.

Groundwater levels measured in shallow observation wells in the fill, organic soils and shallow till ranged from el. 95.0 to el. 112.0, with most levels above el. 98.0. There was no clear pattern to the groundwater levels, nor was there a clear gradient toward the harbor. These data indicate that the groundwater elevations at any given point may be influenced more by local subsurface drainage, such as sumps, leaking sewers or storm drains, or drainage along pipeline bedding, than by the overall groundwater flow pattern.

The groundwater levels in the deeper observation wells and piezometers in clay, glaciomarine, till and bedrock strata ranged from el. 68.0 to el. 111.0, with most levels above el. 89.0 and below el. 102. Most of the piezometric levels in the clay, glaciomarine deposits, till and bedrock are below mean sea level, indicating that they may be affected by local subsurface drainage or pumping.

The tidal fluctuations in the groundwater level in the fill and shallow organic, clay, glaciomarine and till deposits ranged from 0 to 2.3 feet with most values below 0.6 feet, although values as high as 2.9 and 7.6 feet were measured at two locations. The tidal fluctuations in the deep wells and piezometers in the glaciomarine deposits, till and bedrock ranged from 0.1 to 2.0 feet with most values below 1.4 feet.

The groundwater elevations reported here are based on groundwater data collected mostly in 1991 and 1992.

Borehole Permeability Testing

In-situ permeability tests were performed in boreholes to estimate the permeability of the soils and bedrock during the drilling program that was conducted in the Central Area of the project. These tests included rising head and falling head tests in soil and rock, and packer pressure tests in rock.

The borehole permeability data indicate that the granular fill and the rock are considerably more permeable than most of the other soils, which act as low permeability layers between the two primary aquifers. The groundwater observation well and piezometer data support the observation that there are two primary aquifers: a shallow unconfined aquifer in the fill and a confined aquifer in the rock.

The borehole permeability test results indicate that, in general, the permeability of the rock ranges from 10^{-6} to 10^{-2} cm/sec. Permeability values as low as 2×10^{-7} cm/sec were measured. In general, the rock was found to be 1 to 3 orders of magnitude more permeable than the overlying till. However, in many areas the granular till was found to be as permeable as rock.

The bedrock permeability data indicate that, in many borings, the more permeable zones occur essentially at random within the rock, both horizontally and vertically. The borehole permeability data did not show a clear trend of decreasing rock mass permeability with depth within the depth of the borings (in excess of 100 feet below the top of the rock). The data also indicate that there does not appear to be a significant difference in the range of permeability between the completely

weathered argillite (B_1) and the less weathered rock (B_2 and B_3).

Pumping Tests: Installations

The installations at each of the pumping test sites consisted of a 12-inch diameter pumping test well screened and sealed in the desired test stratum, piezometers and observation wells. An array of piezometers was installed in the pumping stratum at progressively larger distances away from the test well in two directions approximately perpendicular to each other.

Pumping Test Well Installation. At each site, the piezometer boring closest to the pumping test well (typically 5 feet away) was drilled prior to start of the test well drilling. Split-spoon samples were collected at 5-foot intervals, and strata changes were recorded in this boring. The information obtained was used during the test well installation to select depths for well screen installation and to characterize the subsurface strata.

The pumping test wells consisted of 6-inch-diameter polyvinyl chloride (PVC) screen and riser pipe installed in a 12-inch-diameter drilled hole. Drilling for the test well was performed by a mud/air rotary drill rig using a direct mud rotary drilling technique. To prevent the collapse of the drill hole in the fill, the top 20 feet of the drill hole were cased with a 16-inch-diameter steel pipe. The remaining depth of the test well was drilled open hole with drilling fluid. A biodegradable drilling fluid was used in continuous circulation to remove the cuttings and keep the hole stabilized. The fluid is an organic drilling fluid that breaks down naturally. The purpose of using a biodegradable drilling fluid was to minimize the effects of the drilling fluid on the permeability of the formation to be tested.

In all the test wells, the screened sections were 6 inches in nominal diameter and 20 feet long. The screened sections were made up of 10-foot-long Schedule 80 PVC sections with an internal diameter of 5.68 inches and provided with 0.02-inch-wide slots. The riser pipe also consisted of Schedule 80 PVC pipe. The bottom of the screened sections were provided with PVC silt traps and plugs. The silt trap was a section of riser pipe typically 1.5 feet long (except for Sites 1 and 4 where it was 8 feet long). The longer silt traps were used where the well screens were in soil to permit setting the pumps at a lower elevation.

Two standpipe piezometers were installed in the test well hole concurrently with the installation of the well screen and riser to permit measurement of the water level in the test well. The standpipes were taped to the outside of the test well screen and riser. The standpipes consisted of a 1.38-inch-inside-diameter slotted Schedule 40 PVC pipe 24 inches long attached to a 1.38-inch-inside-diameter Schedule 40 PVC riser pipe. The slots in the screen were 0.01 inch wide. The standpipe piezometers were also provided with a 1.5-foot-long silt trap and bottom plug. Two identical standpipes were provided in case one malfunctioned.

The test wells were completed by installing a filter pack around the well screen. The filter pack consisted of well gravel composed of particles ranging from No. 50 to No. 16 sieve size with a uniformity coefficient of 1.9. The filter pack generally extended 2 to 11 feet above the top of the well screen and 2 to 13 feet below the bottom of the well screen. The drilling fluid was flushed from the hole with clean water prior to installing the filter pack. Flushing was continued until the return water was usually clear. All well materials (filter pack, seals and riser backfill) were tremied into place during the installation.

A bentonite seal was installed above the filter pack, and the drill hole was grouted above the seal using a bentonite grout to prevent leakage from the upper layers. The bentonite seal varied in thickness from 2 to 7 feet. A 12-inch-diameter road box was installed at the surface to enclose the test well riser pipe and the two standpipes.

Piezometer & Observation Well Installations. Vibrating wire piezometers were used as the primary means of monitoring drawdown in the pumping test program because of their ability to provide a rapid response to changes in piezometric head without significant lag. An array of piezometers was installed in the pumping stratum at progressively larger distances away from the test well in two directions approximately perpendicular to each other. In addition, a few piezometers and observation wells were installed in the overlying clay, till

and fill layers to measure groundwater elevation or piezometric head changes in these overlying soils.

The piezometers installed for the pumping test program were assembled within a 12- to 14-inch-long, 1.25-inch-inside-diameter machine-slotted Schedule 40 PVC well screen. At several locations, two or three piezometers were installed in the same borehole with each piezometer located in a different stratum. The elevations of the piezometer tips ranged from el. -3.9 to el. 58.8.

All observation wells in the pumping test program were installed as shallow observation wells primarily in the fill, organics or upper portion of the clay deposit. Each observation well generally consisted of a 2-inch-inside-diameter machine-slotted Schedule 40 PVC well screen, connected with a threaded flush joint to a 2-inch solid-wall Schedule 40 PVC riser pipe extending to the ground surface.

In addition to the piezometers and observation wells installed as part of the pumping test program, a number of piezometers and observation wells installed earlier as part of the geotechnical investigation for the various design sections in the Central Area were also monitored during the pumping tests. (Detailed descriptions of the drilling and sampling procedures and of the piezometer and observation well installation procedures are provided elsewhere.[1-3])

Split-spoon samples were collected at 5-foot intervals, and strata changes were recorded in the piezometer boring closest to the pumping test well. The other borings drilled in the pumping test program for piezometer and observation well installation were generally advanced to the required depth of installation without sampling.

Test Well Development. Well development was performed to remove fines from the filter pack around the well screen and the surrounding pumping stratum. Test well development was performed using a two-step procedure. The first step included air-lift development and mechanical surging, while the second step utilized a combination of pumping and mechanical surging.

During the first step of the well development, air-lift surging and mechanical surging were combined to achieve a gross cleaning of the fines in the filter pack and surrounding material. This step was used to preclude the clogging of the submersible pumps. Pumping during this step was performed by a suction pump and an eductor pipe. During air surging, a 1-inch-diameter air line was lowered inside a 2-inch-diameter PVC eductor pipe, and compressed air was injected into the well to lift the water to the surface. Surging was achieved by alternately injecting and cutting off the air supply to allow the water column to rise and fall. Mechanical surging was performed by operating a plunger up and down in the well screen and riser sections to raise and lower a close-fitting surge block causing water to flow in and out of the screen. Development was continued by alternately repeating these two techniques until the water being pumped through the eductor pipe was observed to be relatively clear. This first step in the development process generally lasted about 4 hours.

During the second step of well development, a submersible pump was lowered into the well and the well was alternately pumped and mechanically surged with surge blocks. This procedure was repeated until the pumped water was observed to be clear and free of sand, silt and bedrock fragments. The second step was generally continued for 4 to 8 hours except in the case of Sites 2, 3 and 4 where a longer period of up to 16 hours was required.

Pumping Tests: Procedures

A step drawdown test was performed prior to the primary pumping test at each of the pumping test sites. The purpose of the step test was to determine the initial pumping rate for the pumping test. The step tests were performed by increasing the pumping rate in steps, while monitoring the drawdown in the test well. The steps in the pumping rate were generally 25, 50, 75 and 100 percent of the target pumping rate. The target pumping rate was established on the basis of observations made during the well development. Each step typically lasted for 2 hours, after which the pumping rate was increased to the next higher step without allowing the well to rebound.

The drawdown observations made during the step test were used to select the initial

TABLE 1.
Summary of Aquifer Parameters

Test Site No.	Location	Pumping Zone: Stratum*	Pumping Zone: Elevation of Sanded Zone	Estimated Effective Thickness of Pumping Stratum (feet)	Pumping Rate (gpm) & Duration (hours)	Elevation of Initial Water Level at Test Well Location (feet)	Drawdown in Test Well at End of Test (feet)
1	N. Washington at Valenti Way	C_2T, SG_2, C_4	65.7 to 31.4	30	5 (60) 10 (54)	101.9	34.5
2	Blackstone at Hanover	B_1, B_2, B_3	16.1 to -16.4	40	25 (72)	98.0	42.0
3	Atlantic Avenue opposite Columbus Park	B_2, B_3	28.8 to 0.8	40	50 (120)	97.1	81.0**
4	Surface Road at India	T_2	57.8 to 20.8	40	0.7 (120)	88.4	31.5
5	Atlantic Avenue at Boston Edison Site	B_1	35.4 to -1.1	50	2 (120)	89.4	43.8
6	Atlantic Avenue at Summer	B_1, B_2	9.2 to -17.6	50	15 (10) 13 (17) 10 (46) 7.5 (47)	87.3	72.6**

Notes: * Soil and rock not sampled in test well. Stratum is assumed to be same as nearest well, typically 5 feet away. See Figure 2 for strata descriptions.
** The drawdown caused the water level to drop below the top of the well screen in the test well at the end of the test.

pumping rate so that the projected drawdown at the end of the pumping test would be large enough to be representative of the dewatering anticipated during the construction period but not so great as to cause the water level to drop below the top of the well screen. In Pumping Test No. 6, the pumping rate had to be decreased during the test because of excessive drawdowns. In Pumping Test No. 1, the pumping rate had to be increased during the test because of small drawdowns. Table 1 presents a summary of the aquifer parameters.

