# CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

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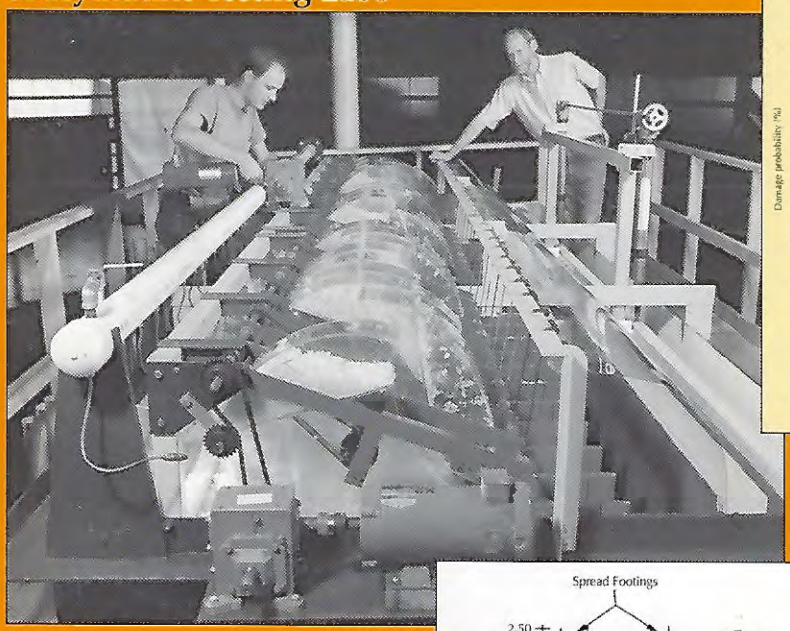
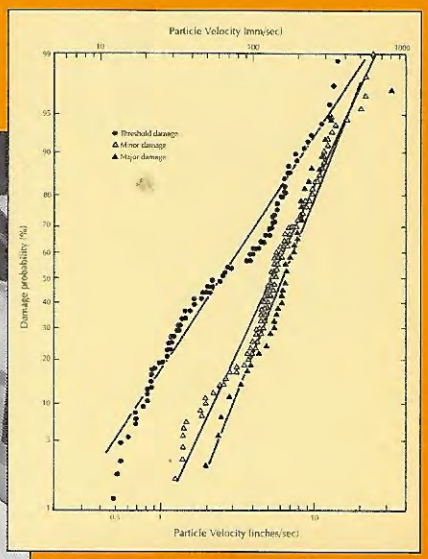
## The Role & Contributions of Hydraulic Testing Labs



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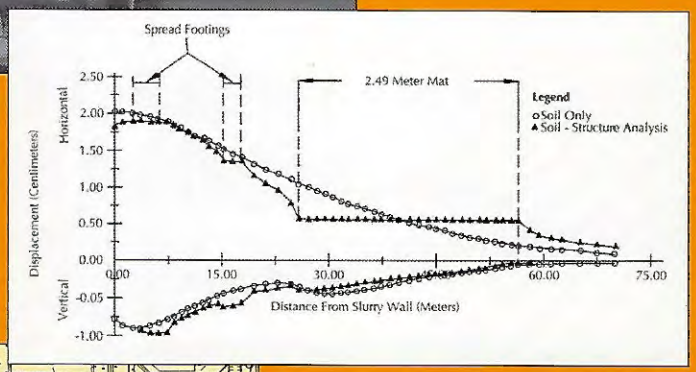


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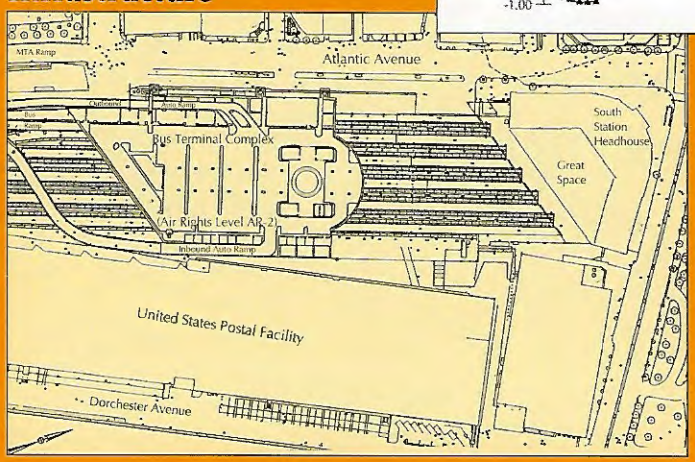


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*In Memoriam*

**For those who perished in**
**New York City, Washington, D.C.,**
**and Pennsylvania**
**on September 11, 2001**

~

**Touri Bolourchi,**
**mother-in-law of Ali Touran**

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# Carrying the Torch

It is now sixteen years since the Boston Society of Civil Engineers (BSCE) started publishing its journal (which originated in 1914) under the title of *Civil Engineering Practice*. The current journal has focused on practice-oriented papers written by practitioners, regulators and academicians to fill the gap that exists between material published in most archival journals and engineering practice. Many of the articles have been based on local projects and we have been fortunate that the large number of challenging projects in the New England region has fostered a healthy supply of interesting articles. What the journal has achieved in this respect cannot be underestimated, as many ASCE journal editors are struggling to increase the number of contributors from the industry with little success. As an example, all but one of the articles in this issue were written by industry authors, and even the sole article written by an academic is a practice-oriented case study. We are grateful to our contributors, reviewers and all the people who have solicited papers for the journal and who have helped *Civil Engineering Practice* to achieve its goals.

For the past eight years, Brian Brenner, in his capacity as the chair of *Civil Engineering Practice*'s editorial board, has been instrumental in providing leadership for the journal's direction and in remaining loyal to the journal's mission. Even though Brian stepped down from the editorial chair position in June 2001, he intends to continue his support for the journal and, indeed, has remained active in the past few months by providing feedback on articles published in the current issue and on other matters related to the journal. We at the editorial board look forward to working with Brian in the coming years and thank him for his dedication, wisdom and leadership over the past eight years. Also, we urge all our colleagues involved in exciting and innovative projects to share their knowledge and experience with our readers. Others who may be interested to serve as reviewers for submitted articles can contact me at *atouran@lynx.neu.edu*. Let us all help to keep our journal at the cutting edge of civil engineering practice.

Ali Touran
Editorial Board Chair,
*Civil Engineering Practice*

# The Role & Contributions of Hydraulic Testing Labs: Part V, Current & Future Trends

*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

Among the numerous types of model studies performed at the Alden Research Laboratory (ARL), some were related to the everyday needs of people — such as studies on potable water, sewage treatment and the disposal of unwanted flood or rain water. These studies posed interesting hydraulic problems, both in the laboratory and in the field.

## Lucite Models

In 1936, the city of Hartford, Connecticut, had a major flooding problem. The Park River, which passed through the center of the city, was backed up by the high waters of the Connecticut River. This back-up caused the serious inundation of areas of the city that had high real estate values. The city's Board of Engineers of Rivers and Harbors suggested one solution to this back-up problem — building higher walls along the Park River. However, the city felt that, due to the many large grade changes necessitated by streets crossing the river, this option would be extremely costly and cause too much burden on its citizens. The city government offered its own solution — putting the Park River in closed tunnels — and proposed paying the difference between the cost of building the tunnels versus the cost of elevating the river walls.

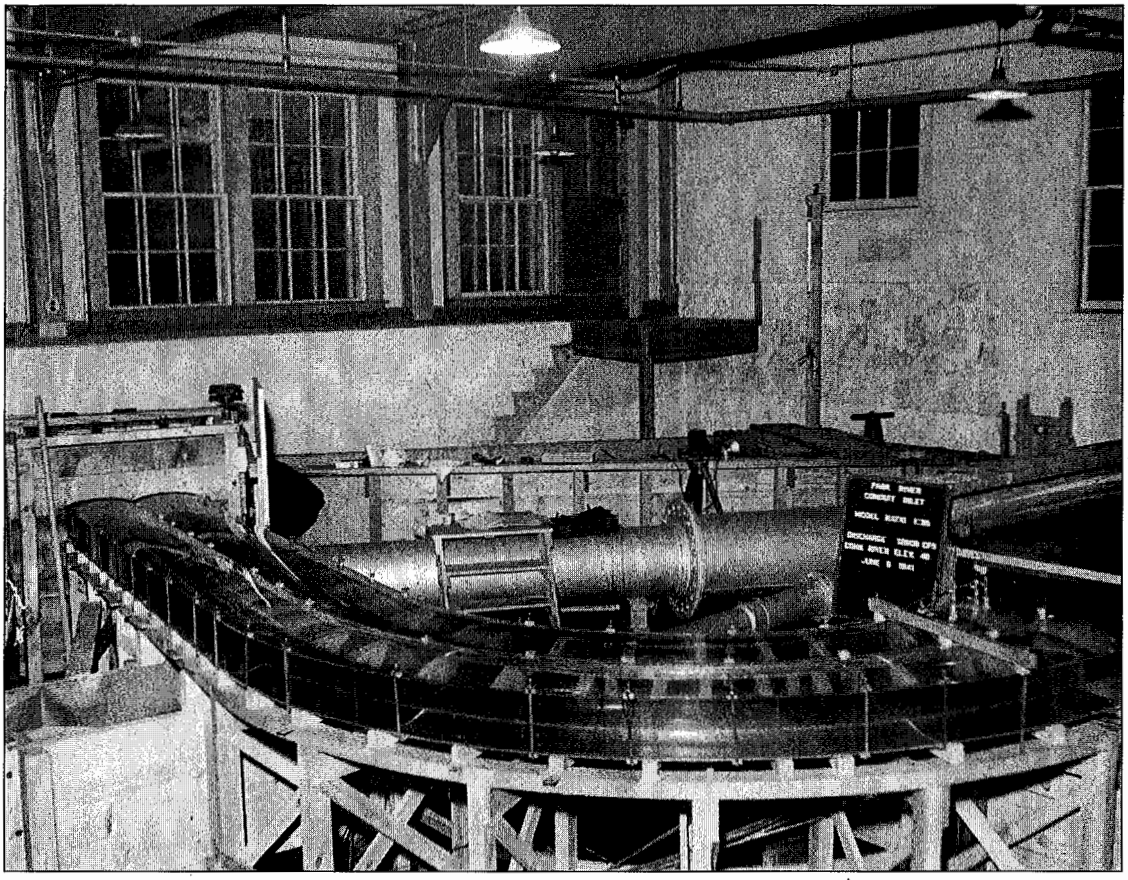

**FIGURE 1. Park River (Hartford, Connecticut) model constructed of Lucite.**

In 1941, ARL was hired to model the tunnels, starting at Bushnell Park near the capitol building and ending at the Connecticut River. The study included the intake to the tunnels as well as the discharge at the river. It was necessary to visually observe the flow patterns inside the tunnels because of the complexity of the tunnels. Because of its cost, glass was eliminated as the construction material for the model. However, ARL was aware that a clear acrylic material named Lucite had been developed in 1936. After looking at the properties of Lucite, it was decided to model the tunnels using this material (becoming the first model built at ARL using all clear material).

To construct the model, wooden forms were made in the shapes of the various sections of the tunnels (see Figure 1). Three- by 4-foot sheets of Lucite were then hung in a large insulated box and heated to 240°F using a coal-fired furnace. After the sheets became flexible, they were draped over the wooden forms, with everyone using cotton gloves to keep the surfaces as smooth as possible. Because of the fast-setting nature of the glue, all laboratory staff helped to assemble the tunnels. When the glue failed on the first sections, a new, heavier glue with the consistency of molasses was used and it was found to be satisfactory. (It is interesting to note that today, sixty years later, essentially the same technique of building acrylic models is still being used at ARL.)

The tunnels were placed on wooden supports. Since acrylic plastic can expand and contract with temperature variations, the model was attached only at the upper end and allowed to float freely on the supports.

The success of these model tests allowed the building of sections of the tunnel in, basically, the Park River channel. By 1974, a total of four more tunnel sections had replaced the downstream portion of the Park River as it passed

through Hartford, bringing the project to the junction of the North and South Branches of the Park River. At this point, ARL was asked to perform another model study. The project modeled 625 feet of the North Branch, 875 feet of the South Branch, 750 feet of the tunnel downstream of the junction and 600 feet of an auxiliary conduit that drained directly to the Connecticut River. Since the auxiliary conduit was lower than the tunnels, a drop structure with an air vent was necessary at the junction. The model was used to design a junction with satisfactory flow distribution and minimal head losses.

## City of New Orleans Jefferson Parish Flood Control

Potential flooding was also of concern in New Orleans. The city is located between the Mississippi River on the south and Lake Pontchartrain on the north. Hydraulically, it is an interesting city. It is one of only three sites in the United States that has a mean land elevation below sea level. Death Valley, California, is 282 feet below sea level, the Salton Sea in California is 236 feet below sea level and New Orleans is 8 feet below sea level. In addition, the city experiences an average annual rainfall of 58 inches, the largest of any major city in the United States. When you consider that the city has an area of 58,785 acres, it means that the city needs to pump approximately 96.5 billion gallons of rainwater a year out of the city as the result of rainfall alone.

Although New Orleans was settled in 1718 on the high ground near the Mississippi River, it was not until 1893 that city leaders started planning systems for drinking water, sewage and rainfall disposal. The system designed for rain water removal consisted of subsurface drains that collect the rain water and discharge it into either open or covered canals. The city has over 90 miles of covered reinforced concrete canals ranging from 8 to 28 feet wide, and more than 82 miles of open canals ranging from 6 to 125 feet wide. The capacity of all the pumps removing the water from the city is approximately 24 billion gallons per day. The city's Pumping Station No. 6 is probably the largest drainage pumping station in the world, with a seven-pump capacity of approximately 6.5 billion gallons per day.

Starting in 1981, ARL conducted studies related to the New Orleans area due to proposed expansions of the drainage system by the New Orleans Sewage and Water Board and the adjacent Jefferson Parish Department of Public Works. Some models involved pumping station expansions, other models investigated canal junctions and also evaluated increased flow conditions in specific canals (see Figures 2 and 3 on the next page). ARL also investigated the redesign of wave protection and silt minimization of Bucktown Harbor in Lake Pontchartrain.

Among others, two pumping stations operated by the New Orleans Sewage and Water Board — the Broad Street Pumping Station and Pumping Station No. 6 — were studied. Twenty-seven percent of Broad Street Station flow was supplied by Melpomene Canal and seventy-three percent by the Broad Street Canal. The study of enlarging the Broad Street Station by adding two pumps with a combined capacity of 1,200 cubic feet per second (cfs) included modeling both the suction and discharge basins. The model indicated the need for a number of guide walls in the suction basin to improve flow conditions. In addition, the pump hood of the most westerly pump was rotated 90 degrees to improve the performance of the two new pumps.

The study on Pumping Station No. 6 (located on the Metairie Outfall Canal) examined expanding its capacity by 2,160 cfs (from 7,780 cfs). The flow from the station is conveyed to Lake Pontchartrain via the 2-mile-long Metairie Canal. The station expansion would require that the transition from the station to Metairie Canal be widened. Since a single-track Southern Railroad trestle was located approximately 70 feet downstream of the station, the trestle needed to be lengthened. The Southern Railroad also planned to build another parallel track, so these factors had to be taken into account in this study. Besides measuring velocities at the pumps, velocities were also measured in the transition area and in the canal area to determine the potential for scour.