Pumping Test & Recovery Monitoring Procedures. The pumping tests were performed using a submersible electric pump at the bottom of the wells. The submersible pump used for the pumping tests was rated to provide a maximum flow rate of 50 gpm at 130-foot head. Electric power was supplied to the pump by a diesel-powered generator. A backup generator was also provided.

Flow was measured by an accumulating total flow meter, which measured the volume of water in cubic feet. The average flow rate was determined by dividing the volume of water pumped by the elapsed time between successive readings. Flow meter readings were initially taken every 30 seconds. After a stable flow was achieved, the flow was measured every 30 minutes until the end of the test. The pumped water was discharged into a sedimentation basin before being discharged into storm sewers as required in the discharge permits obtained for the program by the project's contractor. Samples of the pumped water were collected for water quality analyses for each of the pumping tests to demonstrate compliance with the requirements of the discharge permits.

Piezometers and observation wells were monitored during the pumping test to measure drawdown. The water level also was monitored in the standpipes installed in the filter pack around the well screen. The flow velocities through the well screen in the test wells were estimated to be small enough so the head loss through the well screen could be considered negligible.

Water levels in the test well standpipes and in the observation wells were measured to an accuracy of 0.01 foot. Prior to conducting the

pumping test, water levels were measured under static (non-pumping) conditions in the pumping test well at 6-hour intervals for a 24-hour static data collection period. This data collection period started at least 6 hours after completion of the step test to allow the wells to recover.

Upon the start of the pumping test, water levels and piezometric levels were monitored at intervals ranging from 10 seconds to 4 hours, with the most frequent monitoring taking place within the first 5 hours according to the schedule shown in Table 2.

After pumping was stopped, groundwater recovery was monitored in the test well filter pack and in selected piezometers for a minimum of 24 hours for each of the pumping tests. The frequencies at which recovery readings were taken was the same as those described above in the schedule for the pumping test.

All pumping tests were continued for 120 hours, except for Pumping Test Nos. 1 and 2. Pumping Test No. 1, located in a parking lot at the corner of North Washington Street and Valenti Way, was performed for 114 hours and was terminated 6 hours before the scheduled time because of generator failure. Pumping Test No. 2, on Blackstone Street, was terminated after 72 hours because of restrictions on permissible working times in the Haymarket area.

TABLE 2.
Monitoring Schedule

Time After Commencement of Test (minutes)	Reading Frequency
0-2	Every 10 seconds
2-5	Every 30 seconds
5-15	Every 1 minute
15-50	Every 5 minutes
50-100	Every 10 minutes
100-300	Every 30 minutes
300-2,880	Every 1 hour
2,880+	Every 4 hours

Test Data

Details about the pumping test program including installation procedures, borehole permeability testing, subsurface profiles at the pumping test sites, data reduction, analysis and interpretation, as well as water quality sampling and testing are provided elsewhere.[1-3]

Figures 3 and 4, which pertain to Pumping Test No. 3, are presented as examples of the data that were collected during each of the pumping tests. Figure 3 shows the drawdowns in the test well and the different piezometers and observation wells that were monitored during the pumping test as well as the flow rate versus the log of time. Figure 4 shows the residual drawdown in the piezometers and observation wells monitored during the recovery period versus log of time (with time, t, equal to 0 — being the time when pumping was stopped). The drawdown versus log of distance is plotted for Pumping Tests Nos. 1 to 3 in Figure 5 and for Pumping Test Nos. 4 to 6 in Figure 6.

Results of Analyses

Because of the limited pumping periods during the pumping tests, drawdowns were still increasing at the end of the pumping tests. Therefore, transient methods of analyses were used to analyze the test data. Transmissivity and storativity values were obtained using two different methods — namely, the Jacob Semilog Plot Method and the Hantush-Jacob Curve Matching Method for leaky aquifers. For comparison, the transmissivity also was calculated using the drawdown versus distance plot method, which assumes a steady-state condition at the end of the test.

Jacob Semilog Plot Method. The Jacob Semilog Plot Method consists of plotting the measured drawdowns in any well versus the elapsed time since the start of the pumping test on a semi-logarithmic scale and fitting a straight line through the data. The slope of the line as well as its intercept with the horizontal axis (*i.e.*, the time axis) are used to calculate the transmissivity and the storativity. The Jacob Semilog Plot Method offers an advantage when analyzing confined aquifers with impermeable and/or recharge boundaries, because

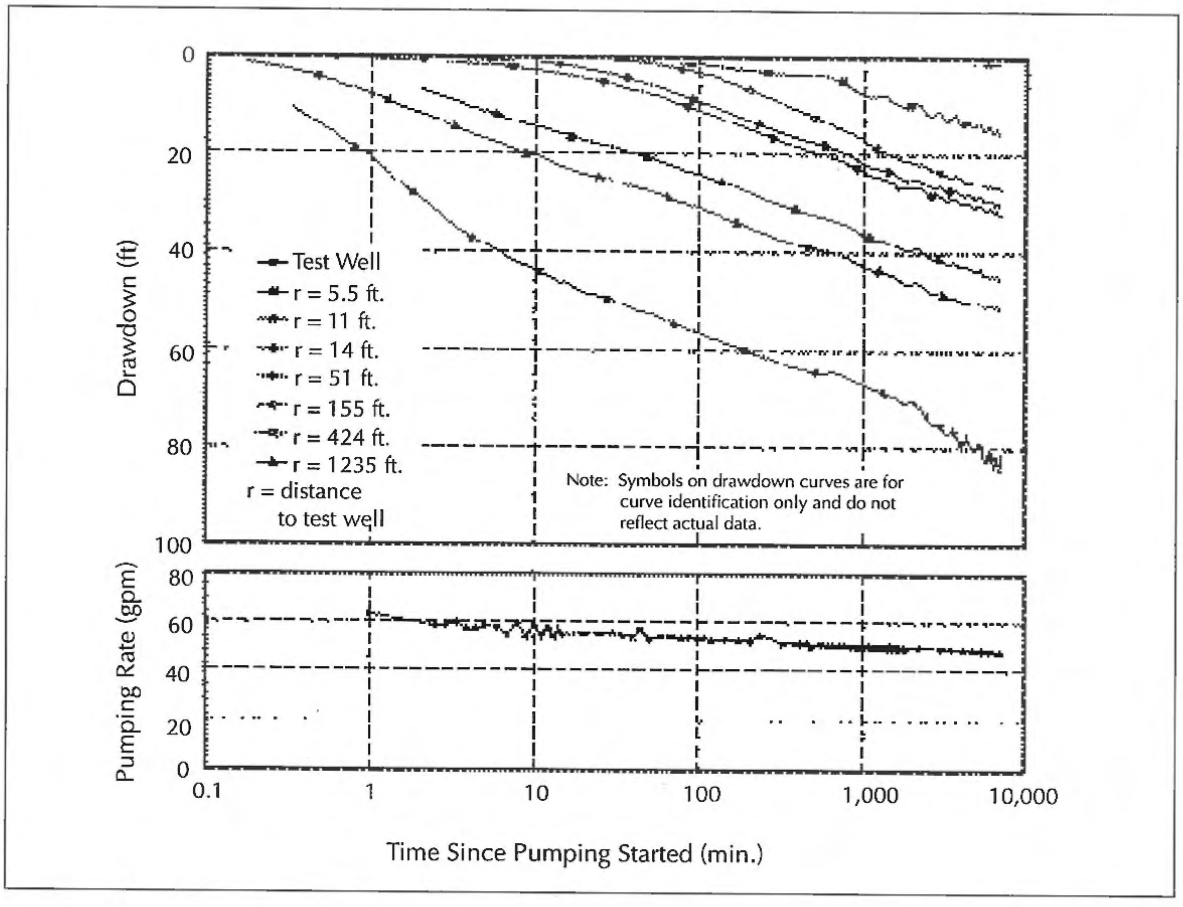

FIGURE 3. Drawdown in rock and the flow rate versus time for Pumping Test Site No. 3.

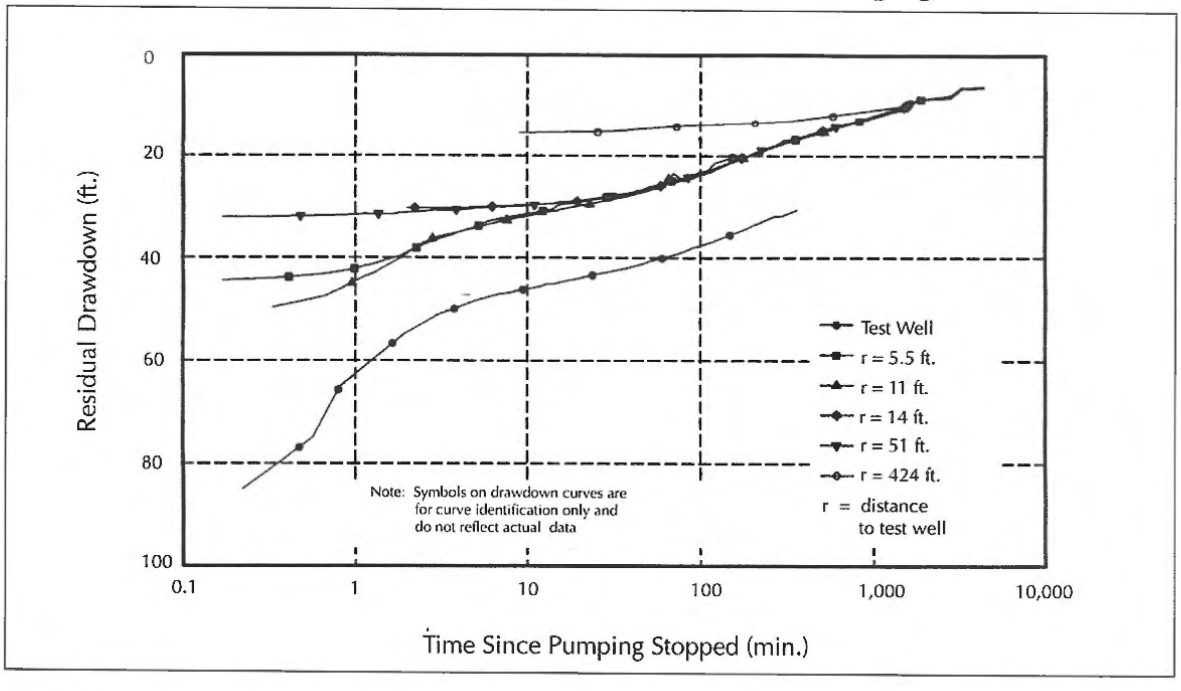

FIGURE 4. Recovery curves for instruments installed in rock for Pumping Test Site No. 3.

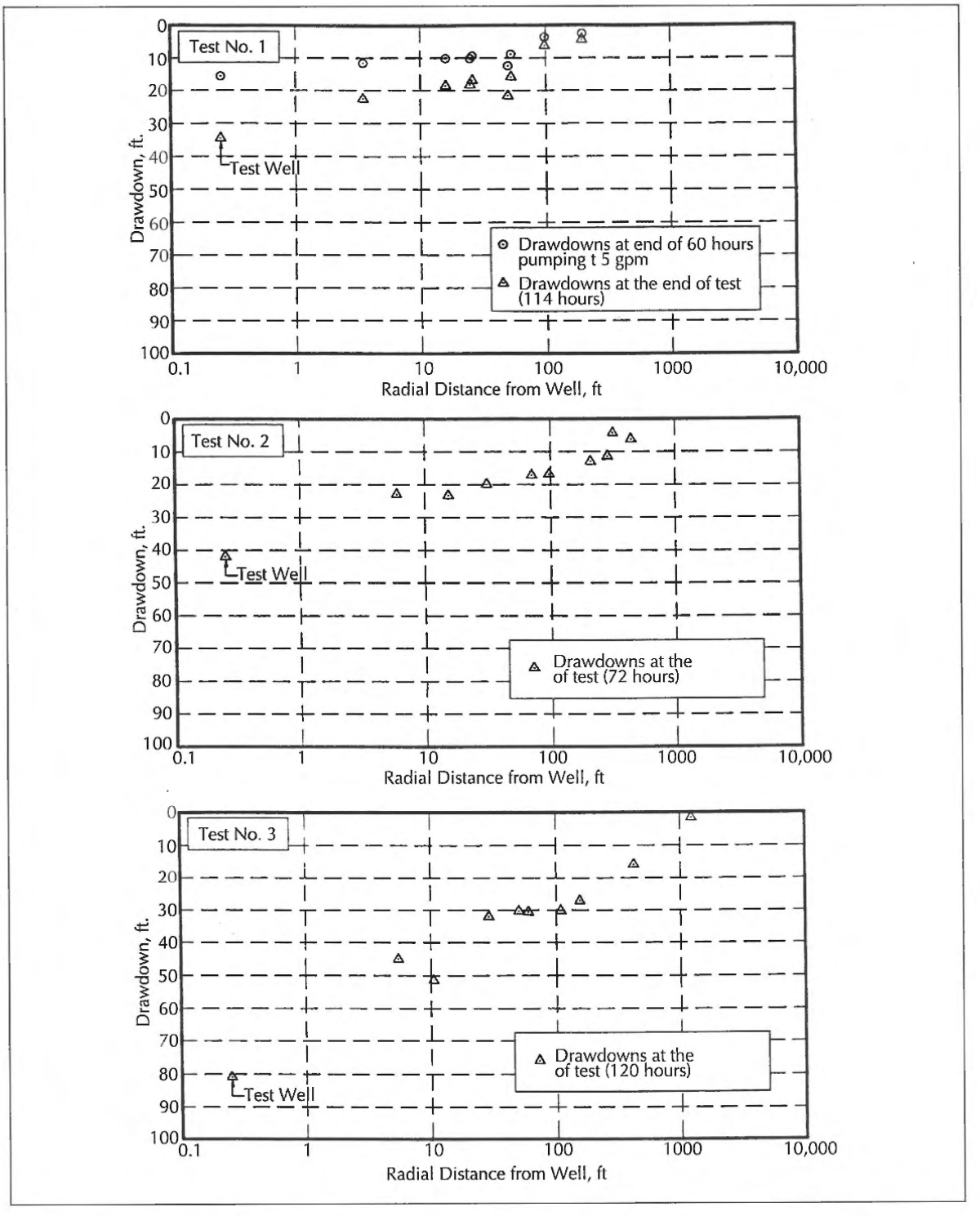

FIGURE 5. Drawdown versus distance (logarithmic scale) for Pumping Tests Nos. 1 to 3.

the presence of a boundary is reflected by a break in the drawdown versus time plot.[4]

The use of this method assumes that the aquifer is confined, saturated, homogeneous and isotropic. The groundwater flow is assumed to take place radially in the horizontal direction. The method also assumes that the test well is screened over the entire thickness of

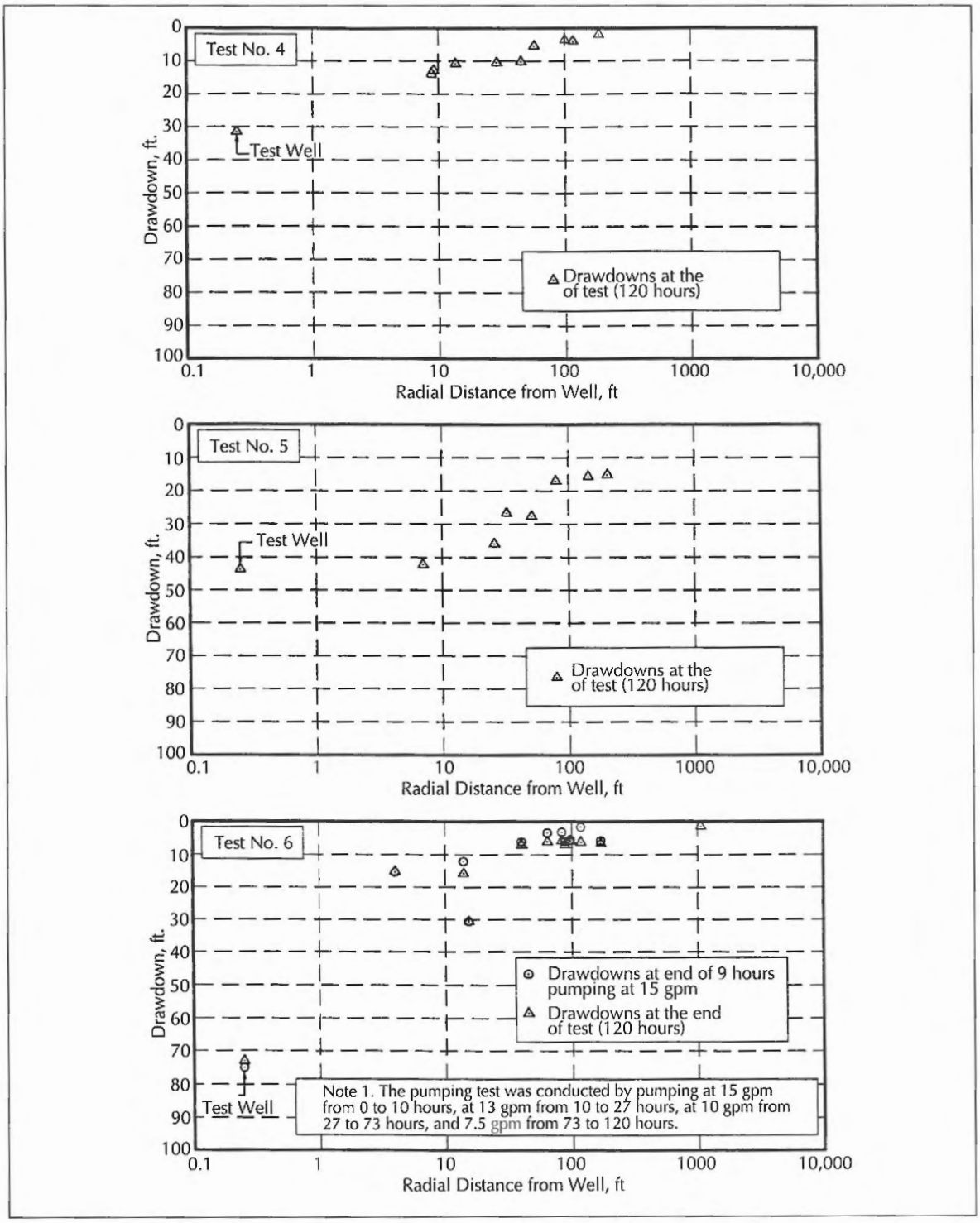

FIGURE 6. Drawdown versus distance (logarithmic scale) for Pumping Tests Nos. 4 to 6.

the aquifer. A number of other assumptions made for mathematical convenience in the Jacob Semilog Method are discussed by Dawson and Istok,[5] and Driscoll.[6]

The Jacob Semilog Plot Method is applicable for values of $u = (1.87\ r^2S/Tt)$ smaller than 0.05 — *i.e.*, for large values of t and small values of r (r is the distance from the center of a

pumping well in feet, S is the coefficient of storativity, T is the coefficient of transmissivity in gpd/ft and t is the time since pumping started in days).

Hantush-Jacob Curve Matching Method. The Hantush-Jacob Curve Matching Method is applicable to leaky confined aquifers. In a leaky aquifer, a part of the pumped water comes from water infiltrating (or leaking) from a less permeable overlying stratum called the aquitard. This leakage, caused by the drop in head in the pumped aquifer, consists of water from aquitard storage and/or the flow of water from an upper unconfined aquifer through the aquitard. The Hantush-Jacob Method assumes that all leakage occurs as a result of downward flow through the aquitard (*i.e.*, no water is released from aquitard storage).

In the Hantush-Jacob Curve Matching Method, the measured drawdowns in any well (other than the test well) are plotted versus the elapsed time since the start of the pumping test on a log-log scale. The obtained curve is then compared to a set of dimensionless plots of the theoretical response of an ideal homogeneous aquifer, called type curves. The shapes of the type curves vary with the degree of leakage. Once a match is obtained, a match point is selected — the coordinates of which are used to calculate the transmissivity and the storativity. The permeability of the aquitard can also be determined using this method of analysis.