Expansion of Pumping Station Nos. 1, 2, 3 and 4, and various canal improvements in Jefferson Parish were also studied at ARL. Pumping Station No.1, located on Bonnabel

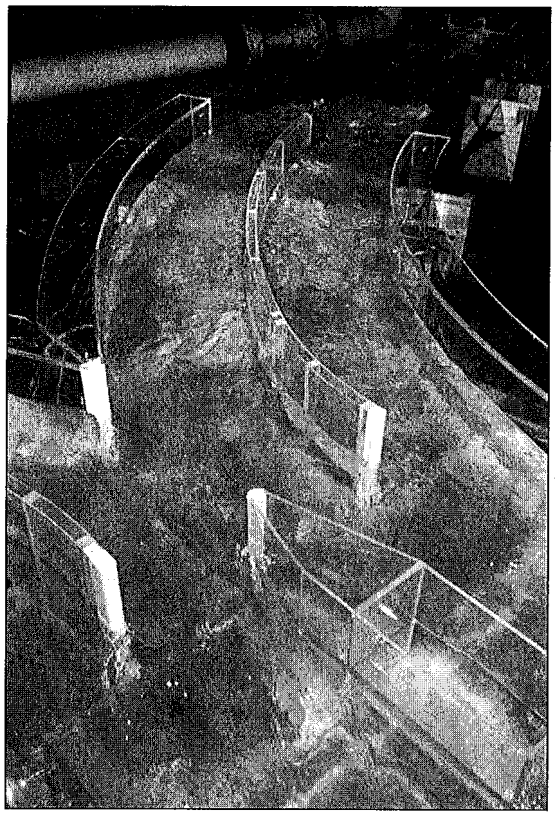

**FIGURE 2. Flow in flood control canals in New Orleans (model).**

Canal, was to be expanded from a capacity of 800 cfs to 2,600 or 3,600 cfs, depending on the final number of pumps to be used. The bridge upstream of the station was modified in the model using flow deflectors to improve the flow distribution into the station. Pumping Station No. 2 was expanded from four to six pumps with the capacity increasing from 1,650 to 2,950 cfs. Curtain walls and floor cones were used with the pumps, and a modified bridge configuration was investigated upstream of the station. A replacement for Pumping Station No. 3, located at the northern end of Elmwood Canal near Lake Pontchartrain, would have a capacity of 3,400 cfs. Finally, Pumping Station No. 4, on Duncan Canal, would increase in capacity from 1,250 to 3,600 or 4,600 cfs. The junction of Duncan Canal with Canal No. 7, including a bridge near the junction, was also studied. For all stations, the pump approach velocities and vortexing were recorded and remedied to maximize station outflow.

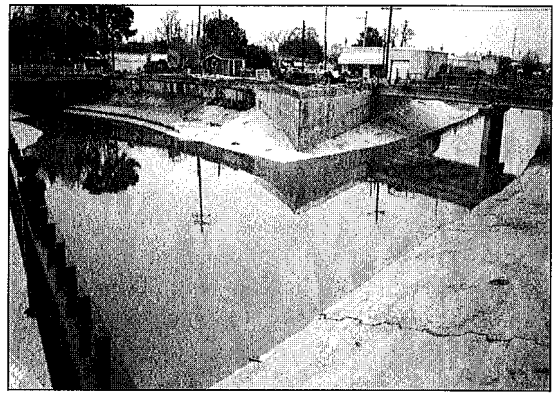

**FIGURE 3. Flood control canals in New Orleans (prototype).**

If all the proposed changes were made, the total capacity of the four Jefferson Parish pumping stations would be 14,550 cfs. This amount represented a significant improvement over the 1,760 cfs total capacity in 1926 and the total capacity of 4,990 in 1970.

A canal junction for Jefferson Parish was also studied. Soniat Canal flows north and discharges into Canal No. 3. Two branches of Canal No. 6 discharge into Soniat Canal near the Airline Highway Bridge. At this point, Soniat Canal takes a northwestern jog, a northeastern jog and then a northern return after the bridge. The model was used to study the proposed realignment of the junction and bridge pier and to develop improvements that would maximize flow through this area.

To increase the capacity of the Broad Street Canal, a proposed parallel canal would be built, starting where Napoleon and Fontainbleau canals joined Broad Street Canal. Approximately 600 feet downstream of the junction, General Taylor Canal intersected both sides of Broad Street Canal. In the vicinity of Washington Street, the two Broad Street canals would transition into a double box canal, which would continue to Pumping Station No. 1. This transition had a steep downward slope in addition to some curves.

A model study investigated the reconstruction of the new canal intersections. One of the interesting aspects of this study was the presence of hydraulic jumps in the Napoleon and Fontainbleau canals at the intersection. At this point, an open area between the existing and

the proposed Broad Street canals was all that was required to balance the flow in both canals.

ARL was hired to model study two intersections, at Palmetto and Hoey canals, that were to be affected by the widening of Metairie Canal. Two bridges also crossed Metairie Canal near the junctions, and the effect of the bridge piers on flow and head loss had to be evaluated. Several different intersection schemes were tested to find the minimum head loss at each intersection. A number of proposed bridge pier designs were studied to minimize costs and head loss. ARL also studied an improvement to the 17th Street Canal. The study to quantify head loss involved the junction of the 17th Street Canal and Hoey Canal, the Hoey Canal and Geisenheimer Canal junction, the transition between the Bypass and Upper Protection canals, and the support pier at Airline Highway Bridge.

ARL also performed two other studies for New Orleans. One study evaluated the head losses and canal elevations for Palmetto Canal crossings between Airline Highway and Eagle Street. There were a total of four flow obstructions in this reach of the canal: Airline Highway, a railroad crossing, a 50-inch-diameter water main and the Eagle Street bridge. Twelve different modifications were made in the model to prevent overtopping. The second model evaluated the redesign of the Carrollton Avenue Box Culvert to accommodate increased flow.

The ARL studies for the New Orleans area point out the complexity of the city's drainage system and how difficult it is to expand it. Increasing flow capacities at one location necessitated changes in other areas, sometimes precipitating a domino effect. Trying to understand such changes without a physical model would be almost impossible.

## Sewage Treatment Plants, Wastewater Improvement & Potable Water Facilities

Sewage treatment pumping plants sometimes have intake problems similar to those of the above-mentioned pumping stations. ARL has modeled and evaluated a number of these pumping facilities, and found that these problems and solutions are similar to any pump intake structure. Ideally, the flow needs to be uniform approaching the pumps and no detrimental surface or subsurface vortices should be allowed.

A proposed diversion structure in Houston, Texas, was modeled. This structure was designed to have uncontrolled influent lines of 84-, 96- and 120-inch diameters and sluice gate controlled effluent lines of 78-, 84- and 144-inch diameters. The structure, located near the intersection of Lockwood Drive and Clinton Road, would intercept a portion of the sewage normally going to the Northside Wastewater Treatment Plant and would divert it to a new 69th Street Wastewater Treatment Plant. Three different flow splits were studied. Since the influent varied from domestic to industrial, mixing was required in the diversion structure. The model tested the mixing efficiency by using Rhodamine WT dye injected in one of the influent lines and measuring the dye, using a fluorometer, in all three effluent lines. Discharge coefficients for all control gates at the entrance to each effluent lines were also determined in the model.

In addition to the diversion structure model, ARL also performed a mathematical transient model study for a section of the 69th Street Wastewater Treatment Plant. The model simulated the flow system from the lift station to the first step reactors and included the following components: grit chambers, splitter box, feed forward line and the connecting channels. The objective of the study was to determine the rate of change of water elevation in the splitter box as the result of changes in the lift pump operation. This information, in addition to the control valve information, would allow automatic operation and flow distribution.

Pump transient studies (mathematical modeling) have been conducted by ARL staff for a number of years. These studies have mostly been applied to transients in power station circulating water lines. Other studies have investigated pipe breaks and valve closure times.

A fluorometer and Rhodamine WT dye were also used in the study of a chlorine contact tank. Maximum chlorine contact time and no shortcircuiting were the objectives of this study. Many modifications using walls and baffles were tried in the model but, in the final analysis, neither the client nor ARL recommended any of the modifications used in the model.

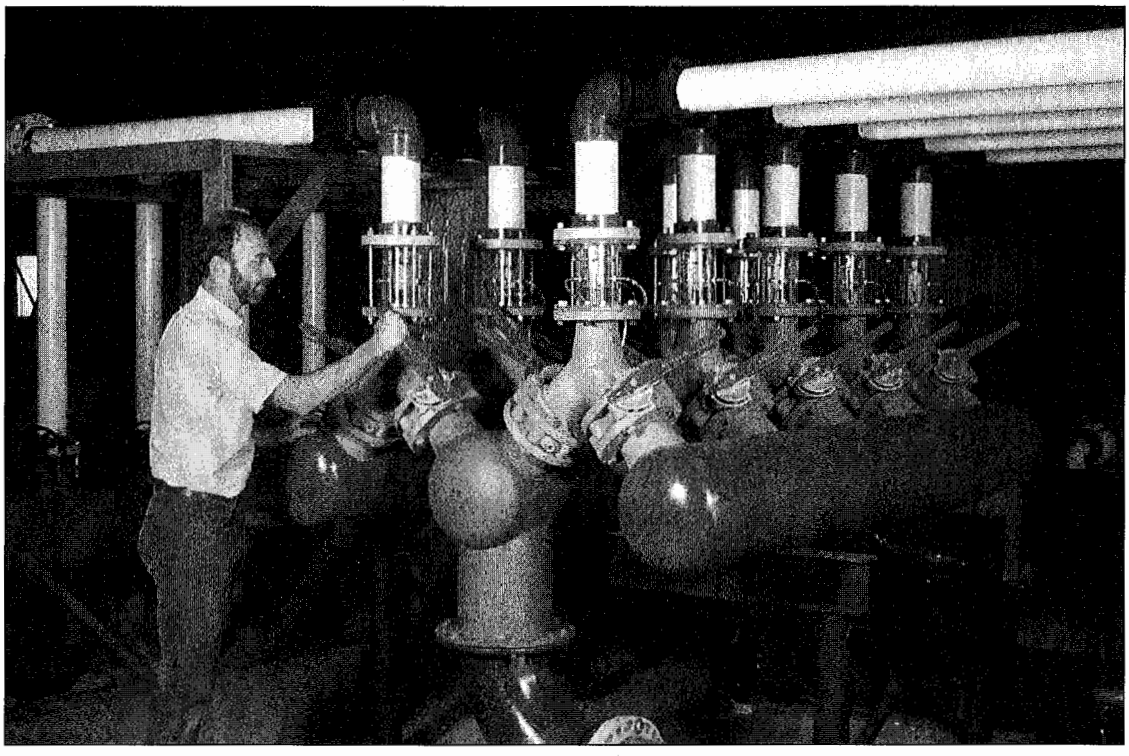

**FIGURE 4. Pump manifold for the North Main Pumping Station (Boston Harbor Clean-Up Project).**

Shortcircuiting chlorinated water in a circular filtered water reservoir was studied at ARL. Both the intake and discharge pipes were located close to each other on the tank's periphery. Without tank baffling, the inflow almost went directly to the discharge, thereby minimizing the retention time in the reservoir. Since the purpose of the tank was to maximize retention time, various baffling schemes were tried with success in the model.

In a study of stacked secondary clarifiers for the Deer Island Wastewater Treatment Plant for the Boston Massachusetts Harbor Clean-Up Project, impact plates with holes were used in front of the incoming water jet to quickly slow down and uniformly distribute the inflow water. The purpose of this study was to develop methods to slow down the inflow in order to precipitate out the remaining particles in the water, utilizing the whole length of the clarifier. Minimizing re-entrainment of sludge as it was scraped into the sludge hopper below the entrance flows was also of major concern.

In another study for the Boston Harbor Clean-Up Project, ten pumps were to be replaced with higher capacity ones (110 million gallons per day at a 140-foot head) without changing the existing complex piping configuration in the approach to the pumps (see Figure 4). Because of bends, Tee connections, wye and reducers in the suction side manifolds, undesirable velocity distributions and swirl could adversely affect the pumps. There were also numerous combinations of pump operations that could affect overall performance. A 1:12 scale model indicated high swirl intensities and nonuniform velocities at the pump suctions. Vanes were determined to reduce the swirl and improve the velocity distribution.

Evaluation of some site-specific equipment is sometimes only possible by using a physical model. One such model looked at an energy dissipating scheme where flow from the plant discharged into a long vertical shaft. Mechanical methods, as well as a turbine, were modeled to dissipate the energy. Methods to minimize

air bubbles at the discharge from the tunnel were studied as part of the project.

Launders (long perforated pipes used for flow distribution) at the Wanaque Water Filtration Plant of the New Jersey District Water Supply Commission were studied for mixing and uniform flow distribution. The prototype launders were 100-foot-long pipes made up of 50 feet of 24-inch-diameter pipe, followed by 25 feet of 18-inch-diameter pipe, and ending with 25 feet of 14-inch-diameter pipe. The water flows from eight of these launders into settling basins 100 high by 100 wide by 21 feet deep as part of the filtering process. In this study, the distribution was found to be within acceptable limits and no changes to the design were recommended.

An interesting study related to sewage evaluated the use of a proposed Shockoe Creek Retention Basin for the city of Richmond, Virginia. Diversion structures, which direct combined sewage and storm runoff flow to the retention basin, were to be installed in existing arch and box sewer tunnels. Normal dry weather sewage will pass through screens in the diversion structures, which remove the large debris, and flow directly to the treatment plant. In case of rainy weather, when the combined sewage and runoff levels nearly reach the point when the overflow would go into the James River, gates would be activated to contain and divert the flow into the proposed retention basin. These flows were to be held until the rains abated and the sewage and retained flows could be treated in the plant. If the capacity of the retention basin were reached, the gating would be activated to divert combined flows to the river. Model design changes were made to the initial design to maximize the volume retained in the basin and to keep water elevations within bounds to prevent the back-up of sewage in the incoming lines.

The problems and solutions associated with potable water pump sumps are no different than those for any other pump sumps. Filtering of potable water is a long-standing art. However, new equipment is always coming onto the market. Full-scale studies were conducted on a section of an underdrain tile filter to determine pressure and flow variations along the laterals during backflushing. With this information, ARL developed a mathematical model to predict the hydraulic performance of laterals of various lengths that could be used by the client in designs.

This compilation of projects in the area of potable water, sewage and drainage represents just a small amount of the model testing conducted in these areas benefiting clients interested in improving plant or equipment performance or in making sure a new plant or a new piece of equipment would work properly.

## First Wave Study

The first wave study at ARL was performed in 1961 for the enlargement of a barge harbor and its inlet channel located at Barbers Point on the southwestern tip of the island of Oahu in Hawaii. The basin measured approximately 520 by 630 feet and had a 220-foot-wide channel 1,000 feet long. The basin had been carved of coralline limestone and had steep slopes of 1:4.

The model waves were produced by a 20-foot-long paddle driven by a variable speed motor. The configuration of this wave generator paddle created a large amplitude at the top and a smaller amplitude at the bottom to produce an oscillating wave motion. Data on wave amplitude were recorded on photographic paper using resistance-type gauges in the model. These gauges consisted of two 0.125-inch-diameter stainless steel rods spaced 0.5 inches apart with 0.4 volts potential applied to the electrodes. In addition to measuring wave amplitude, observations of the movement of bottom sediment in the basin were made and photographed. The model sediment was a natural black bitumen material of coarse sand size called Gilsonite (found in Utah and Colorado).

Approximately 120 tests, including the use of breakwaters at the entrance of the channel, were made. The best solution to minimizing wave heights near the barge wharf was found to be the use of "beaches" around the basin. These "beaches" were actually slopes of 5:1 carved out of the limestone inside the basin, which caused the attenuation of the waves entering the basin and gave the most stable basin for all possible size waves encountered in this area.

## Power Plant Wave Studies

ARL's second wave study, conducted two years later, examined a sea water intake for the

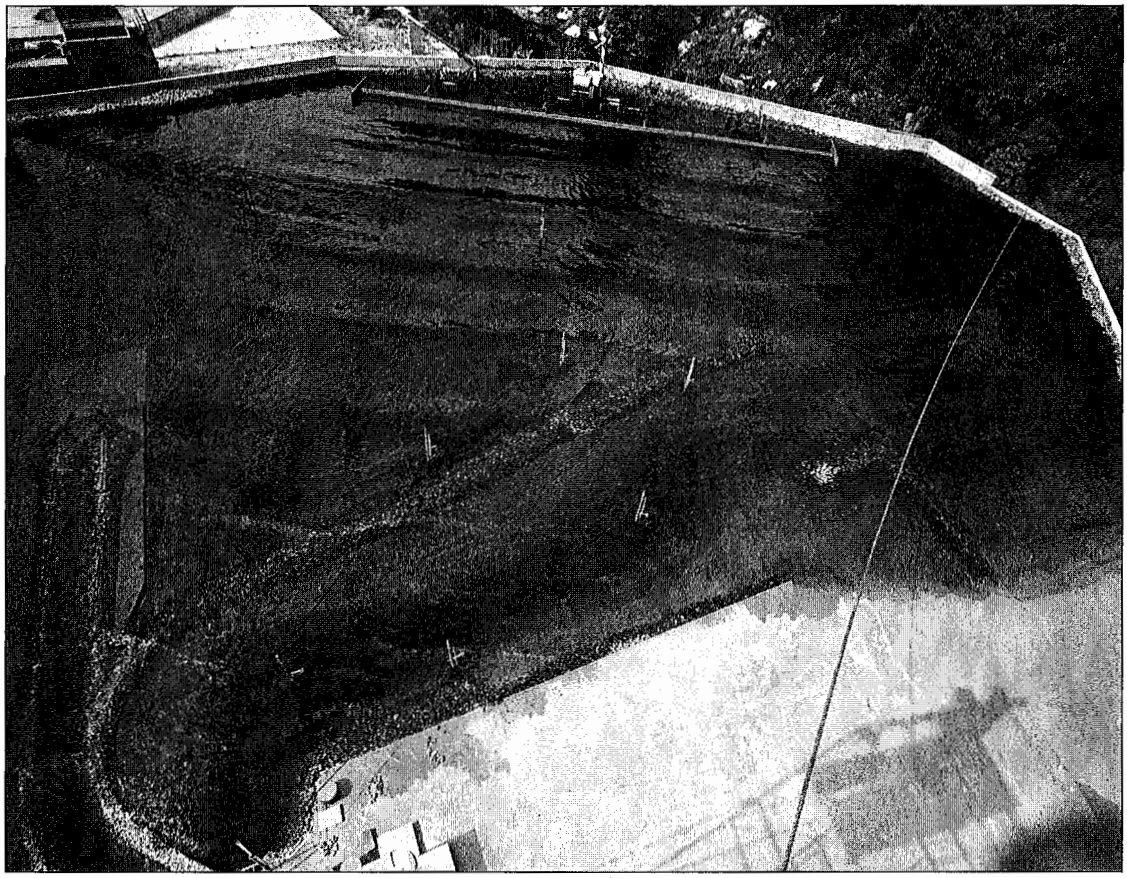

**FIGURE 5. Pilgrim Nuclear Power Station wave model.**

Palo Seco Steam Plant in Puerto Rico. The testing investigated the wave action in an existing intake with jetties on both sides comparing various proposed jetty and breakwater designs.

The same wave generator used in the first study was modified for this study. The mechanical linkages were changed to give translational and rotational movement to produce the waves, and a filter was also installed in front of the wave generator to smooth out the wave shape. Since this was a sea application, the wave reflection from the model walls was minimized by using stones sloped against each model wall.

Fourteen resistance-type electrodes used in the previous study were scattered throughout the model. In addition to the same recorder used in the previous study, another, more accurate recorder was obtained and used in the testing.

There were two main results from this study. For the conditions given to ARL, a new jetty addition was designed that decreased the agitation at the intake forebay from 5.6 to 0.80 feet. This modification was better than any proposed breakwater structure. The second result was a warning that longer period waves produced high waves in the intake. The report strongly suggested that field studies be undertaken to document wave periods at the site.

As in the past, one study would lead to another study concerned with the same plant or fluid mechanic phenomena. In 1967, ARL conducted a 1:50 scale thermal discharge model for the proposed Pilgrim Nuclear Power Station. The station is located on Massachusetts Bay and is exposed to ocean storm waves. This model basin was available to do wave run-up studies when the Nuclear Regulatory Commission (NRC) required maximum probable flood (MPF) studies to be conducted to ensure plant safety. Building on the experience of the previous two studies, the third wave study exam-

ined the run-up on the buildings at the Pilgrim Nuclear Plant (see Figure 5). To be conservative, the model tests were conducted with 19-foot waves and periods ranging from 8 to 10 seconds. The same wave generator used in the other studies was modified to a length of 49 feet, and the input power was increased to compensate for the increased load.

Run-up on the reactor buildings was measured using a water-finding paste commonly used in the petroleum industry. Prior to the test, each building was given a coating of the paste. After testing, the areas where water had touched the buildings had changed color, and these locations and elevations were measured and recorded. Under worst conditions, the run-up on the buildings did not exceed 6 inches (prototype).

An interesting story evolved from this project. Coarse riprap was used on the shores near the power plant during the model study to help dissipate incoming waves. The field contract was awarded to a firm new to this type of work. In their enthusiasm to do a good job, they laid all the stones on the slopes so that all of the flat portions of the stones were on top and made a nice flat slope. When ARL was shown photographs of the job, they agreed it looked very nice but it probably did not do very much to dissipate wave energy or reduce wave run-up.

## Lake Michigan Wave Studies

The next two sites for wave studies were on Lake Michigan. One of these was for the D.C. Cook Nuclear Plant in Bridgman, Michigan. Using a number of models, studies were conducted to examine:

- wave-induced loading on the submerged offshore intake and discharge structures;
- discharge jet scour with and without wave action and bottom protection schemes;
- the internal hydraulics of the multislotted jet discharge structures; and,
- scour around the cooling water intake pipes.

A 1:75 scale model, originally constructed for thermal studies, was used for the scour studies. A hardwood sawdust bed was added to the model and it was used again to qualitatively examine scour due to the slotted jet discharge (see Figure 6). When it appeared that there would be scour problems, a suitable protective bed was developed in a 1:20 scale discharge model that was tested in a 4- by 4- by 40-foot-long flume. This flume had a horizontal displacement type wave generator operated by a variable speed motor driven piston. Stainless steel shavings were placed at the far end of the flume to absorb the wave energy and to prevent the reflection of the waves from the end plate. Wave heights were measured using the same two rod resistance-type gauges used in previous studies.

Dye was used to observe the action of the jet discharges. Due to waves, a cyclic up-and-down motion of the jets was observed that caused lake bed erosion around the structure. After numerous tests, it was found that a mixture of 5- to 500-pound stones (prototype) — with roughly 63 percent of the mixture larger than 70 pounds — was needed to prevent scouring caused by any size of waves anticipated in the area.

The wave flume was also used to investigate wave-induced forces on 1:50 scale models of the intake and discharge structures. The structures were made so they could be rotated to simulate waves approaching from different directions. Flush-mounted pressure transducers, connected to an amplifier and recorder, were installed on the top and bottom of each structure. By slightly varying the period of the wave, the waves were made to break at different positions over the structures. The transducers indicated that a bending moment was created on the roofs of the submerged structures when the waves passed over them. The maximum pressure differential between the top and bottom of either structure was determined to be 9.5 feet of water.

The D.C. Cook study also involved an investigation of the discharge structure hydraulics using a 1:75 scale model. These tests involved measuring the discharge velocity distribution and altering the approach piping to make the distribution as uniform as possible.

In the spring of 1973 at the D.C. Cook plant, it was discovered that sections of the three

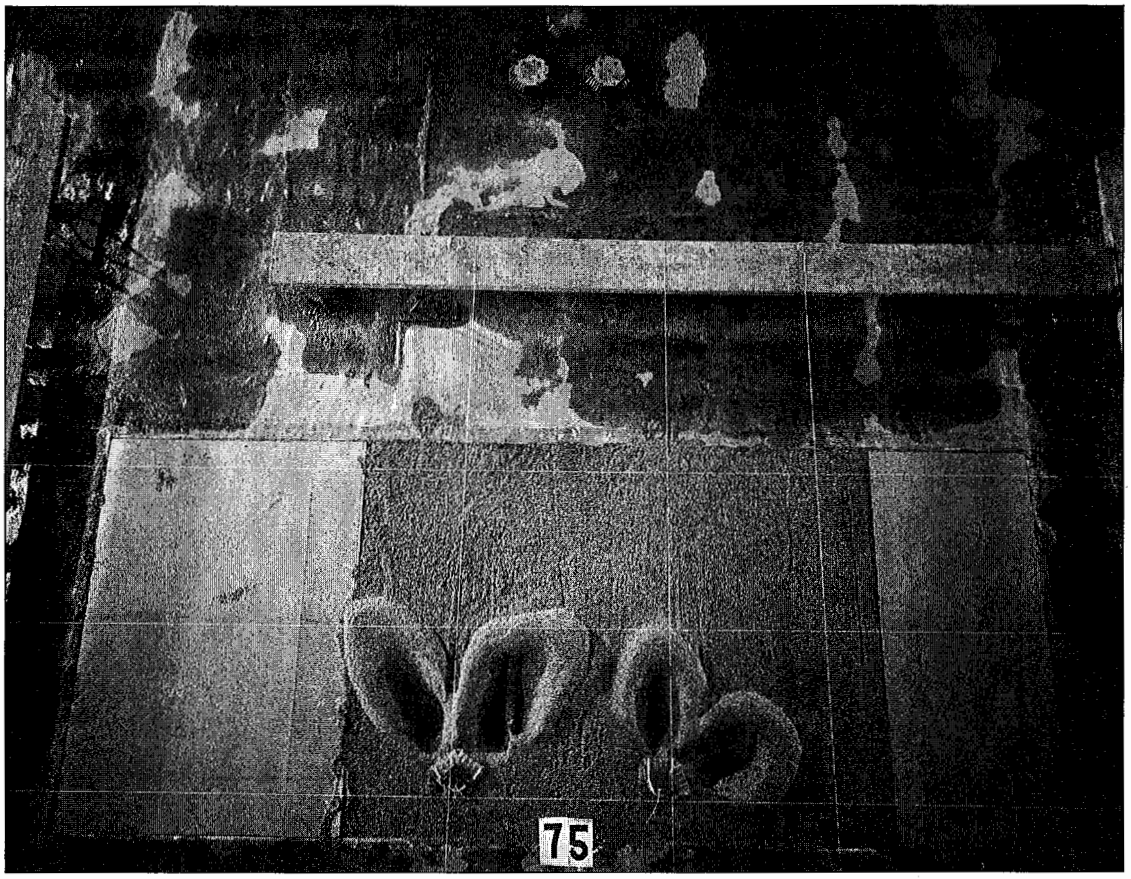


**FIGURE 6. Lake bottom erosion at discharge structures (D.C. Cook plant study).**

16-foot-diameter cooling water intake pipes, covered with 3 feet of dredged sand and clay, had been exposed and some seams and pipe bands had failed. In addition, some scouring had occurred at the edge of the 5- to 150-pound riprap near the intake structures. A 1:60 scale model was constructed for this study and installed in the flume mentioned above. The flume was modified to include a centrifugal pump that allowed the modeling of currents as well as waves. Tests were performed using polystyrene injection molding material, hardwood sawdust and sand as bed materials.

To obtain a better quantitative prediction of scour, a 1:25 scale model was tested in an outdoor facility using sand as the bed material (see Figure 7). The test results indicated that the pipe protection scheme should include a 0.5-foot layer of sand over the pipe, followed by filter cloth and a 2-foot layer of 5- to 150-pound riprap. The models also indicated that further erosion around the intake structures would not continue.

The second site on Lake Michigan for which ARL conducted a wave study was the J.H. Campbell Electric Generating Station, on the eastern shores of the lake near Port Sheldon, Michigan. The plant's owner was proposing a plant size increase that would have enlarged the condenser cooling water from 303,000 to as much as 1,060,000 gallons per minute. The plant inflow from Lake Michigan was through a narrow inlet into a small lake called Pigeon Lake. The discharge was through a long narrow canal back to Lake Michigan. In the new scheme, the discharge would be through underground pipes to offshore structures, and the inlet to Pigeon Lake would be enlarged to maintain an average velocity of 1 foot per second. This enlargement of the inlet would expose Pigeon Lake to a more severe wave climate than presently ex-

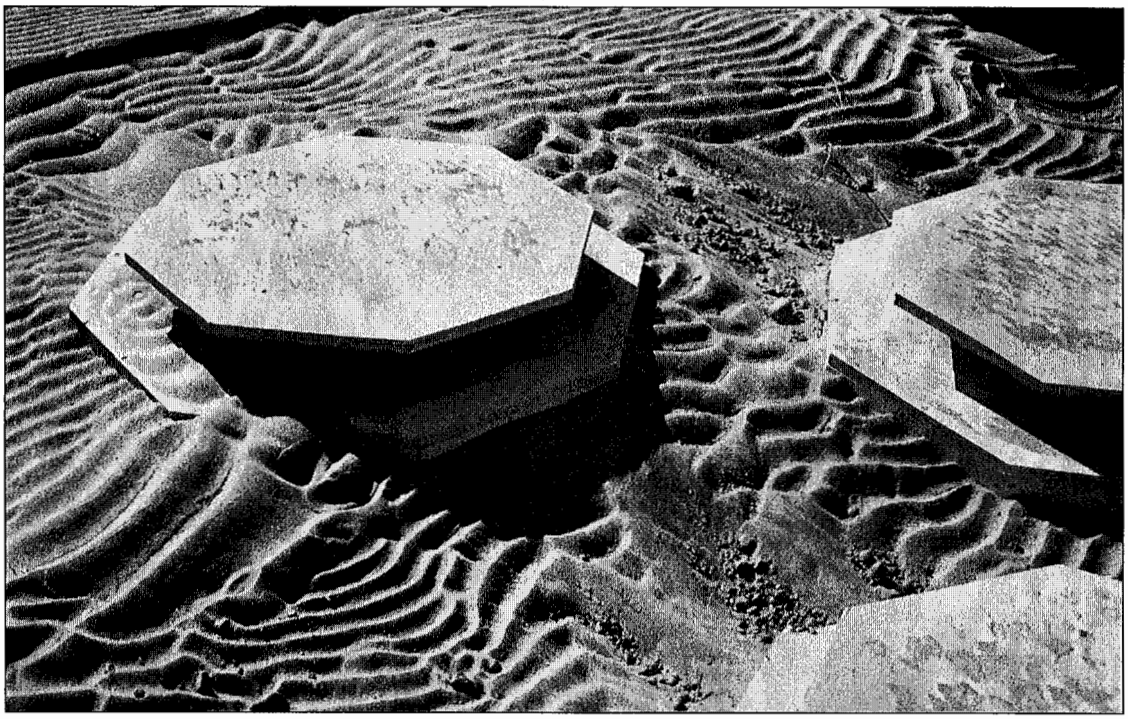

**FIGURE 7. Model study of wave-induced erosion at the D.C. Cook power plant intakes.**

isted and could cause severe shoaling problems in the inlet area.