The Hantush-Jacob Curve Matching Method assumes that the aquifer is bounded above by an aquitard and an unconfined aquifer and below by an impermeable stratum. The aquifer and the aquitard are assumed to be homogeneous and isotropic. The groundwater flow in the aquitard is assumed to be vertical, while the groundwater flow in the aquifer is assumed to take place radially in the horizontal direction. The method also assumes that the test well is screened over the entire thickness of the aquifer. A number of other assumptions made for mathematical convenience in the Hantush-Jacob method are discussed in detail by Dawson and Istok,[5] and Driscoll.[6]

Distance Versus Drawdown Method. The Distance Versus Drawdown Method consists of plotting the measured drawdowns on an arithmetic scale versus the distance from the test well to a logarithmic scale and fitting a straight line through the data. The equations developed for this method indicate this plot should be linear. The slope of the line, as well as its intercept with the drawdown axis, are used to calculate the transmissivity.

This method assumes that the aquifer is homogeneous and isotropic (having a constant thickness). Although the basic solution assumes that the well screen fully penetrates the aquifer, modified solutions for partially penetrating wells are available.

The equations and the assumptions associated with the drawdown versus distance method are discussed by Mansur and Kaufman.[7]

Table 3 summarizes the results and the calculated aquifer parameters for the six pumping tests performed in AGC Area 04. The radius of influence for each of the pumping tests was estimated by extrapolating the drawdown versus distance data plotted on a semilog scale. The table lists the ranges of values of transmissivities and storativities obtained by the three analysis methods used.

Data from all of the six pumping tests were analyzed assuming confined aquifers. This assumption is reasonable for the pumping tests performed in bedrock (Pumping Tests 2, 3, 5 and 6) because the subsurface conditions correspond to a pervious fractured bedrock formation overlain by a relatively impervious cover of glacial till, glaciomarine or clay deposits. These aquifers were also considered to be leaky confined aquifers because piezometers installed in the till and even in the clay overlying the bedrock (*i.e.*, installed in the aquitard) displayed substantial drawdowns. These drawdowns were observed to be higher at the bottom of the aquitard but were less than those in the pumped aquifer, clearly indicating downward flow (*i.e.*, leakage).

The data plots indicate that the fractures in the bedrock are sufficiently interconnected so that the assumption (inherent in the analytical method) that the aquifer is isotropic is a reasonable one.

The subsurface conditions for Pumping Test No. 1 show that the pumped stratum of sand

TABLE 3.
Summary of the Pumping Tests

Test Site No.	Location	Pumping Stratum	Estimated Radius of Influence (ft)	Aquifer Parameters				
				Jacob Semilog Plot Method		Hantush-Jacob Method		Drawdown Distance Method*
				Transmissivity (gal/day/ft)	Storativity ($ft^2/ft^2/ft \times 10^{-5}$)	Transmissivity (gal/day/ft)	Storativity ($ft^2/ft^2/ft \times 10^{-5}$)	Transmissivity (gal/day/ft)
1	N. Washington at Valenti Way	C_2T, SG_2, C_4	350	194-314**	55-1,300§	190-197§	220-1,500§	266
2	Blackstone at Hanover	B_1, B_2, B_3	2,200	673-942	20-880	670-796	19-1,100	1,166
3	Atlantic Ave. opposite Columbus Park	B_2, B_3	1,900	886-1,257***	30-1,300***	868-970***	44-1,300***	1,266
4	Surface Rd. at India	T_2	300	28.9-44	5.7-220	—	—	35.8
5	Atlantic Ave. at Boston Edison	B_1	350	34.1-46.3	1.6-70	29.4-35.8	5.8-49	44.3
6	Atlantic Ave. at Summer	B_1, B_2	1,200	330-762§	35-3,700§	312-716§	26-3,000§	204-824

Notes: * Transmissivity calculated using drawdowns at the end of the pumping test.
** Calculations based on data from first 60 hours of pumping.
*** The analysis assumes that the aquifer is fully saturated and that the water level does not drop below the top of the well screen in the test well.
§ Calculations based on data from the first 9 hours of pumping.

and gravel and till is overlain by clay and underlain by a layer of hard clay that pinches out 100 to 300 feet west of the pumping well, which supports the assumption of a confined aquifer. The drawdown in piezometers installed in rock in the immediate vicinity of the pumping well at the end of this test was less than 4 feet.

In Pumping Test No. 4, the pumping was performed from a glacial till stratum overlain by glaciomarine and fill deposits and underlain by bedrock. The observation wells in the overlying glaciomarine and fill deposits did not undergo any drawdowns during the pumping test. Although the bedrock underlying the glacial till may be more pervious than the till, the data were analyzed as if the till were a confined aquifer. If the bedrock were a source of partial recharge for the glacial till, the assumption of a confined aquifer would result in a calculated permeability value greater than the actual permeability of the glacial till.

The effective thickness of the aquifer or the pumping stratum was estimated on the basis of the height of the sanded zone around the well screen and subsurface conditions at the test well location. The estimated aquifer thicknesses are shown in Table 3.

Conclusions

The pumping tests indicated that rock is frequently more permeable than the overlying soils. Of the four tests that were conducted in rock, three indicated permeabilities in the range of 0.5 to 2×10^{-3} cm/sec, while the fourth indicated a permeability of about 5×10^{-5} cm/sec. These results are in the same range as indicated by the borehole permeability tests in rock. One pumping test was performed in glacial till and indicated a permeability of about 5×10^{-5} cm/sec, which is near the middle of the range measured by the borehole permeability tests in glacial till. One pumping test was performed in glacial till and sand and gravel deposits and indicated a permeability of about 5×10^{-4} cm/sec.

The range of permeability values estimated from the pumping test for various strata were consistent with borehole permeability data and with permeability data values from grain size correlations as shown in Table 4.

The radii of influence estimated from the four pumping tests in rock ranged from 350 to 2,200 feet for drawdowns in the pumping wells ranging from 42.0 to 81.0 feet (as shown in Ta-

TABLE 4.
Summary of Permeability Values

Soil/Stratum Bedrock	Permeability (cm/sec)		
	Pumping Test	Borehole Permeability	Correlation With Grain Size
Bedrock	B_2 & B_3 Bedrock: 0.5×10^{-3} to 2.0×10^{-3} B_1 Bedrock: 5×10^{-5}	1.0×10^{-6} to 1.0×10^{-2} with values as low as 2.0×10^{-7}	NA
Glacial Till	5.0×10^{-5}	T_1, T_2: 1.0×10^{-6} to 4.0×10^{-4} T_3: 1.0×10^{-4} to 3.0×10^{-3}	3.0×10^{-5} to 5.0×10^{-4}
Sand & Gravel	5.0×10^{-4}	1.8×10^{-5} to 3.4×10^{-4}	3.0×10^{-5} to 5×10^{-4}

Note: NA = Not available

ble 3). The extent of the zone of influence from pressure relief during construction may be affected by a number of factors including variations in rock mass permeability; the amount, duration and area of pressure relief; and the use of grout cutoff curtains. The zones of influence during construction are likely to be somewhat larger than those observed in the pumping test program unless grout cutoff curtains are used in more permeable zones of rock.

Groundwater control in the deep excavations for the mainline highway tunnels and the ventilation buildings will consist primarily of relieving piezometric pressure in the rock below the excavation. The proposed excavations for the mainline highway tunnels and the ventilation buildings extend to depths in excess of 100 feet below the piezometric water level in the rock underlying the bottom of the excavations. In much of the Central Area, excavation to the final subgrade elevation will leave a relatively thin layer of lower permeability soil remaining below the bottom of the excavation or will reach the rock. Pressure relief is necessary to avoid excess upward gradients in the less permeable soils and completely weathered rock, which could cause severe disturbance and softening, and to avoid mass uplift of the less permeable layer remaining below the bottom of the excavation. At the same time, it will be necessary to prevent excessive lowering of the piezometric pressure in the rock outside of the excavation limits that could lead to lower piezometric levels and cause consolidation and settlement of the overlying cohesive soils.

The relief wells will need to extend to a depth great enough to intercept potential water-bearing layers as necessary to control uplift and upward gradients. Therefore, the pressure relief wells may extend well below the bottom of the slurry walls and may lower the piezometric level outside the limits of CA/T construction. The pumping test program provided direct indications of potentially high pumping rates and large zones of influence. The significance of this observation is that grout curtains alone may not be effective in minimizing the drawdowns outside the limits of the excavations for the cut-and-cover tunnels during the pressure relief operations because of the presence of deep permeable zones of rock. Therefore, recharge wells are required to control drawdowns outside the excavation walls.

Observations from the pumping tests also indicate that lowering of the water pressure in the rock aquifer is not likely to directly affect the groundwater level in the fill due to the thickness of lower permeability soils (organic deposits, clay, glaciomarine and till) between the fill and rock.

NOTES & ACKNOWLEDGMENTS — *GEI Consultants, Inc., of Winchester, Mass., together with Mueser Rutledge Consulting Engineers of New York City and Barrientos and Associates of Somerville, Mass., formed the Area Geotechnical Consultant (AGC) team for the Central Area. D.L. Maher Company of North Reading, Mass., was the contractor for the pumping test program. The Mas-*

sachusetts Highway Department and Bechtel/Parsons Brinckerhoff were the owner and management consultant for the CA/T Project, respectively. All elevations in this article are referenced to the CA/T Project datum, which is 100 feet below the National Geodetic Vertical Datum (NGVD) — i.e., project el. 100 is equal to the mean sea level of 1929.

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Smart Growth Strategies for New England

It will take the concerted efforts of government agencies on all levels, non-profit and professional organizations, engineering firms and an educated public to effectively manage growth.

CYNTHIA CHABOT & BRIAN BRENNER

On February 2, 1999, the first Smart Growth Strategies for New England Conference was held in Boston. The concept of the conference grew out of discussions among staff and leaders of the U.S. Environmental Protection Agency's (EPA) New England office and many of the agency's partners in preserving and protecting this special corner of the country. The purpose of this one-day conference was to provide attendees with an opportunity to hear about ways to foster economic growth while protecting natural resources and quality of life.

Specifically, the conference's agenda was comprised of a morning plenary session, followed by concurrent breakout sessions that focused on the barriers to and incentives for achieving smart growth. Using a working lunch format, the agenda then provided for the identification of specific actions needed to create and support livable communities in urban, suburban and rural settings. The conference agenda closed with a plenary summarizing the lunchtime breakout sessions and a call to action.

The speakers and chairs of all of these sessions represented the region's leaders and experts from various facets of the development, planning and conservation communities. The conference planning committee was comprised of representatives of diverse organizations. The committee worked together in a cooperative and collaborative manner, reflecting the overall spirit needed to create and support livable communities in New England. With EPA-New England, the following organizations planned and co-sponsored the conference:

- National Association of Industrial and Office Properties;
- National Trust for Historic Preservation;
- The Nature Conservancy;
- New England Chapters of the American Planning Association;
- New England Governors' Conference;
- Trust for Public Land; and,
- Urban Land Institute.

Also participating in planning the conference were representatives of the Massachusetts Executive Office of Environmental Affairs and the Maine State Planning Office.

Fostering Smart Growth

After welcoming the approximately 1,000 attendees, EPA Regional Administrator John DeVillars described the uniqueness and variety of New England — its villages, urban centers, forests, mountains and estuaries. He also described the adverse land use and fiscal effects that unplanned growth is having on the region, including:

- the loss of more than 1,200 acres of New England's open space a week to development — including nearly 2 acres each hour in Massachusetts alone;
- the development of 26,000 acres in Rhode Island over the past ten years — an area the size of two Providences — while the state's population has remained stable;
- the expenditure in Maine of $727 million (1970-1995) on new school construction in fast growing towns, while the state's public school population shrank by 27,000 students; and,
- for every $1 in revenue a low-density housing development brings to a New England municipality, it costs the community as much as $1.50 in increased expenses to pay for schools, roads and other services.

DeVillars then provided numerous details regarding the environmental impacts of this unplanned growth. Among these impacts:

- more than half of all New England's water pollution now is due to non-point sources;
- habitat destruction and fragmentation are threatening more than 80 percent of the endangered species in the region; and,
- New Englanders are driving nearly a third more miles than they were just a decade ago, an increase of another third from the previous decade. (This increased travel accounts for about 40 percent of the smog pollution that causes violations of health-based air standards for more than 20 days a year in parts of New England.)

Given these statistics, DeVillars said there is a need for all levels of government, as well as other organizations and groups, to work together to solve these problems. He subsequently announced the "EPA-New England Smart Growth Action Plan" as the agency's contribution to solving these problems. The plan has four major components:

- building effective partnerships with a wide variety of organizations such as the conference co-sponsors and associate sponsors, as well as EPA-New England's federal and state agency partners;
- reshaping EPA-New England's programs and policies by using federal authorities to oppose or modify projects that contribute to sprawl, as well as to further develop and support efforts such as Brownfields projects and the Urban Environmental Initiative;
- strengthening local capacity through a "Fundamentals of Smart Growth" training program and a new competitive grants program; and,
- elevating public awareness through workshops, outreach and local conferences across New England.

In conclusion, DeVillars said it was particularly important for EPA-New England to engage a wide range of business, government and nonprofit stakeholders in these efforts, given New England's long and rich history of local rule. (The full text of DeVillars' speech and the Smart Growth Action Plan can be viewed on the EPA-New England web site at www.epa.gov/region01/.)

Legislative Initiatives

The Governor of Vermont, Howard Dean, gave the conference's keynote address. He provided an overview of Vermont's efforts to combat unplanned growth, beginning with a description of Act 250, the state's land use law that guides and supports economic growth without "paving over the state." While Act 250 is not without its faults, Governor Dean said that, for example, the law's prevention of speculative development had a critical role in averting major real estate and banking failures during the recession of the early 1990s. He stated that good land use planning can have economic and unexpected benefits. Nonetheless, Vermont is developing rapidly. In the past 20 years, vehicle miles traveled have doubled, despite only a 25

percent population increase. Meanwhile, areas such as the City of Burlington and the rest of Chittenden County are experiencing enormous growth from commercial development. As a result, the private sector has been an important participant in Vermont's overall efforts to control sprawl. Among the initiatives the Governor mentioned are:

- Government agencies should not be funding roads and sewers that facilitate sprawl. Also, state agencies are required by executive order to locate their offices in downtowns. Further, the Vermont Board of Education adopted a policy in 1997 that recognizes that schools are an essential part of the community by requiring all school districts to thoroughly study renovation options prior to pursuing new school construction that could occur in greenfields or outside of a community center. Post offices also were cited as an essential component of most downtowns, even doubling as general stores in smaller communities. Accordingly, the state has discouraged the U.S. Postal Service from relocation to non-downtown sites.
- Incentives are needed to draw people to downtowns. Vermont's "Downtown Bill" is a start, as it provides for tax credits towards reinvestment projects, planning grants and tax rebates for the rehabilitation of historical structures. Further, the state has assisted with the development of downtown parking garages to facilitate people getting to and utilizing downtowns. Also cited were Vermont's successful negotiations with "big box" developers such as WalMart, which facilitated their location in two instances in either an existing development or in a downtown.
- Land conservation is an essential component of growth management. Governor Dean cited an example of a corporate headquarters that received an initially inappropriate curb cut to locate near national forest land. Instead of using litigation to stop this development and potentially allowing future development by another party, an agreement was negotiated with the developer where a permanent conservation easement was placed on lands adjacent to the headquarters. The conserved land then was deeded to the Vermont Land Trust.

Governor Dean concluded his remarks by stating that New England has a unique opportunity to create land settlement patterns for the next 100 years. For Vermonters, these 200-year old patterns have created a sense of where their communities begin and end, as well as a sense of the land, since they are able to easily access and use it. While not all of New England's residents live in communities of less than 2,500 (as do more than two-thirds of Vermonters), this sense of community can be created throughout the region through local action. To start, he suggested saving critical pieces of land in individual communities and neighborhoods, and creating bike paths out of railroad beds so people have contact with each other. The Governor stated that maintaining a sense of community will take daily work by all involved in order to measure every proposed project against what effect it will have on a community's settlement patterns.

Resources

The following web sites present useful information on creating livable communities and effectively managing growth:

- www.livablecommunities.gov — Federal Livable Communities web site
- www.farmland.org — American Farmland Trust
- www.epa.gov/region5/sprawl/index.html — Antidotes to Sprawl: Federal Contacts to Help Communities Promote Sustainable Land-Use
- www.brook.edu/es/urban/urban.htm — Brookings Institution: Center on Urban and Metropolitan Policy
- www.sustainable.doe.gov/landuse/luintro.shtml — Center of Excellence for Sustainable Development: Land Use Planning Introduction
- www.subjectmatters.com/indicators/index.html — Hart Environmental Data: Indicators of Sustainability
- www.uwex.edu/lgc/growth/growth.htm — LGC Growth Management

- www.lincolninst.edu/main.html — Lincoln Institute of Land Policy
- www.canr.uconn.edu/ces/nemo/ — Nonpoint Education for Municipal Officials
- seagrant.gso.uri.edu/scc—Rhode Island Sea Grant's Sustainable Coastal Communities
- www.sierraclub.org/transportation/ — Sierra Club
- www.smartgrowth.org — Smart Growth Network Homepage
- noel.pd.org/topos/sprawl.html — SPRAWL, an electronic journal.
- www.plannersweb.com/sprawl/sprawl5.html — Sprawl Resource Guide
- 204.97.3.30:8080/apps/sprawl.nsf — Sprawl, Rochester Style (by Mayor William A. Johnson, Jr.)
- www.sprawlwatch.org — Sprawl Watch Clearinghouse
- www.susdev.org — The Sustainable Development Institute
- www.transact.org — Transportation Action Network
- www.tlcnetwork.org — Transportation for Livable Communities Network
- cua6.csuohio.edu/~ucweb/pubs.htm — The Urban Center Publications
- www.vtsprawl.org/index3.htm — The Vermont Forum on Sprawl

The following publications are also good resources on managing growth:

- *Alternatives to Sprawl*, Lincoln Institute of Land Policy, Cambridge, Mass., 1995.
- *Changing Places: Rebuilding Community in the Age of Sprawl*, Richard Moe and Carter Wilkie, Henry Holt & Co, New York, 1997.
- *Cities Without Suburbs*, David Rusk, Woodrow Wilson Center for Special Studies, 1995.
- *Cityroutes Cityrights: Building Livable Neighborhoods and Environmental Justice by Fixing Transportation*, Conservation Law Foundation, Boston, Mass., 1998.
- *Comeback Cities: A Blueprint for Urban Neighborhood Revival*, Paul S. Grogan and Tony Proscio, Westview Press, 2000.
- *Costs of Sprawl: Executive Summary; Costs of Sprawl: Detailed Cost Analysis;* and *Costs of Sprawl: Literature Review and Bibliography, Council on Environmental Quality, Washington, D.C., 1974.*
- *The Geography of Nowhere: The Rise and Decline of America's Man-Made Landscape*, James Howard Kunstler, Simon & Schuster, New York, 1993.
- *Land Use in America*, Henry L. Diamond and Patrick F. Noonan, Lincoln Institute of Land Policy, Cambridge, Mass., and Island Press, Washington D.C., 1996.
- *Why Smart Growth: A Primer*, International City/County Management Association with Goeff Anderson, ICMA-Smart Growth Network, Washington, D.C., 1998.

Another Initiative

The Boston Society of Architects — in cooperation with the Environmental Protection Agency; the Lincoln Institute of Land Policy; the Massachusetts Chapter of the American Planning Association; the Boston Bar Association, Environmental Law Section; and the Urban Land Institute, Boston District Council — will be sponsoring three community-based planning workshops in February and March 2001 to support their initiative Creating Livable Communities — A Civic Initiative for New England.

Finally

One of our biggest asset is our countryside — the mountains, forests, rivers, lakes and valleys. We must not squander what should be cherished. Once we develop land, it is no longer the same. We can look to other countries that have faced these development issues and learn from their experiences — both good and bad. We need to come together as a community, decide what we want for our future and implement plans. We need to create resources that draws people to a community such as good education, affordable housing and places for our elderly and we need to sustain the life of our cities and towns. These decisions and plans beg for community participation and involvement.

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The Restoration & Treatment of Burlington's Groundwater Supply

Meeting and safeguarding a town's water needs posed challenges due to contamination and restrictions on the available land resources.

PAUL C. MILLETT

The town of Burlington, Massachusetts, restored and treated its contaminated groundwater supply from 1982 to 1997. As a result, 4.5 million gallons per day (mgd) — or 3,125 gallons per minute (gpm) — of groundwater is now being withdrawn and treated from the town's largest aquifer, the Vine Brook aquifer.