After various breakwater studies, a sloping rubble-mounded breakwater on the interior of the existing caisson jetties was determined to have the most effective wave damping characteristics. This design also minimized interference with navigation and maximized the ease of construction.

## Great Lakes Elevation Datum

In all the studies conducted at ARL on the Great Lakes, it was interesting to note that water depths and lake bottom elevations were referred to the International Great Lakes Datum (IGLD) of 1955. The datum zero was established at Pointe-au-Père (Father Point), Québec, located approximately 7 miles northeast of Rimouski on the southern shores of the St. Lawrence River. This datum was the first established as a joint venture between the United States and Canada to coordinate hydrological information in the more than 295,000-square-mile Great Lakes–St. Lawrence River System. Prior to this, many references were used in both countries. In the United States, such references were the Plane of 1873, changed to Low Water Datum in 1894 (Lake Superior), Mean Level 1860-1870 (Lake Erie), Mean Lake Level 1860-1875 (used by all lakes), Standard Low Water (adopted 1901), 1903 Datum, Improved Planes (adopted 1916), Low Water Datum (adopted 1932) and the 1935 Datum.

Because the earth's crust moves with respect to sea level and because the rate of movement is not uniform throughout the area, the U.S.–Canada coordinating committee proposed that the datum must be adjusted every 25 to 35 years. The current datum is called the IGLD of 1985 and was implemented in January 1992. The reference zero of this new datum was moved to Rimouski, Québec, due to deterioration of Pointe-au-Père.

## Port Engineering

A multiphase wave study was undertaken in 1976 for the expansion of the port of Champerico, Guatemala. The port's over-100-year-old steel pile pier is located on the Pacific Ocean approximately 25 miles southeast of the Mexico-Guatemala border. The Pacific alluvial

plane in the vicinity of Champerico extends east from the coastline to the Sierra Madre mountain range, approximately 31 miles away. Sand and mud are carried by many rivers, especially the Rio Samala to the east and the Rio Ocosito to the west of Champerico, and create bars, deltas and shoals in the area. This shoaling material is especially prevalent during the rainy season from May to October.

For a long time, Champerico was unable to be directly serviced by large ships. These ships had to anchor about 1 mile offshore, and the cargoes were transferred to and from the pier with small boats (lighters). Needless to say, this method was inefficient and sometimes dangerous.

Three different models were used in this study with four different two-dimensional breakwater models tested in the wave flume in ARL Building 1. These breakwaters consisted of:

- a precast gravity concrete caisson with flat vertical walls;
- a cellular steel gravity sheet pile structure;
- a rubble-mound structure with concrete dolosse armor units; and
- a rubble-mound with stone armor.

The four schemes were investigated for wave overtopping, wave run-up, wave energy transmission and induced structural damage. In this study, capacitance-type gauges were used to measure wave heights.

A second fixed bed model was used to investigate expected wave height distribution and potential shoaling for two preliminary harbor concepts. The results obtained in the first model were implemented in this test program.

The third model employed anthracite coal as the bed material in a movable bed model (see Figure 8). Three different harbor schemes were studied to evaluate the stability of the near-shore zone, the potential shoaling, the maintenance problems of the harbor and channel, and the long-term shoaling on the operational life of the existing pier.

Based on the hydraulic model tests, soil conditions, material handling, construction costs, economic return on investment and possible future expansion, it was possible to devise plans to enlarge the Port of Champerico. In addition, the model predicted that it would take ten years after construction commenced for the breakwater entrance to begin silting in. After that, about 91,500 cubic yards of material would have to be trapped and removed yearly, a manageable task.

## Floating Bridge Failure & Forensic Engineering

The area around Seattle, Washington, contains a number of floating bridges. On the night of February 13, 1979, a storm with sustained winds of 75 miles per hour and gusts of up to 100 miles per hour caused approximately 4,000 feet of the 6,600-foot-long Hood Canal floating bridge to sink. This bridge, west of Seattle on Washington State Route 104 near Port Gamble and approximately 43 miles north of the infamous Tacoma Narrows (Galloping Gertie) Bridge, was mounted on concrete box pontoons 360 feet long, 50 feet wide and 15 feet deep. A 600-foot draw section in the middle of the bridge could be opened for navigation purposes. The bridge was held in place by a system of prestressed stainless steel cables connected to anchors on both sides of the bridge.

As the result of the accident, the company that insured the bridge decided to investigate the cause of failure and to review the structural design of the bridge. The investigating firm applied mathematical modeling techniques to calculate the dynamic loading of wind, waves and currents experienced by the bridge and anchor system during the storm. Because of the complexity of these calculations, ARL was hired to perform physical hydroelastic studies on a 1:60 scale model to verify the mathematical model.

Complete similitude required that the physical and elastic characteristics of the bridge and its anchorages be scaled. The modes of vibration that had to be included were roll, heave (up and down), sway (left to right) and surge along the longitudinal direction. The wave climate associated with the storm had to be reproduced. The wave generator in this study consisted of a motor-driven variable-speed vertical triangular displacement mechanism. Wave heights were measured with resistance-type gauges, and wave probes were mounted on the bridge to measure wave run-up.

The model bridge was constructed from cellulose acetate butyrate, including the pon-

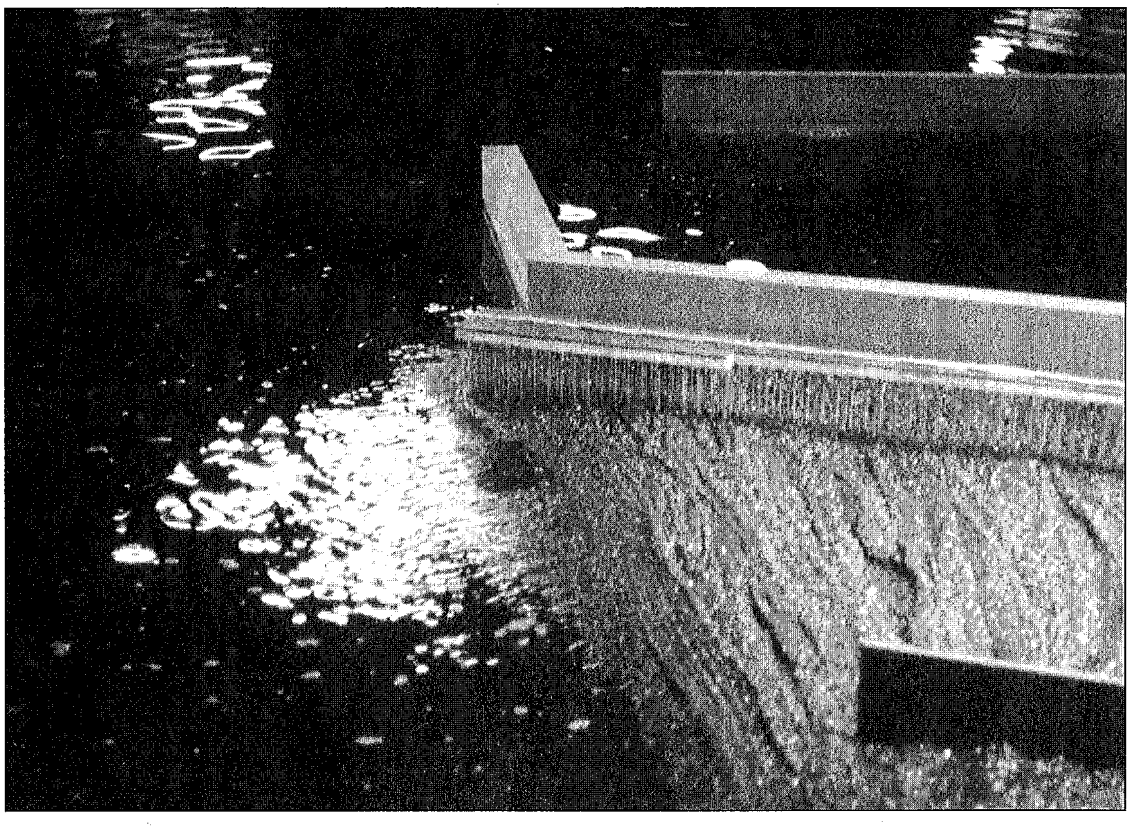

**FIGURE 8. The Port of Champerico, Guatemala, coal model.**

toons. This material was chosen to model the stiffness of the prototype. The pontoons were built of the same 0.060-inch-thick material with some internal ribbing. These pontoons were tested to be sure that they modeled the prototype stiffness in heave, sway and torsional modes. In addition, the center of gravity, the weight and the submergence of the pontoons as well as the bridge were modeled.

The draw span was held in place by gravity to simulate its contribution to the total bridge structure. The bridge anchors were modeled by prestressed wires that went down to the model topography at the same angle as the prototype. However, at this point the model contained a set of pulleys that brought out the wires to strain gauge instrumented cantilevered beams (see Figure 9). These beams were used to obtain the forces on the wires during operation of the model.

Thirty-one channels of data were scanned sequentially in less than 3 milliseconds using a minicomputer. These channels included four wave probes mounted in the basin, eight wave probes located near the bridge, one reference wave probe and 18 strain gauge anchor force measurements from the bridge. These data verified portions of the mathematical model and provided the necessary information to conclude the mathematical analysis.

Wave height measurement is accomplished using a variety of gauges. For the Pilgrim nuclear power plant model, a resistance-type gauge was used. It consisted of 30 individual stainless steel rods in a 2.4-inch-diameter bundle so that they created very little wave modification. The floating bridge project used parallel wire resistance-type probes.

## Lake Pontchartrain Wave Studies

As in other types of studies, wave problems can sometimes be addressed using analytical approaches. One such case was the ARL study of silting minimization at the Bonnabel Recreation Complex in Jefferson Parish, Louisiana. The recreation complex was under construction on the southern shore of Lake

**FIGURE 9. Hood Bridge Canal under wave attack.**

Pontchartrain and was to include a harbor and a boat launch area protected by breakwaters. A few months after the breakwaters were built, a sediment that appeared to be much like coffee grounds was discovered accumulating inside the marina. These organic-type sediments were present in large amounts in the area and had caused problems a few years previously at a nearby boat launch area. Since the sediments were probably put into suspension by waves (with deposition in protected zones), several breakwater relocation schemes implemented there were unsuccessful.

The sources of the sediment were swamps on the western portion of the lake and from a pumping station used to dewater low-lying areas south of the lake. The sediment was transported by waves produced during periods of sustained high northwesterly winds.

ARL collected a sample of the sediment and proceeded to determine its density, settling rate and beginning-of-motion characteristics (critical shear stress). The density and settling characteristics were measured, and the critical shear stress was obtained in a 1- by 1-foot flume where sediment formed the bottom bed. Analysis of the sediment motion in the area of the complex was made considering local wind data and hindcast waves, small tidal currents due to the Gulf of Mexico and the sediment data generated at ARL.

Two mechanisms were studied as possible causes for the sediments to be put into suspension. The first mechanism was the possible instability of a two-layer system comprised of the top layer of water over a bottom layer of saturated grounds causing this layer to behave as a fluid. With the passage of surface water waves, interfacial waves could cause the saturated sediments to go into suspension. The second mechanism considered the waves causing a sufficient bottom velocity so that the bottom shear stress produced by water would entrain the "coffee grounds."

The analysis revealed that large amounts of suspended sediments were generated by waves and transported to the marina where deposition occurred. The analysis indicated that a possible two-stage extension of the harbor entrance breakwaters would reduce deposition.

## Power Generation Using Wave Energy

In the 1970s and 1980s, when the oil situation was causing power anxieties, a number of schemes were proposed to generate power by the harnessing of wave energy. One such study sponsored at ARL involved a submerged pressure-sensitive wave power conversion device. U.S. Patent 3,353,787 describes the device:

> "[C]omprises generally a plurality of elongated tubes, spaced one behind the other adjacent to the floor of the ocean near the shoreline and parallel to the wave front. Each tube has a flexible upper surface, is closed at its ends and has a bottom conduit leading to a collecting conduit, which in turn, leads to a fluid motor. The tubes, conduit, and motor comprise an essentially closed system containing an incompressible or hydraulic fluid such as water or oil. As an ocean wave passes over each elongated, compressible tube, pressure is exerted on the flexible surface so as to cause the fluid in the closed system to be pushed into the fluid motor. The outlet conduit for each tube has a valve permitting rapid outflow and slow return flow. Sufficient compressible tubes are provided to average the effect of the waves, thus producing a substantially smooth flow of water into the collecting conduit."

The study at ARL was a preliminary investigation prior to the construction of the prototype. The study gathered the following information:

- approximate operating efficiencies of the compressible tube assembly;
- expected operating efficiencies of a complete generating system; and,
- a general engineering evaluation of the submerged-pressure sensitive wave power conversion concept.

The result of the tests on the device indicated that its performance could be characterized as a function of the ratio of the total area of the outlet pipe to the surface area of the membrane. It was also suggested that the pressure output would be unsuitable for water turbine operation without the use of a pressure intensifier. The report on the study also contained an extensive bibliography on ocean wave power and conversion.

While this device depended on subsurface pressure variations, other schemes depended on variations on the surface profiles of the traveling waves. One such device was studied in a student project. An inventor had developed an anchored float containing a number of piston pumps, along with a series of one-way valved lines, that would eventually end at a power-producing device. The piston rods would be anchored to the bottom of the seabed, and pumping would be by the up-and-down action of the waves and springs inside the piston housing. The scheme was found to have little practical application at the time.

## Wave Generation

In all of the ARL wave studies, three different but traditional methods of generating waves were utilized. Another method was contemplated and preliminary investigations were made. This method consisted of displacing water using air. In the testing, a short 8-foot section of a tunnel model was capped at each end. Eight slits, approximately 2 inches high and 11 inches long, were cut on the bottom side of the tunnel, and a 2-inch-diameter pipe was welded to the top of the tunnel to admit air. The operation of the wave generator consisted of admitting compressed air at the top of the tunnel, forcing water out the slits and then venting the air out from the top. The wave amplitude could be changed by varying the amount of air pumped into the top of the tunnel, while the period was controlled by changing the rate at which the air was pumped in and vented. Unfortunately, time to do the testing on the model where this system was being developed precluded finalizing the system details and an existing wave generator was used.

## Unique Projects

Not counting the scores of field tests, the many student theses that started in the 1920s, the numerous student projects, many thousands of fluid meter calibrations and the uncountable consulting projects, it is estimated that ARL conducted about 1,100 physical hydraulic model studies during its first century. To this

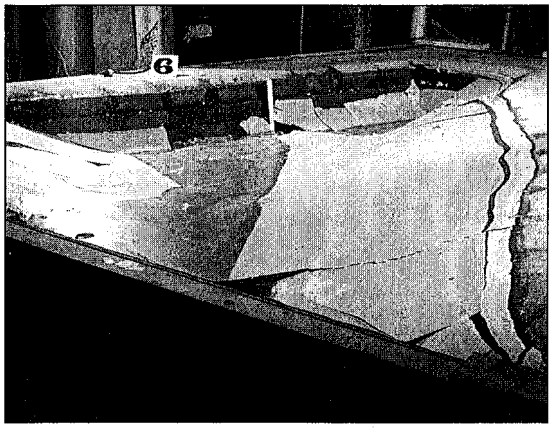

**FIGURE 10. Cabin Creek ice model.**

number must also be added many analytical studies. Some of these models were one-of-a-kind or could not be categorized. However, the uniqueness of these models makes it necessary to include some discussion of them.