This project is an example of innovative planning, design, engineering and construction that provides the town of Burlington with a reliable, safe drinking water supply for the next 30 years. Pumped water from the scattered wells is piped to a single treatment plant. Through legal and technical persistence, the town was able to obtain approximately 40 percent of the cost of this plant from the Potential Responsible Parties (PRPs) whose former industrial activities may have led to the aquifer contamination. The plant is fully automated and includes a state-of-the-art computer system that controls seven wells in the aquifer, and three water storage tanks scattered over 5 miles in the town. The plant can operate unattended.

Background

The town of Burlington draws water from two supplies: the Mill Pond surface supply (with its own treatment plant constructed in 1973) and a series of groundwater supply wells located in the Vine Brook aquifer. The history of the development of the water system, source capacity and status is presented in Table 1.

Before 1949, the town's population of approximately 3,000 relied entirely on private wells. In 1949, the first municipal supply — the Main Station wellfield — was constructed off Meadow Road. This wellfield consisted of thirty-five 2.5-inch tubular wells manifolded

TABLE 1.
Burlington Water System

Component	Year Built	Capacity (gpm)	Status
Main Station (Tubular Wellfield)	1949	500	Abandoned 1975
Well 1	1958	400	Active
Well 2	1959	450	Active
Well 3	1962	250	Inactive 1981-1984*
Well 4	1963	300	Inactive 1981-1984*
Well 5	1965	250	Inactive 1981-1984*
Well 6	1966	250	Abandoned 1975
Well 7	1966	250	Abandoned 1988
Well 8	1968	250	Abandoned 1987
Well 9	1970	250	Abandoned 1975
Well 10	1998	1,100	Active
Well 11	1992	350	Active
Mill Pond	1973	4.0 mgd	Active

Note: *Inactive due to high volatile organic compound concentrations.

together that produced approximately 500 gpm. In 1954, the wellfield was expanded to seventy 2.5-inch wells that produced 700 gpm. However, the wellfield was difficult to operate due to recurring leaks in the manifolds and the absence of well screens and well seals, and it was abandoned in 1975.

As indicated in Table 1, Wells 1 through 9 were constructed to tap the Vine Brook aquifer between 1959 and 1970. These wells served the increasing demands of the rapidly growing town and replaced the failing and eventually abandoned Main Station wellfield.

Throughout the 1960s and 1970s, manufacturing activities in the industrial parks surrounding the westerly and southerly sides of the Vine Brook aquifer led to the contamination of several wells with volatile organic compounds (VOCs). Wells 6, 7, 8 and 9 had to be abandoned in the late 1970s and early 1980s due to declining yields and poor water quality. Wells 6 and 9, located off Sandy Brook Road, were abandoned in 1975 due to excessive iron and manganese concentrations, and the presence of sulfate-reducing bacteria. The cost to treat these wells would have been excessive due to the organically bound nature of the iron and manganese. In the early 1980s, the contamination reached concentrations that forced the town to abandon the then-active Wells 3, 4 and 5, primarily due to elevated levels of trichloroethylene (TCE), a known cancer-causing compound. Well 7 was abandoned in 1987 due to elevated concentrations of VOCs ranging from 12 to 20 parts per billion (ppb) (5 ppb is the maximum allowable contaminant level) and iron and manganese problems. Well 8, located on Wyman Street, was abandoned in 1987 due to similar circumstances.

Therefore, by 1985, Wells 1 and 2 were the only remaining wells capable of being used without treatment. The water quality from these wells was consistently high with acceptably low iron and manganese concentrations.

In 1984, the town decided to built an interim facility to remove VOCs from Wells 3, 4 and 5. This facility, known as the Phase I Groundwater Treatment Plant, allowed the town to use these wells (which had been inactive since 1981) during summer months when water demands were higher than could be sustained from the Mill Pond and Wells 1 and 2. The

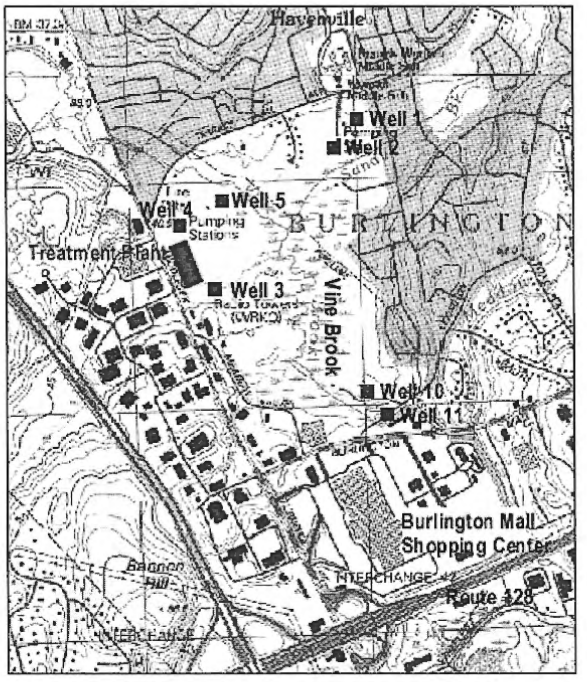

FIGURE 1. Location of Burlington, Massachusetts, water supply system components.

Phase I plant consisted of a pre-engineered metal building that housed electrical panels and controls, a liquid disinfection system, a fluoride system and two blowers for two air stripping towers located outside. The sole purpose of the plant was to provide short relief until a permanent solution could be developed.

From 1979 to 1989, the town engaged in lengthy technical and legal proceedings with the PRPs whose former industrial activities may have led to the contamination. With the support of the Massachusetts Department of Environmental Protection (DEP) and the Attorney General's office, the town reached a settlement of approximately $2,900,000 with the PRPs, enabling the town to proceed with the design and construction of the Vine Brook Groundwater Plant. In addition, the town successfully explored two new well sites, known now as Wells 10 and 11, within the aquifer.

The Vine Brook plant treated water from seven wells at one centralized facility, thereby eliminating the need to construct three separate treatment plants. Wells 1 and 2 were located off Terrace Hall Avenue; Wells 3, 4 and 5 were existing wells located in the vicinity of the existing Phase I emergency plant off the Middlesex Turnpike and were seasonally used by the town in recent years. Wells 10 and 11 were located in the wetlands behind the Knights of Columbus, located off Lexington Street. The locations of the wells and the treatment plant are shown in Figure 1.

New transmission mains were necessary to convey untreated groundwater from Wells 1, 2, 3, 4, 5, 10 and 11 to the new plant. All wells contained high levels of iron and manganese.

New Well Development, Access & Construction

New wells were needed to replace the wells that had been contaminated beyond treatment capabilities. A town-wide groundwater exploration program was developed. Preliminary field work using test wells produced no potential sites outside of the aquifer. It was then decided to focus the search for new sources within the aquifer with the expectation that the sources would likely need treatment for iron, manganese and VOCs. The search was finally narrowed to the westerly corners of the Vine Brook aquifer, in the vicinity of the old Well 7. This well had become unusable due to elevated levels of iron and manganese that had clogged the well's intake screen beyond rehabilitation.

An exploratory test well program was developed in the early 1990s with DEP approval. In 1991, Well 11, located at the edge of the wetlands, was drilled to a depth of approximately 75 feet to serve as a permanent replacement well for the abandoned nearby Well 7. However, an additional well supply with a capacity of at least 900 gpm was needed to compensate for the other wells lost to aquifer contamination. The additional well posed several challenges in terms of hydrogeology, access and construction.

The location of high-yielding well sites was not an easy hydrogeological exercise due to the formation of the overburden aquifer. The water-rich sand and gravel layer narrowed with depth and resembled a cone in shape. After exploration drilling, the final location of Well 10 was determined. This well was located approximately 220 feet in the wetlands.

The Burlington Conservation Commission was concerned about wetlands intrusion dur-

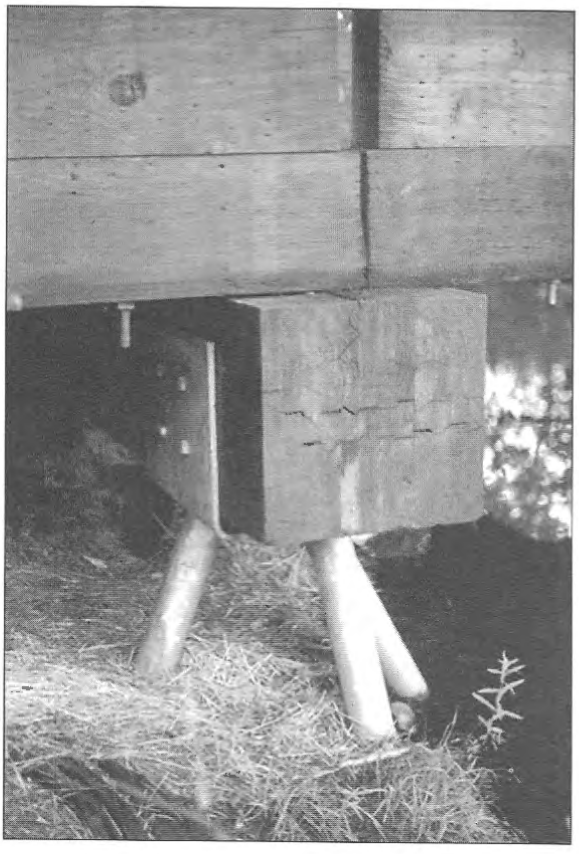

FIGURE 2. Pile cap detail (for accessway).

ing the initial field exploration program and with the long-term impacts associated with a permanent access road built using fill material, wetlands water quality degradation during construction operations and the long-term impacts on the flow path of the Vine Brook that meandered through the site from the southwest to the northeast. In addition, the commission was concerned about aesthetics since this area was a popular vantage point for bird watching. As a response, during the exploration activities a temporary boardwalk system was built to provide access to the test wells and monitoring well network.

To address the Conservation Commission's concerns and to expedite the regulatory review permitting process, it was decided to provide a permanent elevated accessway 24 feet wide and 220 feet long across the wetlands to Well 10. The accessway was designed as a wooden bridge, supported by helical piles supporting wooden beams and the accessway wooden deck was capable of supporting AASHTO H20 loading. The accessway was completed in time to be used for the well-drilling operation that required a 46-ton well drilling rig.

Each deck section was 14 feet long and was supported on a wooden beam within a galvanized beam channel welded to the pile cap of three helical piles (a pile group) on each end (see Figure 2). Pile groups were spaced 12 feet on center laterally and 14 feet on center longitudinally. Each pile was designed for a 13-ton load. Piles were battered at a typical slope of 4:1 (vertical:horizontal). Typical pile installed lengths ranged from 20 to 37 feet. Pile testing was conducted by recording the applied field torque applied per foot of pile, and converting the measured torque to ultimate load. Field torques ranged from 5,500 foot-pounds to 9,600 foot-pounds. At the end of the accessway where the drill rig was situated, an opening approximately 12 by 8 feet was framed into the deck with pile groups at each corner of the framed opening.