## Ice Studies

Four ice-related studies have been conducted at ARL over the years. Three of them — Cabin Creek, Blenhein-Gilboa and Davis — were upper reservoir studies in pumped storage schemes. In these three, the ice study was just a small portion of the investigation to obtain information to prevent ice jams in the upper reservoir intake during the generation mode. Paraffin was used to simulate ice. For the Cabin Creek study, hot paraffin was poured on the model water surface and allowed to cool (see Figure 10). The model was then cycled to simulate the pumping-generating cycle on the upper reservoir. Unfortunately, during the first tests, the paraffin did not crack up like real ice. For subsequent runs, the paraffin was scored in hope that it would break up. However, this solution also did not work, so the paraffin was broken up manually and scattered in the reservoir. Small pieces of paraffin were also used in the other two studies.

The fourth model study was specifically intended to investigate ice pile-up in front of New York Power Authority's tunnel intakes on the Niagara River. The very large model created for this study had three channels feeding into the main Niagara River channel upstream of the intake structure. Two of the channels, Tonawanda and Little West, were located on either side of Grand Island. The third channel represented the Chippawa Channel, and was located between Navy Island and the Canadian shore. The model ended downstream of the International Control Structure.

The objective of this model study was to understand how ice jams formed in the area of the intakes and to minimize the formation of ice dams in the river that cause flooding in the city of Buffalo. The ice enters the river from Lake Erie. For certain wind conditions, the water elevation at the head of the river can be 10 feet higher than normal, causing a large ice-carrying flow to go down the river.

One of the most difficult parts of the study was to find a material that would simulate ice. Different plastics were investigated and some were found to have the proper density but not the proper internal friction of ice. Finally, after meeting with personnel of a plastic recycling plant and running various crushing tests on plastics, a polypropylene with the same specific gravity as ice (0.92) and the same internal friction was chosen. Use of this crushed material is unique in hydraulic scale modeling. The very large size of the model and the use of 15.4 tons of the "ice" make this the largest model of its type ever built.

Now the problem became one of calibrating, operating and putting ice into this large model. After field velocity data had been obtained using drogues and the global positioning system (GPS), it was decided to divide each of the three incoming channels into ten sections across the width of each channel. The flow into each portion of the channels was fed from a pipe coming from a head tank. This head tank was unique since it served many functions (see Figure 11). All of the water and plastic ice used in the model were pumped into the tank after it had reached the downstream end of the model. In the tank, the ice was separated from the water and floated up to the surface in one portion of the tank. The water flowed into separate tanks over three weirs, representing the three river channels. Ten adjustable weirs leading to the pipes feeding the model were installed in each of the three tank sections. The ice was fed into the water downstream of the individual weirs. A variable-speed rotating scoop, which could

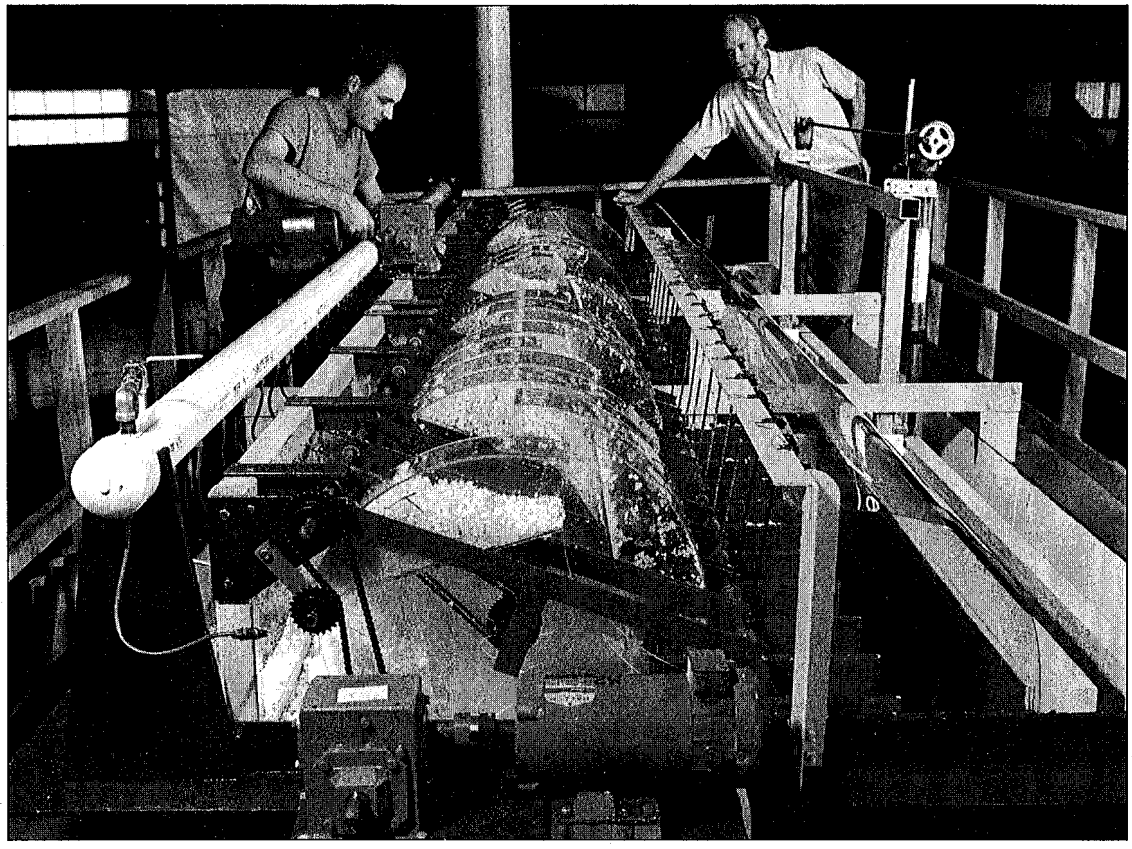

**FIGURE 11. Ice feed mechanism for the Niagara River study.**

also be raised or lowered, would pick up the ice and drop it into the flowing water. The scoop system was calibrated so that adjustment of rotational speed and depth of the scoop would supply a known volume of ice.

Novel instrumentation used for this model included a paddle-wheel-type ice velocity and direction measuring meter. Ice depth was obtained by using a vertically traversing rod containing light-sensing instrumentation. As customary for later-day models, all of the data were processed by computer and the results were available either on-screen or as hard copy.

## Rippling Water & Kayak Raceway

In 1974, ARL studied a channel in South Bend, Indiana, on the west side of the St. Joseph River between a future development area to be called Century Center and Grand Park. The gradually sloping channel was part of an abandoned power site, and the city wished to convert it into an aesthetically pleasing part of the greater park area. The concept was to provide rippling water descending the channel that could be illuminated at night with colored lights.

The laboratory was given the task to design a series of blocks or steps that would create the illusion of a large rippling brook. For construction purposes, there should be some symmetry to the design but, when viewed, the flow and ripples must appear random.

A full-scale model of a section of the sloping channel was built in the basement of ARL Building 2. A repeating series of three rows of blocks, similar to baffle blocks in a spillway, were used to achieve the desired results. A spotlight with a rotating color wheel in front (normally used with Christmas trees) was used to illuminate the rippling water. When viewed by standing around the model or on stepladders, one got the impression of randomly rippling water as was required.

In 1980, ARL studied the proposed East Race Park Waterway at the same site on the east side

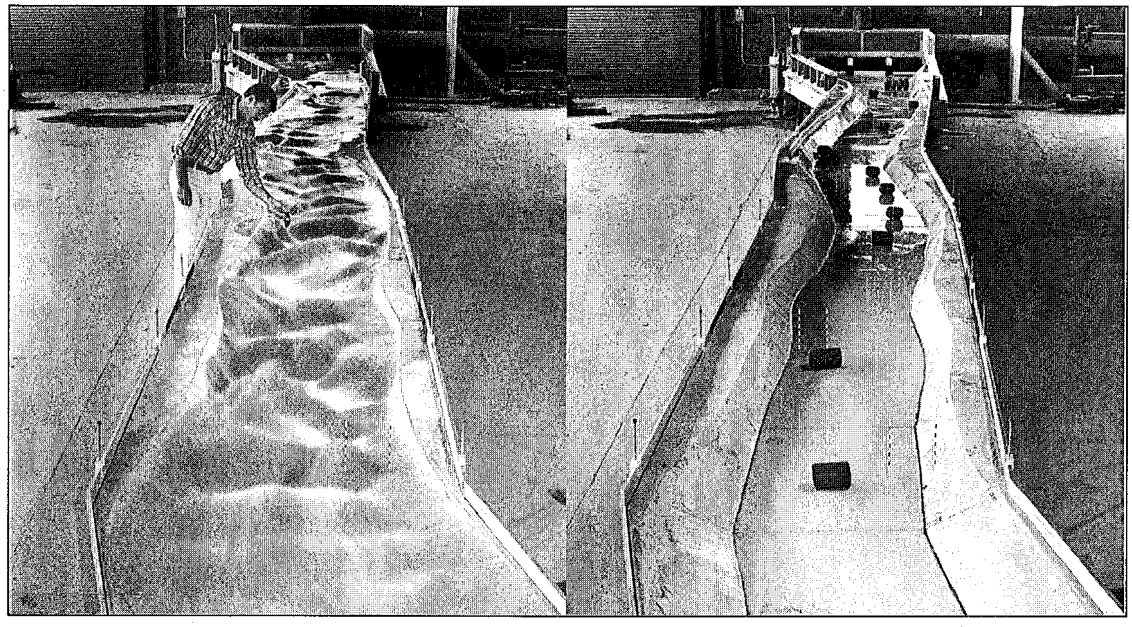

**FIGURE 12. St. Joseph River kayak raceway model with flow (left) and without flow (right).**

of the St. Joseph River. This development would serve as an aesthetically pleasing water passage for passive recreation, a competitive whitewater race course, a fishway and a recreational waterway for canoeing, inner tubing, kayaking and rafting.

One of the first pieces of information supplied to ARL was an aerial photograph of the site. The photograph clearly showed the bend in the river where the proposed waterway was to be built, including other pertinent surrounding areas. But when someone looked on the west side of the river, there was the symmetrical "random rippling" waterway that was designed six years earlier. From ground level, the water flow appeared random, but from airplane heights, you could easily discern a symmetrical pattern.

The proposed east bank channel would be approximately 1,900 feet long, starting at a structure with four control gates upstream of the dam and ending downstream of the river bend. One of the gates would be permanently blocked off. The other three gates would be hinged at the bottom so that when they were fully down, they would create a smooth floor. Part of the study was to rate the gates.

Because steelhead trout and salmon fisheries had been extended up the St. Joseph River, the channel also served as a fishway for upstream migration during the months of September and October. During those two months, the channel would be limited to fishway use only. At that time, 13 baffles with a 1-foot drop in water elevation would be installed by crane in the channel. Each of the baffles would be installed in four H-piles installed in sockets in the floor of the channel. After the fish run, the baffles could be easily removed and the channel could revert to recreational purposes.

It was anticipated that the kayak raceway would operate during March, April, May and June (see Figure 12). It would utilize up to 40 obstructions, held to the channel floor containing a grid of anchors. In addition to the obstructions, there would be a series of 25 to 30 race course gates. (A gate consisted of two poles suspended above the water surface through which a competitor must pass.) A kayak raceway consultant was hired to evaluate and make recommendations regarding the course. Since the model was built on a 1:12 scale, the consultant was unsure of the relationship between the model velocities and those in the prototype. To resolve the problem, moving pictures of a model kayak were taken at different locations in the course. These movies were then played back at 3.5 times the speed, representing prototype velocities.

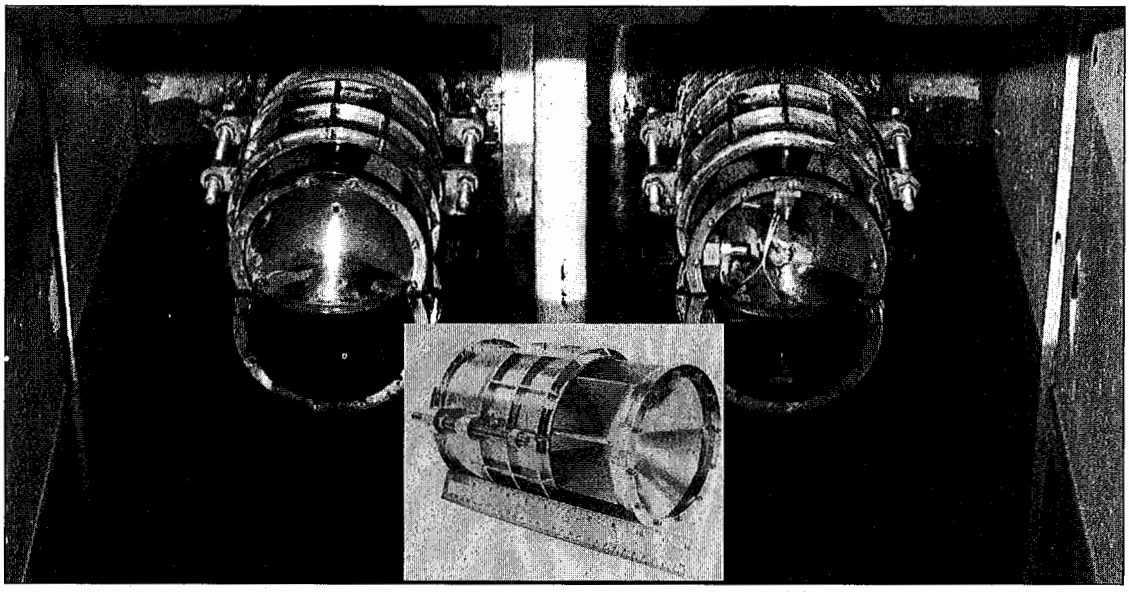

**FIGURE 13. Submerged hollow-cone valves.**

During operation of the recreational channel for canoeing and innertubing, a low flow rate of 100 to 200 cfs would be used with stop logs to produce an interesting and safe ride. Another aspect of this multifaceted project was the measurement of velocities at the channel discharge into the river. These data were used to evaluate the potential for scour in the river bed.

## Dam Rollers

While South Bend was looking for rough water for its kayak raceway, some rough water was creating serious problems elsewhere. In one case, a low dam, where water overflowed, was being used by canoeists in spite of danger warnings posted upstream. People were also walking across the slippery surface of the dam. Under certain flow conditions, a plunging nappe along the downstream face of the dam created a recirculating roller. People who might get caught in this roller due to slipping on the dam or overturned canoes would be in jeopardy of losing their lives.

Model canoes and model debris were videotaped to study the action of the rollers. These videos were used to evaluate various schemes and modifications. By using a step- type configuration on the basin downstream of the dam, the dangerous rollers were eliminated.

## Flow-Induced Vibrations

A 1978 project studied flow-induced vibrations of 54-inch-diameter submerged hollow-cone valves installed at the Dariush Kabir Dam (part of the Doroodzan multipurpose project located near Marve Dasht, Iran). These valves, which had been built for flood control and irrigation purposes, had experienced mounting failures during their first six years of operation.

The problem was studied with a 1:10 instrumented scale model. Two valves were used in the study. The valves were fabricated in a manner similar to the prototype, using the manufacturer's drawings. One of the valves was instrumented with four pairs of semiconductor strain gauges, two accelerometers and a pressure transducer. One of the pair of strain gauges measured the bending force acting on the barrel of the valve, while the other strain gauge compensated for temperature changes. The accelerometers were mounted on the cone of the valve to obtain the horizontal and vertical components of the cone acceleration. The pressure transducer was used to measure the pressure fluctuations in the wake of the jet emerging from the cone.

The tests indicated that forces and accelerations on the submerged valves were as much as four to five times higher than if operated in air,

which is the normal arrangement. The original air-induction system was inadequate. A conical shroud air inducer was designed to reduce the forces and accelerations (see Figure 13). The model test findings also confirmed that the fluid excitation was not near the natural frequency of the valve vanes that had caused vane failures in other similar valves, which is why the prototype vanes had not experienced failures. The valves were effectively vibrating as a cantilever beam.

## Flow Through Porous Gravel Structures

In 1976, ARL studied head loss in porous gravel structures being contemplated for use as water intake screening structures. Such structures, it was felt, would have less environmental impact and could contribute to the fisheries of New England.

Three different sizes of stones with mean diameters of 1.79, 4.9 and 10.1 inches were studied in rectangular gabion structures and in 1.5:1 trapezoidal configurations. These studies were carried out in an 8-foot-wide by 6-foot-deep and 60-foot-long glass-sided channel. The stone sizes and flow rates used for this study produced higher Reynolds numbers than previous investigators had achieved.

The conclusions drawn from this study were:

- the friction factor for turbulent flow depended primarily on the stone angularity and secondarily on the porosity of the sample;
- the shape of the dike is not important if the effective flow length could be adequately described;
- excessive head loss or flow velocity caused slumping of the dike material; and,
- the velocity distribution at the downstream face was highly irregular due to local variations in porosity.

In the long run, these types of intake structures were not built due to the inability to prevent or remove crustacean build-up and the inability to remove seaweed or other weeds that would get intertwined in the stones.

## Chemical Vessels & Bubble Diffusers

On two occasions ARL studied large vessels used in the chemical industry. One vessel was a bubble reactor and the other was a decanter.

The bubble reactor study was performed with a 1:2.3 scale model of a 13.7-foot-diameter vertical tank designed to generate a chemical reaction between a gas and a liquid. The objective of the study was to achieve an even flow and gas distribution across the cross section of the vessel as soon as possible to ensure maximum use of the reactor vessel. For the purpose of the study, only the lower portion of the vessel was modeled using clear acrylic plastic.

The vessel had a hemispherical bottom with two feed lines into the bottom. An 8-inch injector nozzle located 100 inches above the vessel bottom discharged 7.9 cfs of liquid and 1.6 cfs of gas vertically upwards. A deflector plate forced the flow out horizontally in the vessel. The other inflow came from a 15-inch-diameter downcomer line directing 16.2 cfs of liquid down the side of the vessel in the area of the bottom hemispherical section.

The original design was modified in many different ways. First, the injector nozzle was changed by rounding the exit. Next, the injector line was fitted with static mixers (a fixed tab or other device that produces turbulence) to obtain better mixing of the gas and liquid. Holes were drilled in the injector deflector plate to provide more uniform distribution of flow in the center of the vessel. An annular baffle was also required to force the flow away from the wall.

The downcomer flow deflected off the hemispherical bottom and flowed up the opposite wall. To remedy this situation, a baffle was installed at the bottom of the tank. To keep small particles in suspension, the baffle was mounted about 1 inch above the bottom. When tilted 10 degrees vertically away from the downcomer, a more uniform distribution was achieved.

The second test was on a horizontal decanter that was 53 feet long and 15 feet in diameter with rounded end caps. The model was built of clear PMMA plastic to a scale of 1:5. The objective of the decanter was to separate two essentially immiscible liquids. The liquids entered the decanter through a hori-

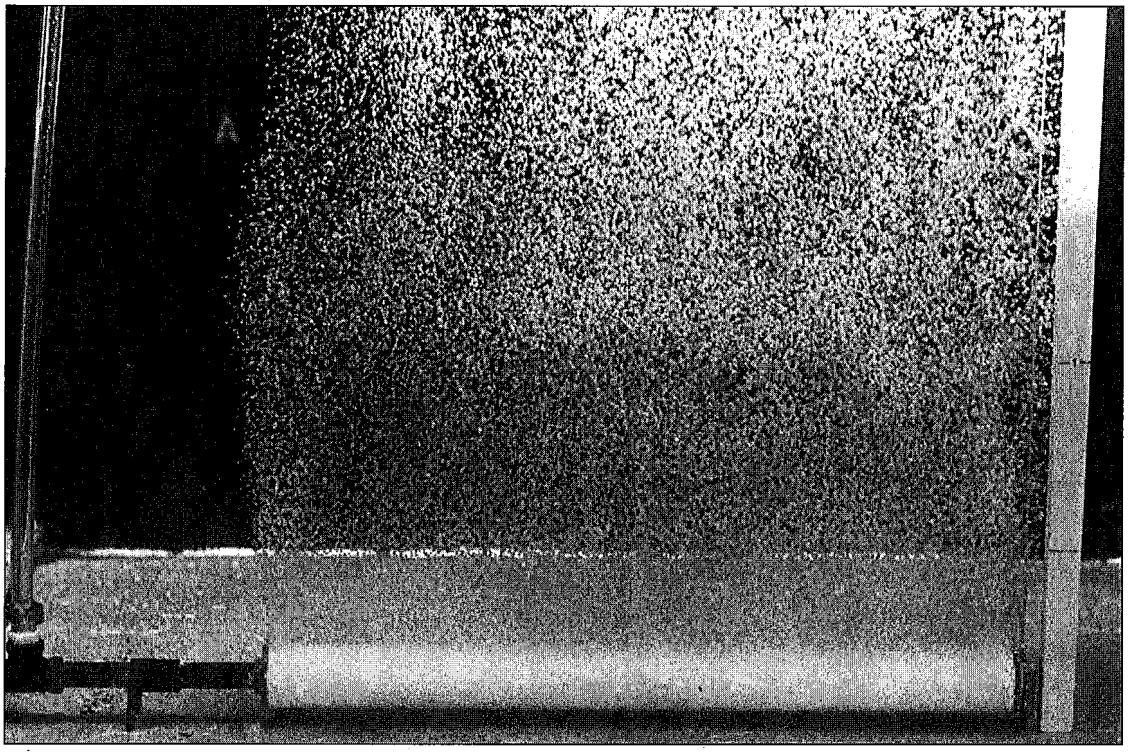

**FIGURE 14. Study on the oxygen transfer rates for bubble diffusers.**

zontal 8-inch-diameter line located 3.5 feet above the centerline, and through an 18-inch-diameter vertical line entering the top of the vessel. The flow from the 18-inch line was deflected toward the end cap by means of a baffle. The inflows also could contain small amounts of gas. A calming baffle, with an off-center 6-foot-diameter hole and an opening at the top to allow gas to pass and a small opening at the bottom to pass settled liquid, was located approximately 6 feet from the inlet end of the vessel.

The decanter had three outlets. One 8-inch-diameter vapor outlet was located at the top of the decanter. At the far end of the decanter, a 14-inch-diameter outlet took out the lighter liquids at the bottom of the decanter. Approximately 10 feet upstream of this outlet, an 8-inch-diameter pipe (also located on the bottom) removed the heavy liquid. An overflow baffle, used to separate the heavier from the lighter fluids, was located just downstream of this pipe.

The 18-inch-diameter inlet pipe was found ineffective in separating the gaseous component. A redesign of the discharge of this inlet consisted of a 36-inch-diameter, 24-inch-high basket with 2.5-inch-diameter holes drilled around the circumference. The basket was filled with plastic packing-type material. The new design reduced the penetration of the gaseous portion of the flow into the decanter. The single-hole calming baffle was ineffective in producing good flow distribution. A perforated-plate calming baffle was found more effective, as indicated by hot wire anemometer traverses. A perforated pipe extending 7.5 feet below the top of the vessel was added to the outlet pipe at the top of the vessel to remove the gaseous phase of the inflow, which allowed gases that were still in the liquid (as well as separated gas) to be removed.

In a study that has application in the wastewater, pulp-paper and chemical industries, the performance of cylindrical fine bubble diffusers was studied (see Figure 14). Proper design of re-aeration systems using fine bubble diffusers requires a detailed knowledge of the oxygen transfer performance for individual diffuser units. The parameters investigated

with 12- and 24-inch diffusers were oxygen transfer efficiency, influence of tank depth and head loss flow characteristics.

## House Roof Shingles

ARL performed testing on house roof shingles in 1984. Forty 4- by 4-foot panels containing different full-sized roof shingles of different configurations were tested in wind conditions of up to 110 miles per hour and temperatures ranging from 30 to 40°F. The study determined the minimum wind speed that would lift the shingle edges. The wind speed that would blow the shingles off the panel was also obtained. Gusting wind studies were also performed.

A 75-horsepower blower was used to supply the wind. A transition section was constructed to go from the blower to the test section. This transition was designed to provide uniform velocity distribution at the entrance to the test section. It also incorporated vanes that could be varied to change the thickness of the exit flow. The thickness of the flow for most of the testing was 15 inches. The test section incorporated a clear viewing window used to observe the performance of the shingles and to videotape the tests.

Much information was gathered about the performance of shingles in cold weather. The client was pleased with the results, but the project was not overly popular with the technicians who spent numerous cold hours running the tests and sipping many cups of coffee to keep warm.

## Pump Testing Facility

In 1960, a pump manufacturer gave its testing equipment to ARL. The facility included a large closed cycle tank for testing vertical pumps up to 100 horsepower. These pumps were driven by a cradled direct current variable speed motor, which was also used as a dynamometer. The tank was modified at ARL to obtain better approach conditions to the pump and to install a calibrated Venturi meter for measuring flow. A 50-horsepower cradled motor for testing centrifugal pumps was also part of the equipment obtained by ARL. All of the equipment was used to perform efficiency tests on pumps and to do experimental work on pump impellers for various applications. However, because of the need for space and relatively little current use, ARL dismantled the pump testing facility about 1980.

In 1983, comparison performance tests were conducted using a different facility on five centrifugal pumps from different manufacturers. All pumps had 3-inch inlets and 1.5-inch outlets. Testing was conducted according to Hydraulic Institute Standards. Similar comparative testing on eleven 0.5-horsepower and three 1-horsepower submersible centrifugal pumps from different manufacturers was performed in 1992. In this case, the brake horsepower was obtained by measuring the electrical input to the motor and obtaining the brake horsepower from the calibration of the electrical motor.

It is interesting to note that while the instrumentation was more sophisticated, this type of testing was similar to what C.M. Allen did some 70 years earlier on small turbines at ARL and in the field.

## Static Mixers

Static mixers are fixed additions to the inside of a pipe or tunnel and are used to mix two fluids in a relatively short section. The additions could be welded tabs or a series of twisted flat plates that force the flow to rotate first one way then the other as it flows down the pipe. A number of tests on such devices were conducted in the late 1980s and early 1990s. These tests were performed with various main line flows and sparger flows (a device that produces mixing). In all cases, the additional flow through the sparger was tagged with Rhodamine WT dye. At a location downstream of the mixer, grab samples were taken from equal sub-areas of the cross section. These samples were analyzed using a fluorometer to determine mixing efficiency. A static mixer proposed for the West Point Treatment Plant in Seattle, Washington, was tested in a scale model, including the tunnel geometry. After the plant was completed, field tests of the full-scale installation using similar test techniques were undertaken that substantiated the model results.

## A Tunnel Inside a Tunnel

In 1987, ARL studied a proposal to put a 78-inch-diameter penstock inside a 196-inch-diameter flood control tunnel to send water to the Bloomington Lake hydroelectric plant located on the North Branch of the Potomac River near

Westernport, Maryland. The Bloomington Lake Outlet Works consisted of a free-standing intake tower in the reservoir, a 1,885-foot-long concrete-lined flood control tunnel, a stilling basin and a discharge channel. The intake tower contained a low-flow release system and a flood-flow release system. The low-flow release system was designed to maintain the water quality of the released water by mixing water from five different control-gated water elevations, ranging from 1,342 to 1,449 feet, discharging into two wet wells. A complicated transition brought the flow from the wet wells to the flood release tunnel.

The planned hydroelectric plant would use water from the low-flow release system to supply the penstock pipe suspended from the top of the flood release tunnel. The already complicated transition from the wet wells would be further complicated when modified to release flow into the power tunnel. In addition, there were time constraints for the construction of the modified transition to limit flow interruption.

One series of tests involved obtaining losses and coefficients from the intake structure to the power tunnel. Piezometers on the tunnel were linked to electronic differential pressure transducers interrogated by a personal-computer-based data acquisition system. From these data, a mathematical model developed rating curves for the system's operation. These curves took into account factors such as density variations, trash racks, etc., that had not been physically modeled.

Since the water from the low-flow release was to be used for power production, it was important that swirl, and especially air entrainment, from the wet wells be eliminated. This problem was studied in the intake model using vortex suppressors to remedy the problem. At the same time, tests were made to look at existing vibrations, possibly caused by the complicated nature of the flow in the wet wells.

Another concern about the project was slug flow (unsteady flow of large quantities of air and water) in the flood release tunnel during filling and draining operations. Slug flow may produce the violent expulsion of entrained air from the tunnel, causing adverse unsteady flow and pressures in the tunnel. The model was used to identify these slug flow conditions so that operation at these conditions could be minimized.

Finally, near the end of the flood release tunnel, the power tunnel had a wye-type configuration with one branch exiting the flood release tunnel to go to the powerhouse. The other branch went to a cone valve that sprayed the low-flow release flows that exceeded the powerhouse demand into the stilling basin. The complete operation of the stilling basin (including the flood discharge), the powerhouse discharge and the spray from the cone valve were also investigated in this study.

Another ARL study using the same power tunnel inside a flood-flow tunnel scheme was undertaken for the Gathright Dam located on Moomaw Lake, 10 miles north of Covington, Virginia. Many similar tests were conducted, including force measurements to assist in the tunnel design during the wide range of free surface and pressurized flow conditions, including slug flow.

The forces exerted by the flows on the power tunnel were measured in the model by installing a short length of tunnel containing a three-component force transducer. Forces were measured at different locations along the tunnel using different flow rates, ranging from a low flow just splashing on the bottom of the power tunnel to those that completely filled the flood tunnel.