This innovative accessway was received favorably by all regulators including the DEP, the U.S. Army Corps of Engineers and local boards, saving valuable permit review time and enabling the construction of the well to proceed. After public bidding , the accessway construction contract was awarded in November 1996 for $420,000.

Construction progressed through the winter and the accessway was completed in March 1997 with no change orders. The contractor used a conventional backhoe with a modified arm to screw the helical piles in place. Pile lengths averaged 32 feet; pile penetration productivity averaged 150 feet per day.

In May 1997, work commenced on Well 10 construction (see Figure 3). The 46-ton drill rig was assembled in pieces on top of a load distribution frame consisting of four I-beams welded in a rectangular frame to spread the load and to avoid overstressing the capacity of the helical piles. With this arrangement, the deflection of the deck never exceeded 0.5 inch.

Well construction for the permanent 18-inch well proceeded as expected until approximately 59 feet below ground surface. A large boulder was encountered that brought drilling progress to a halt. Three options were explored, namely:

FIGURE 3. Well No. 10 construction.

- drilling and blasting the boulder;
- abandoning the partially driven well and starting over approximately 5 feet laterally from the original location; and,
- drilling through the boulder.

After reviewing the risks and costs of each option, the last option was selected. The first option was dismissed due to the risks of uncontrollable uplift of the piles. The second option was dismissed based on cost considerations. Upon authorization from the town, the contractor temporarily dismantled the pull-down rig; mobilized and assembled a dual air-water rig within 5 days and drilled through the boulder using a 12-inch bit.

The wedge of boulder removed was of a gneiss-granite composition. After its removal, the dual air-rotary rig was withdrawn and disassembled and the pull-down rig reinstalled. This rig continued down to the design depth of 90 feet. After analyzing the sand and gravel sieve characteristics, a 20-foot-long, double wire stainless steel Johnson-type screen was selected, capable of delivering up to 1,100 gpm. Well construction was completed eight days later.

New Well Testing

Wells 10 and 11 were tested together by pumping simultaneously 24 hours per day for 11 days until stabilization was reached. Water levels were monitored in these wells and in up to eight monitoring wells to determine the hydraulic conductivity and safe yield of the aquifer. (Stabilization is defined as no more than 0.1 foot drop in water elevation in a well in a 4-hour period.) At the completion of the 11 days, the water elevation in Wells 10 and 11 had dropped 10 and 17 feet below static groundwater elevations, respectively.

Water samples were collected at the completion of the testing program to determine water quality and to enable the design team to confirm iron, manganese and VOCs concentrations.

Water Mains

Approximately 10,000 feet of new cement-lined, ductile iron water main were installed to convey well water from Wells 1 and 2 (3,500 feet to the plant) and Wells 10 and 11 (6,500 feet to the plant). Four stream crossings were required and were constructed in the wetlands and in accordance with the Conservation Commission's conditions (see Figure 4). Air relief valves were located at selected high points along the pipe route to vent air from the mains during filling operations. The water flow rate from each well was controlled by throttling butterfly valves at each well station. This work was completed in approximately six months.

FIGURE 4. A view of the construction at one of the stream crossings for the new water mains.

Treatment Plant

Design Criteria. Design criteria for the plant focused on reducing concentrations of TCE to less than 5 ppb and reducing iron and manganese concentrations for aesthetic reasons. Table 2 gives the design flow rates and water quality data for the active wells. The filter loading rate was 3.5 gpm per square foot. Air stripper performance was 99.9 percent TCE removal at 48°F at 120:1 (air:water), at a maximum height of 45 feet. The treated effluent concentrations were 5 ppb for TCE, 0.30 ppm for iron and 0.05 ppm for manganese.

Plant Siting. Plant location was a critical consideration. An interim treatment facility (Phase I) with air-stripping towers and chemical feed equipment had been constructed in the mid-1980s to allow the town to use Wells 3, 4 and 5 to meet summer demands. After assessing the feasibility of building three separate plants and comparing the long-term logistics and costs associated with operating and maintaining separate facilities, it was concluded that the new plant would be sited adjacent to the interim Phase I facility near Wells 3, 4 and 5. Approximately 2 miles of new water mains would deliver water from Wells 1 and 2 via Terrace

TABLE 2.
Well Water Design Flow Rates & Water Quality Data for Burlington

Well	Design Flow (gpm)	Iron (ppm)	Manganese (ppm)	TCE (ppb)
1 & 2	850	0.34	0.68	7
3, 4 & 5	800	0.71	0.83	450
10 & 11	1,450	2.30	0.28	1,230

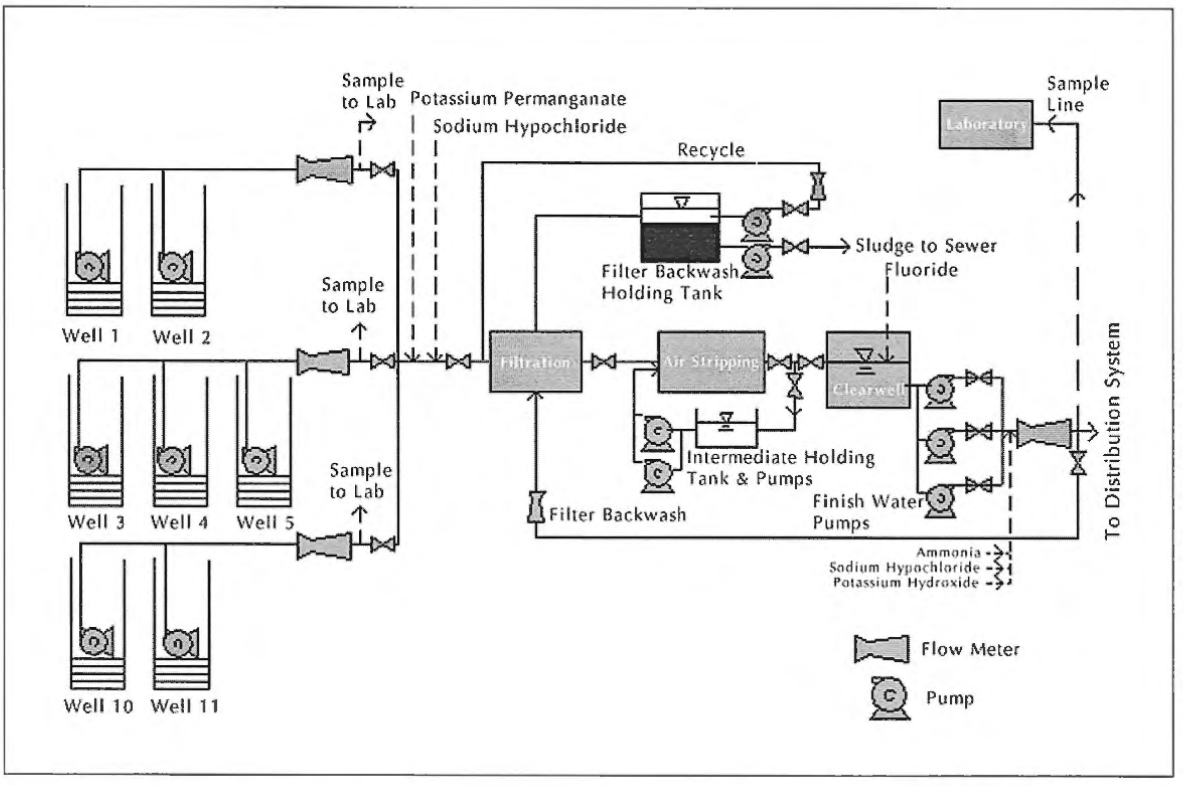

FIGURE 5. Process schematic for the Vine Brook Groundwater Treatment Plant.

Hall Avenue, and from Wells 10 and 11 via Lexington Street and Meadow Road to the plant. A new 16-inch water main running the length of the Middlesex Turnpike would be necessary to convey up to 4.5 mgd of treated water into the distribution system.

Locating the plant adjacent to the existing Phase I facility presented several design challenges. The site was bounded by wetlands on all four sides and the groundwater and soils at the site were contaminated. These issues needed close attention during the final design.

The preliminary design was reviewed by a design-review board assembled by the town, including representatives from the Town of Burlington's Department of Public Works, Conservation Commission, Building Department, Fire Department, Engineering Department and Water Department. Specific design requests included full automation, a state-of-the-art fire alarm system with separate fire, chemical leak and gas leak identification features, a full-service laboratory and provisions for plant expansion.

Treatment Plant Specifics. The new treatment facility consisted of a pre-engineered metal building with a floor plan approximately 140 by 70 feet, attached to the existing Phase I facility on town property. The interior of the Phase I plant was gutted in its entirety in order to provide room for new air blowers and motors and in order to provide space for a workshop. Since this building had never been winterized, the walls and roof were insulated and a heating system added. New louvers and doors entering into the new plant were also constructed.

The treatment process removes iron, manganese and VOCs from the groundwater (see Figure 5). Stormwater from the site is carefully directed through collection piping to a series of infiltrating pipes into the ground and the adjacent wetlands (see Figure 6).

The treatment plant has a capacity of 3,200 gpm, or approximately 4.5 mgd. Well water is pumped to the plant and is treated as follows:

- Filtration of the well water through eight vertical pressure filters to remove iron

FIGURE 6. Storm water infiltration piping outside the treatment plant.

and manganese. Each filter is 12 feet in diameter, and 12.5 feet tall. The filter media consists of 24 inches of green sand (zeolite) and 21 inches of anthracite (see Figure 7). The filters are routinely backwashed using a vigorous air/water mix.

- Air stripping to remove VOCs. Two new towers approximately 11.5 feet in diameter and 41 feet tall are used in conjunction with the two existing towers to provide a two-step air stripping process (see Figure 8).
- Chemical treatment to enhance the filtration process, using potassium as an oxidant and sodium hypochlorite as an oxidant/disinfectant.
- Further treatment for disinfection with sodium hypochlorite and ammonia, corrosion control with potassium hydroxide and fluoridation before pumping into the distribution system.
- Reserve exterior space of approximately 800 square feet and provisions for adding granular activated carbon filters if required in the future to polish the plant effluent.

The plant is fully automated and includes a sophisticated computerized control system. The supervisory control and data acquisition (SCADA) computer system allows the operating staff to control all pumps, valves, filters and air stripping flow rates, and to adjust chemical feed rates and flow rates, as well as to monitor water quality parameters of incoming and treated water quality throughout the treatment plant. The control system records and analyzes water quality data for the plant operators and for mandated reports to the DEP.

Energy efficiency is a key feature of the treatment facility. Premium efficiency motors and variable speed drives are utilized on all large pumps to maximize operating flexibility and to minimize power consumption. Moreover, the use of variable speed drives allows pumping rates and air blower flow rates to be modified while maintaining high efficiencies.

Another noteworthy design feature is the filter backwash water recycle system. Due to the elevated levels of iron and manganese, the filters require backwashing approximately once every two days. Backwashing is conducted in two steps and two different rates. Backwash rates are typically 10 to 15 gpm per square foot (gpm/sf) of filter area during the high rate step and 5 to 7 gpm/sf during the low rate step. Upon completion of a filter backwash, this wa-

FIGURE 7. Greensand filters and piping.

ter is returned to a holding-settling tank with a capacity of approximately 300,000 gallons. The top of the backwash tank is equipped with a floating intake device that allows the cleaner component of the backwash water to be reused. The settled component of the backwash water is collected at the bottom of the holding tank and pumped to the municipal sewer. With this arrangement, over 80 percent of the backwash water is recycled.

A fully equipped laboratory provides analytical instruments to measure iron, manga-

FIGURE 8. Aeration towers.

TABLE 3.
Breakdown of Lowest Bid for Treatment Plant Construction

Contract	Cost ($1,000)
Site Work	755
Dewatering	200
Concrete Work	1,200
Pre-engineered Building	625
Masonry	65
Miscellaneous Metals	44.08
Waterproofing & Sealants	41
Painting	129
Fire Protection	35
Plumbing	55
HVAC	188
Electrical	647
Instrumentation & Control	300
Filters	570
Chemical Feed Systems	375
Process Piping, Pumps & Valves	500
Laboratory	125
Landscaping	45
Hazardous Soil Disposal	20
Non-hazardous Soil Disposal	13.5
Total Bid	5,995
Engineers' Estimate	6,100

nese, color, turbidity and chlorine residual concentrations. The laboratory is also used to perform other routine analyses required for compliance with DEP regulations.

Bidding for the treatment plant work was completed in accordance with Massachusetts General Laws Chapter 30 and 149 for vertical (building) projects. Six general bids were received ranging from $5.99 to $6.6 million. The breakdown of the lowest responsive bid is listed in Table 3.

Construction Sequencing. Construction commenced in May 1997 and was completed in April 1999. Initial construction activities focused on excavation, dewatering and below-ground concrete work. The base slab of the plant measured approximately 140 by 70 feet and required 1320 cubic yards of 4,000 pounds per square inch (psi) concrete in a 3-foot thick mat (see Figure 9). The contractor requested a monolithic pour that was approved after a thorough review of the concrete subcontractors plans for concrete delivery, pumping, quality control and field operations for placement, vibration and curing. On August 28, 1997, concrete placement commenced at 6:00 in the morning and was completed at 2:00 in the afternoon. After finishing, the entire top of the mat was covered in wet burlap that was kept saturated for three days to prevent premature curing and cracking.

Construction Challenges. Dewatering contaminated groundwater was a critical issue that required the construction of a temporary on-site aeration system (see Figure 10). The groundwater table was extremely high (3 to 5 feet below grade), necessitating the installation of eight dewatering wells, each 35 feet deep, feeding a common 12-inch pumped header that was aerated before discharging to an on-site lagoon with a spillway into the wetlands. Each dewatering well pumped typically 100 gpm. The water was sampled weekly.

Geotechnical borings and test pits excavated during the design phase indicated the presence of contaminated soils on a portion of the site. To determine the extent of contamination before foundation excavation began, soil samples were obtained at various depths in a series of test pits and analyzed. With the knowledge gained, excavated soil was segregated into clean and dirty stockpiles and the contaminated soil (approximately 100 tons) was disposed off-site at a licensed facility.

Well Rehabilitation & Renovations

Sequencing of rehabilitation at well pumping stations 1, 2, 3, 4 and 5 required careful consideration. Existing Wells 3, 4 and 5 were rehabilitated first using conventional well cleaning and surging techniques to redevelop well capacity. A staggered approach was adopted to minimize the complexity of the startup process (Wells 3, 4 and 5 first; then Wells 10 and 11, and finally Wells 1 and 2). Renovations consisted of removing the existing pump, piping and valves within the station; cleaning and

FIGURE 9. Base mat concrete placement for the treatment plant.

surging the existing well to remove particulate accumulation on the submerged well screen using chemical and mechanically operated surging techniques; and installation of a new pump, piping, valves and instrumentation consisting of a remote control panel to

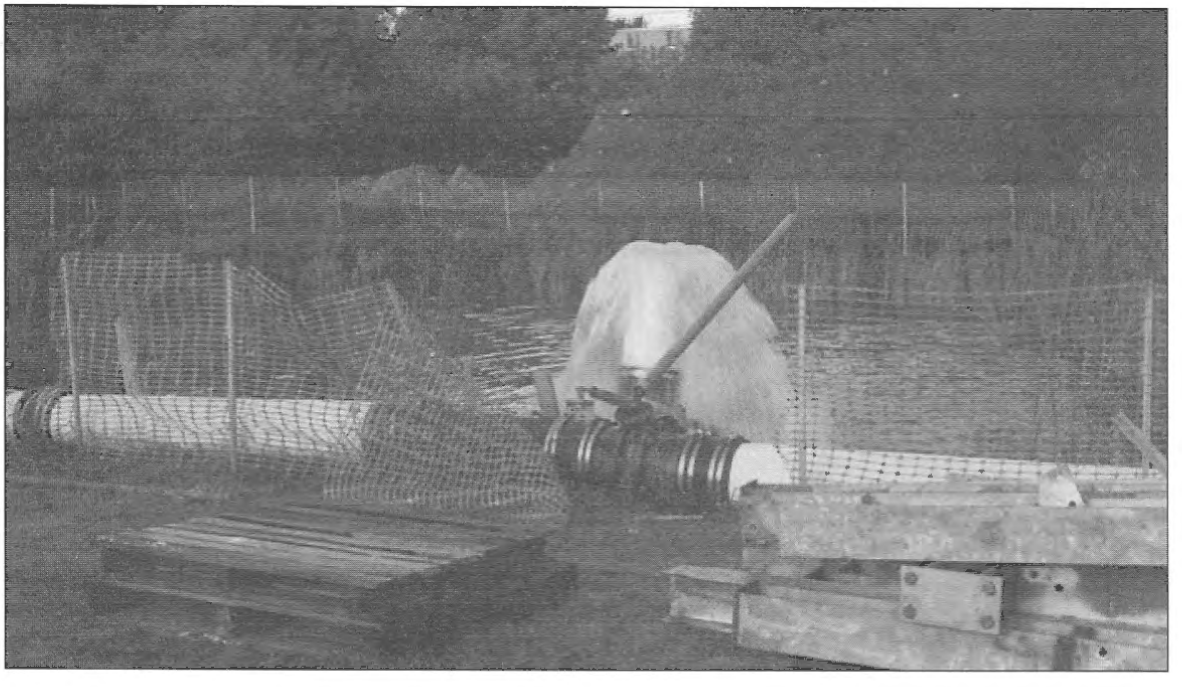

FIGURE 10. On-site dewatering effluent aeration system.

TABLE 4.
Summary of Construction Contracts

Contract	Bid ($)	Change Orders ($)	Subtotal ($)
Treatment Plant	5,995,000	126,000	6,121,000
Well 10	92,000	34,000	126,000
Well 10 Accessway	420,000		420,000
Well 11	75,000		75,000
Water Mains	728,000	28,000	756,000
Total	7,310,000	188,000	7,498,000
PRP Contribution			2,900,000
Net Town Cost			4,598,000

monitor and control well flow, level and pressure.

Testing

Startup and testing consisted of a series of dry runs of the plant processes and pumping the water to waste until the plant as a whole was operating satisfactorily and effluent water quality met the Massachusetts DEP water quality standards. Startup was an iterative process whereby individual process components, such as raw and intermediate pumping equipment, and each chemical feed system were tested separately and then in concert with other combinations of equipment until the entire plant was tested and debugged. As the plant became operational and its output capacity increased, the town's reliance on the Mill Pond Plant lessened, allowing that plant to be taken out of service for maintenance and repairs.

Automation

The treatment plant's control is accomplished by a state-of-the-art system equipped with a backup computer if the primary computer fails for any reason. With this design, the plant operator can, from any personal computer workstation in the plant, monitor the status of the wells, treatment systems and chemical feed systems, and immediately identify any audible alarms.

The plant is attended 16 hours per day. During unattended hours of operation, the plant can transmit any alarm conditions to the town's fire and police departments via a dedicated fiber-optic connection to the town's municipal network. This setup is particularly important for the fire department since it can readily distinguish a fire alarm from a chemical spill or gas leak and respond accordingly.

Training

The operating staff from the town's water department were trained for approximately four weeks in each aspect of the plant's operation. Training was specified as part of the contract specifications. In addition, a comprehensive operation and maintenance manual was prepared by the project's consulting engineer to assist the plant operators and establish a maintenance schedule for all major equipment items. In this way, equipment will be routinely inspected and maintained to prolong the life of the plant.

Costs

The treatment plant, wells and connecting water mains were constructed over a 24-month period. The costs for the complete project are listed in Table 4.

Long-Term Monitoring

Due to the importance of the Vine Brook aquifer in the town's water supply system and its sensitive location adjacent to several developed industrial and commercial office parks, the aquifer is being carefully monitored. On a quarterly basis, samples are collected from all production wells and a series of monitoring

wells around the aquifer. Data generated from this sampling regime enables the town to track trends and changes in water quality and adjust plant operations, if needed.

NOTES & ACKNOWLEDGMENTS — *This paper was the winner of BSCES's seventh annual (1999) Earnest A. Herzog technical paper competition. SEA Consultants performed the groundwater exploration program and design, and served as administrator of all construction contracts. Fiore Construction, Inc., of Leominster, Massachusetts, constructed the Well 10 accessway. Layne Christensen Co., of Dracut, Massachusetts, installedWell 10. Water main work was completed by Tom Gioioso Construction, Inc., of Rockland, Massachusetts. Charwill Construction, Inc., of Meredith, New Hampshire, constructed the treatment plant. Griffin Dewatering, Inc., of Danbury, Connecticut, performed the treatment plant dewatering work.*

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