## Field Flow Measurements

Flow measurement — particularly with respect to pump and turbine efficiency tests — has been an ongoing activity at ARL for almost 100 years. After the introduction of the salt velocity method in 1921, over 400 field tests using this technique were performed by ARL staff, with the last of these tests being conducted on the Windy Gap Pump Station on the Colorado River in 1985 using a computer-based data acquisition system. The last of these tests was performed in 1965. However, the majority of salt velocity tests were conducted prior to the mid 1950s. As plant sizes increased, the use of the method became very expensive. Additionally, the costs of actually running the tests, shutting down a plant to install and remove the pop valves and electrodes meant an additional loss of income to plant owners. Then there was always the potential for accidental damage to the turbine if parts of the pop valves or electrodes were to break and be carried through the

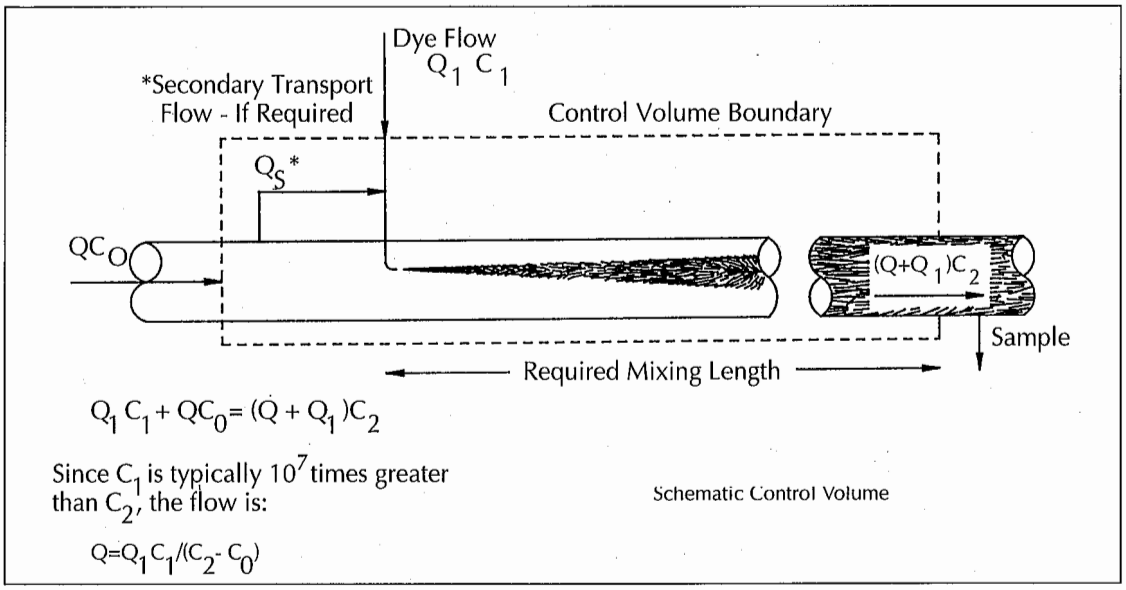


**FIGURE 15. The tracer dye dilution flow measurement method.**

turbine (which happened only once in all of the tests conducted and, luckily, the turbine suffered only a small nick on one of the blades).

The decline of the salt velocity method did not mean that the need for turbine and pump testing in the field had declined. Other methods were being proposed and used to obtain flow rates, some of which yielded questionable results.

## Tracer (Dye) Dilution Method Field Testing

In 1975, ARL was hired to measure the efficiency of the two axial flow circulating water pumps for the Brayton Point Station in Swansea, Massachusetts. After reviewing the system, it was decided to perform the tests using the dye dilution method for measuring the flow. With numerous refinements since then, the dye dilution method is now, in most cases, ARL's preferred technique to measure turbine or pump flows to an accuracy of ±2 percent.

The dye dilution method of flow measurement (later called the tracer dilution method), like the salt velocity method, is conceptually very simple (see Figure 15). Dye of a known concentration is injected at a known constant rate into the flow being measured. Downstream of the injection point, at a location where this dye is completely mixed with the flow being measured, a sample of the mixed flow is taken and its concentration is measured. By using the conservation of dye principle, the unknown flow rate is obtained.

As simple as this may sound, small (and sometimes intricate) refinements contribute to the accurate determination of flow — especially true for all methods of measuring flow in large pipes. One of the advantages of the dye dilution method over the salt velocity method is that it requires no equipment internal to the pipe. The other major advantage is that only a little dye is needed for any study since the measuring instrument — the fluorometer — can accurately detect parts per billion of the dye. Other advantages include no interruption of plant operation, numerous data points can be quickly obtained and repeated if necessary, and the dye can be injected at convenient points as long as sufficient mixing occurs.

For the Brayton Point tests, Rhodamine WT was injected at the bell mouth of the vertical pumps using a metering pump, and the injection flow rate measured using a graduated burette and calibrated electric timer. After the flow had gone through a series of elbows, a sample of the completely mixed flow was taken inside the plant, some distance downstream of the pump. A fluorometer was used to obtain the concentration of dye in both the sample and the injection flow.

The tests indicated a 16 percent loss compared to design conditions. The plant's owner ordered a shutdown and inspection of the system. Pieces of styrofoam cups were found blocking a sizable number of condenser tubes, thereby reducing the flow and increasing the system's head loss. These cups were the remains of coffee breaks taken near the spray cooling canals by the many construction workers still on the site. After the system was cleaned, a repeat test indicated that the pumps were within 2 percent of their designed flow. In later years, tracer dilution tests have been conducted on four additional units at Brayton Point to assess circulating water pump performance.

The start of the modern tracer dilution field tests at ARL began in 1984. At this time, ARL acquired a more refined fluorometer and had run a number of laboratory tests on the effects of fluorometer filters and the temperatures in various parts of the fluorometer with regard to the accurate determination of the sample concentration. This fluorometer was modified so that its outputs could be inputted directly into a personal computer. In the original tests in 1975, grab samples were taken in the field and analyzed at ARL. Presently, the mixed sample flows continuously through the instrument. After a multiple point calibration curve is determined and input into a personal computer, concentration data for the flow sampled from the conduit can be recorded continuously and analyzed in real time. Once the concentrations achieve a steady state, the data can be averaged and recorded.

The tracer dilution technique has been used for many different applications. In a natural gas liquefaction plant in Bethouia, Algeria, the flow rates in the condenser cooling lines were measured and compared to the plant's installed flowmeters. In a number of fossil and nuclear plants in the United States, checks on the circulating water pump rating curves were conducted using tracer dilution. Turbine efficiency tests were performed using the method to measure flow. In many of these tests, the taps on the scroll case were calibrated at the same time to provide a means for the utility to measure the flow rate at any future date.

In one pump storage project, the pump portion of the machine was tested while simultaneously calibrating a recently installed ultrasonic flowmeter. At the Northfield Pump Storage Project in western Massachusetts, both the pumping and generating modes were tested and an ultrasonic flowmeter was calibrated. For the Dietrich Drop Hydroelectric Station located on an irrigation canal in the desert of Idaho, the efficiency of the turbine was obtained, the taps were calibrated and flow capacity was determined. ARL's measurements were commissioned by the turbine manufacturer; however, the skeptical plant owner employed the area velocity method in the canal upstream of the station using a single current meter that verified the tracer measurements.

Tests at the Susquehanna Steam Electric Station in Pennsylvania evaluated the performance of eight pumps in the plant's Engineered Safeguards Service Water System and calibrated the various inline orifice metering sections. Another series of tests were performed at the same plant to measure the circulating water flow to assist in evaluating the cooling tower performance.

Of interest were tracer dilution tests conducted during 1984 in the grinder room of a paper mill in Millinocket, Maine. In 1917, C.M. Allen had used velocity-integration techniques to measure flow and to determine the efficiency of the same units.

In a 1992 pump study in Arizona, a dye injection manifold was mounted at the trash- racks and the "flow-through" sampling mode was used to obtain the flow in a 54-inch pump. This study also involved the investigation of vibration problems in the pump. Three different types of instrumentation were used to evaluate potential causes of the vibrations. Two quartz accelerometers, one perpendicular and one parallel to the intake flow, were mounted on the pump bell housing. Hydrophones were installed in the flow to measure pump noises. The third instrumentation was a video camera mounted on a sump back wall that was aimed at the water area surrounding the pump. Air release at the bottom of the sump aided visualization of the vortex activity in the sump. All the accelerometers and hydrophone outputs were recorded by a personal-computer-based data acquisition and analysis system.

The tracer dilution method of flow measurement can be used in a multitude of applica-

tions, providing that the dye completely mixes with the flow. Most applications involve pumps or turbines where the water quality is reasonably high, all of the inflows are identified and the flow is very steady. These conditions do not have to be true if special care is taken in calibration and field procedures. In 1988, a study of eleven sites on the city of Boston's High Level Sewer System was used to calibrate new flowmeters. These meters were of the ultrasonic type that measured the depth of flow (sewage) in the sewer line in addition to the velocity of flow. From these two measurements and the relationship of the depth to the area, the flow rate was obtained. The tests were interesting in their execution. Injection and sampling stations were set up in the field, sometimes at manholes in the center of busy Boston suburban streets. The staff's concentration, unlike for most testing operations, had to be divided between properly performing the tests and dodging a multitude of "good" Massachusetts drivers. Additionally, upstream of some meters (but downstream of the dye injection point) inflow laterals were suspected that could influence mixing and accuracy. The effects of pH and suspended solids were also considered.

Some field tests were conducted at sites where ARL had performed model studies to evaluate hydraulic features of the project. One such facility was the Blenheim-Gilboa Pump-Storage Plant in New York. In 1988, tracer dilution tests were conducted in the pumping mode to determine the head-flow relationship and the efficiency relationship for the plant's Unit No. 4. A manifold covering the total area of the trashrack was used to inject the dye and to obtain maximum mixing of the dye in the shortest possible time. Mixing was achieved by the flow through the trashrack and by the flow through the pump. To ensure that mixing was complete, samples were taken from the various piezometers around the pipe prior to the actual test and were analyzed to be sure that the readings were all identical.

## Pressure-Time Method Field Testing

The dye dilution method depends on the complete mixing of the dye between the injection and sampling point. If the mixing is not complete, other measurement techniques must be used to determine the plant flow in the efficiency equation. Two alternatives are the pressure-time method and the velocity-area method.

For the Blenheim-Gilboa Plant in 1988, dye-dilution could not be used for turbine efficiency testing because of insufficient mixing. However, the penstock had been equipped with multiple piezometers at two different locations. This configuration was amenable to the use of another method for measuring flow in large pipes, the pressure-time method, originally called the Gibson method after its inventor.

The principle of the pressure-time method of flow measurement is based on Newton's Second Law, which states that the force is equal to mass times acceleration. In application, a steady flow is set in a penstock and then the flow is completely shut off. During this time, a differential pressure-time diagram is generated due to the retarding force necessary to stop the flow (see Figure 16). When integrated, the area of this pressure-time diagram between the pressure line and the friction line multiplied by a constant based on the test length and area provides the flow rate in the penstock before the flow was shutoff. While construction drawings can be used for the conduit dimensions required by the method, the highest accuracy specifies that the conduit be drained and the diameter determined.

The Gibson technique used, and the modern test codes still include, a differential mercury manometer-type instrument with the output on photographic paper attached to a rotating drum operated by a constant speed motor. This instrumentation and data processing are awkward and slow. ARL modernized the technique by using calibrated differential pressure transducers and a personal computer–based data acquisition system running the specially developed software to calculate the flow rate. Shortly after the conclusion of each test run, the flow rate is printed on the pressure diagram. In so doing, the computer has:

- corrected the differential pressure transducer output using the latest calibration;
- generated a corrected pressure-time diagram;

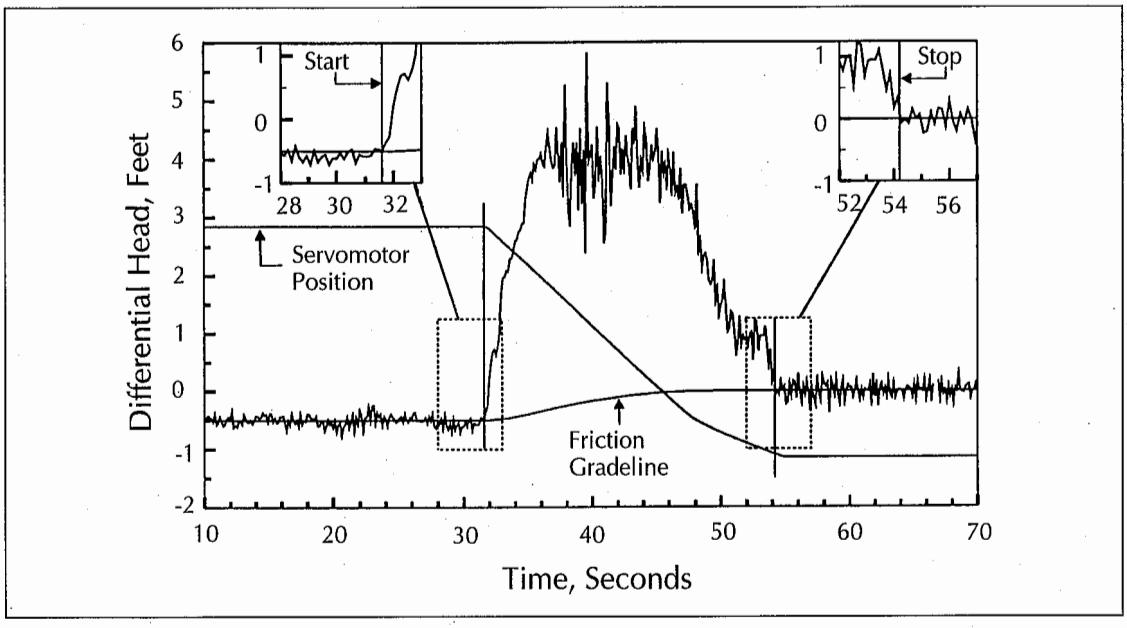


**FIGURE 16. Pressure-time record with start and stop definitions.**

- produced the friction line;
- integrated the area between the pressure and friction line; and,
- computed the flow.
-

The pressure-time method of flow measurement was used several times at a large power plant in New York state and also at the Palmer Falls Hydroelectric Station in Corinth, New York.

## Velocity-Area Technique Field Testing

It is not always possible to use either the pressure-time or the tracer dilution method of measuring flow in some turbine or pump installations. Where the penstock may be short or nonexistent, the velocity-area method should be employed using an array of calibrated current meters to traverse the incoming flow. The gate slots are usually used to guide a carriage with the mounted current meters.

In the velocity-area method, the area of the intake at the metering section is divided into rectangles of equal areas and the velocity is measured at the center of each area. The velocities are obtained by moving a carriage containing several current meters vertically in the gate slot and stopping to take measurements at the desired positions. The total flow rate is then calculated as the sum of the flow rates of each rectangular area (see Figure 17). The meters on the carriage are designed to be sensitive only to velocities aligned with the meter (*i.e.*, perpendicular to the flow area). Therefore, the flow can be determined when intake velocities are very nonuniform. Many different types of low-head turbines can be tested using the velocity-area method. It is interesting to note that during testing in 1989 at the Anson Station in Madison, Maine, ARL staff noticed a large-scale turbine performance curve produced by C.M. Allen in the 1930s. It was on the wall above the desk at which they were conducting analyses of area velocity measurements.

Although flow-measurement-related field studies at the end of ARL's first century comprised a smaller percentage of the laboratory work, they continued to augment the flexibility of ARL's operation.

## Hydraulic Equipment Field Testing

Determining actual turbine efficiency was not always required nor was it always needed in some field tests. It might be that the client wanted to be able to run quick tests on a machine to determine if its efficiency was decreasing with time. To do this, the flow might be "indexed" to some point in the system or to some other source-of-flow indication.

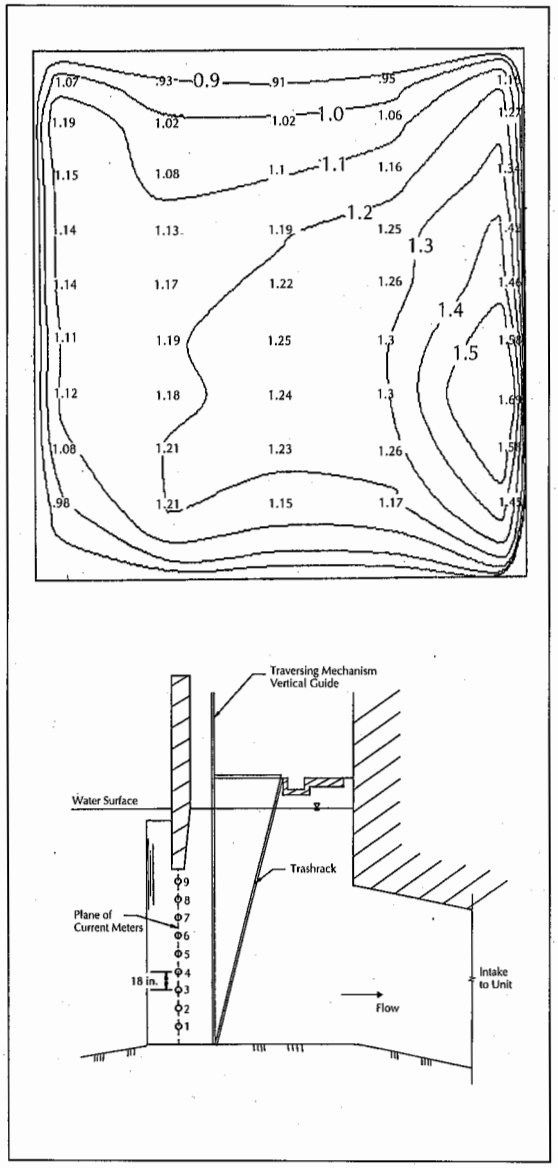


**FIGURE 17. Normalized velocities at an intake section.**

In 1989, ARL conducted index testing at eight power stations containing sixteen turbines in central Vermont. In six of these stations, a pitometer was mounted at the penstock centerline to indicate flow. In one other station, an uncalibrated Venturi meter was used to obtain flow. At the other station, flow was determined using an existing reducer, which simulated a typical Venturi meter.

Because of the nature of power plants (both fossil and hydro), they contain many hydraulic systems that, at times, may not operate as designed. Such was the case at the Robert Moses Niagara Power Plant in New York. In October 1988, a series of tests were conducted to investigate the cause of pressure surges in the penstocks, which caused rough operation of the turbines at some gate settings. Pressure transducers were installed at 11 piezometers that were located in different parts of the penstock and draft tube to determine the magnitude, frequency and source of the fluctuating pressures. In addition to the pressure measurements, the power output, three-vibration signature analysis and the shaft torque were obtained on one of the two units being tested.

Two elements of the data were analyzed to arrive at the source of the problem. The standard deviation of the pressure fluctuations was determined to be the most reliable indicator of the magnitude of the fluctuations. The frequency content was determined by the Fast Fourier Transform (FFT) method, which is an algorithm that decomposes a periodic signal into a Fourier series.

Analysis of the data indicated that the most likely origin of the pulsation was the draft tube. From the draft tube, these fluctuations then carried through the system and caused the rough operation of the turbines.

## Measuring In-Situ Velocities & Currents

As a part of many hydraulic model studies, flow patterns in the field must be documented for "calibrating" the velocities or currents in the model. Originally, C.M. Allen started this practice using hand-held meters. Today, this "mapping" is done using current meters in confined areas or, as was done for the recent Niagara ice model, by tracking floats using GPS. In the late 1960s and early 1970s, drogues were tracked using aircraft. Such aerial tracking was typically done when overall velocity patterns were needed (such as at the James H. Fitzpatrick site on Lake Ontario and Indian Point on the Hudson River). Drogues were fabricated at ARL and dropped/collected using a boat. As the aircraft flew overhead at timed intervals, photographs were taken that were used to measure the displacement of the drogues from which the average velocity could be determined. GPS is a lot easier.

Another float-tracking technique started out using an old-fashioned transit and is convenient whenever there is a high bridge or building nearby. When first used for a plant on the Detroit River, a transit man, with a good note keeper who recorded vertical and horizontal angles, set up on the roof of the station and tracked 8 floats at 1 minute intervals. A boat retrieved and reset the floats.

With the advent of computers, a position indicator with a variable voltage output was coupled to the transit. By use of personal-computer-based data storage and analysis software, more data could be accurately recorded without furious note taking. The final improvement is a "total-station" surveyor's theodolite for tracking the drogues. Recently, floats have been tracked in rivers and power canals as part of determining flow patterns to develop first diversion/guidance booms and sluiceways for downstream migrants.

## Temperature Measurements

When steam-electric plants were required to meet thermal discharge criteria, state regulatory agencies required field verification of the thermal patterns predicted by the physical and mathematical models. Overall surface temperature patterns could be recorded using infrared techniques using an airplane or a boat equipped with one or more temperature sensors. The infrared technique, even with water surface calibration, was accurate to about ±0.5°C absolute or about 1.0°F.

AR developed an automatic data acquisition system for boat-mounted sensors that could simultaneously record the time, surface and vertical temperatures, and position using two shore-based transponders and a transmitter unit on the boat. The data were processed and plotted spatially. Field and model data were then compared for regulatory agencies (see Figure 18). Model and prototype data generally agreed, particularly nearest the point of discharge and away from boundary influences, but wind effects and other factors sometimes made model-prototype comparison difficult. This situation was particularly true at complex sites where there might be multiple plants (with varying heat rates) and/or tidal complications. But even in these cases, a synoptic view of the far field data indicated similarities of field and model temperature data (see Figure 19). As far as it is known for a plant model tested at ARL, no thermal discharge data have ever been determined to be in noncompliance of temperature or mixing criteria.

## Plant Operation Characteristics, Forensic Engineering & Consulting

Numerous investigations, consultations and field tests were conducted every year for a long time and for many clients at ARL. Examples of field services included:

- valve operation or pipe breakage problems related to pump transients or emergency open/closure;
- sprinkler system functioning;
- on-site calibration of flowmeters for plant process control;
- make-up water evaluation for a combined cycle generating plant;
- flow measurement in a fertilizer plant;
- head loss and velocity measurements at the tube sheet face for an air cooler;
- inspection services and leakage tests;
- litigation on failures or other topics;
- condenser targeted chlorination;
- sedimentation;
- thermal discharge analysis;
- insulation evaluation;
- feasibility studies, pressure/pipe burst tests;
- gate vibration evaluation;
- erosion and scour evaluation;
- instrumentation development;
- review of pump station design;
- thermal discharge analysis;
- pump transient analysis;
- and numerous other specialty studies.

Some of these studies involved substantial analysis, which has become a complementary part of many field investigations and physical model studies. The analysis often identifies the most fruitful options to be further investigated and optimized.

## Fisheries Issues at Water Intakes

When a multidisciplinary team of fisheries experts joined ARL in 1994, a wide range of stud-

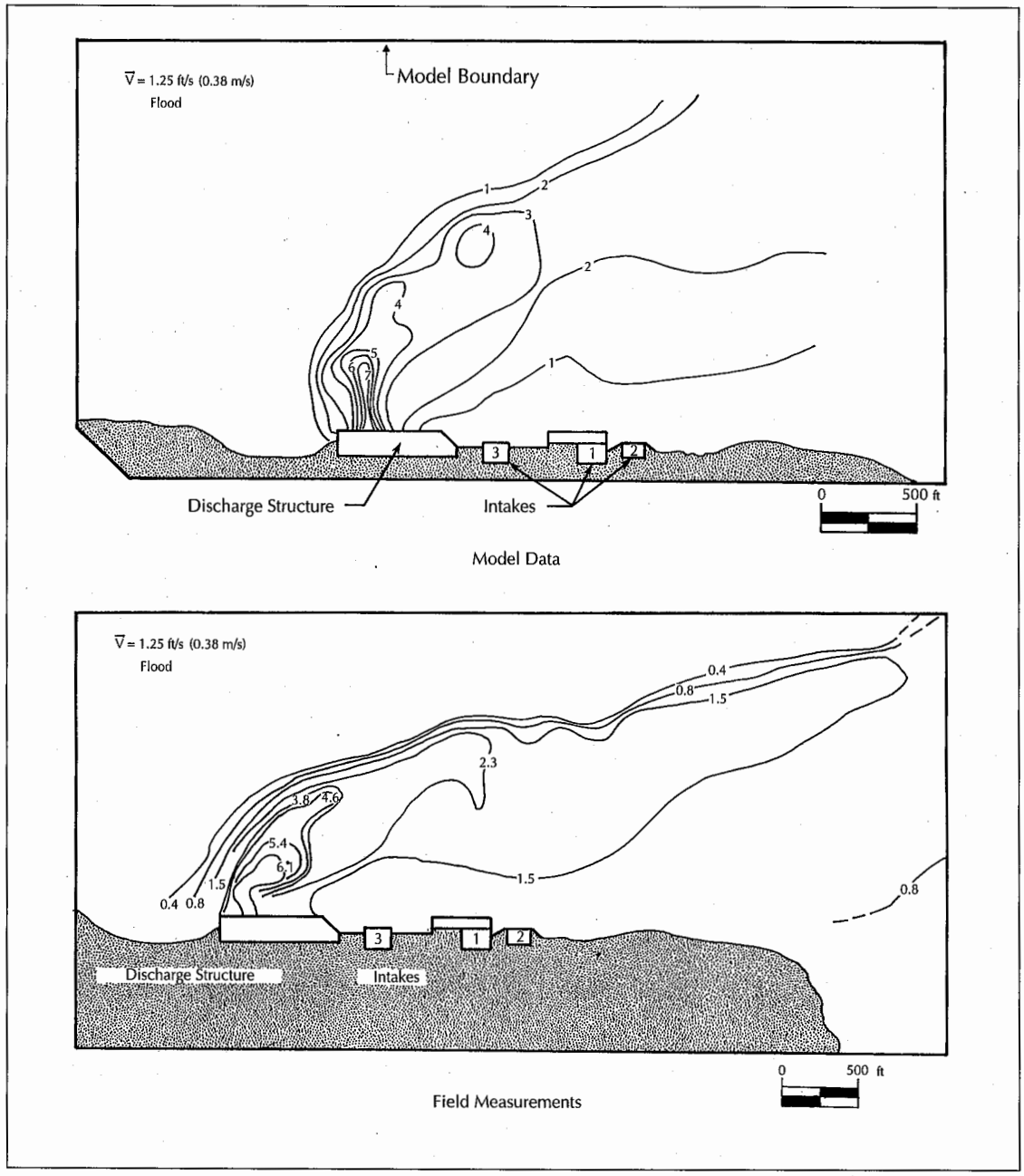


**FIGURE 18. Comparison of nearfield model and field surface temperature rise patterns during tidal conditions at the Indian Point Station (Hudson River, New York).**

ies relating to fish protection and passage was enhanced. Guiding or deterring fish from entering high-flow water intakes has been accomplished using various devices, including underwater sound and strobe lights. Since the performance of these systems is species-dependent, testing to determine effectiveness is required. In some cases, tests in a laboratory flume are most effective. In other cases, parameters dictate that the response of fish be determined in the field. For this purpose, ARL's floating test platform can evaluate on-site the deterrence effect of strobe lights and noise. The platform supports the sound system so that fre-

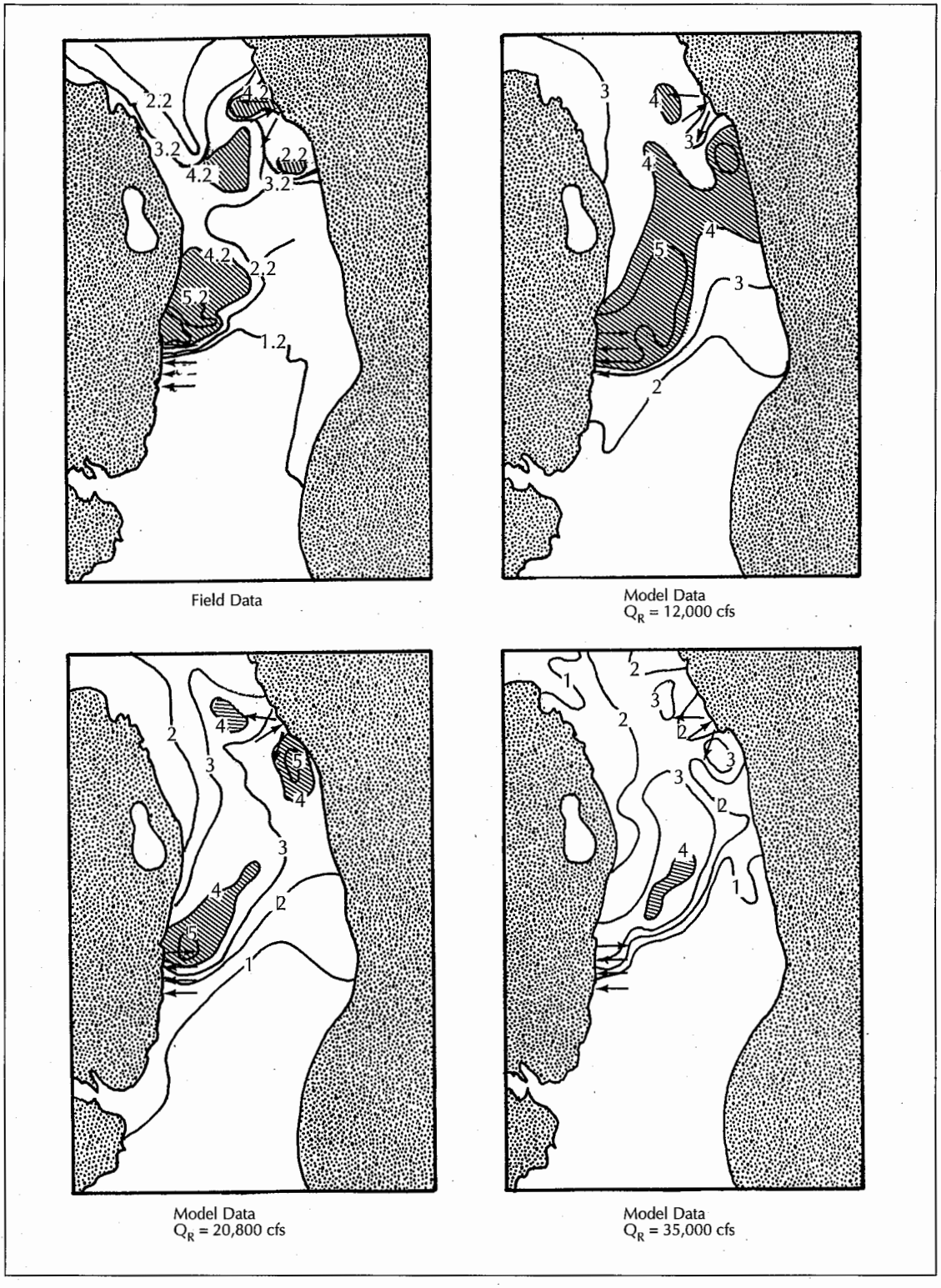


**FIGURE 19. Field/model comparison of overall temperature rise patterns at the Indian Point Station (Hudson River, New York).**

quencies, wave forms, pulse patterns and other parameters can be evaluated with the existing site background noise pattern. Underwater video equipment records the response of fish.

Resident and migratory species of freshwater, estuarine and marine fishes have been evaluated, including salmon, striped bass, anchovy, perch, walleye, northern pike and catfish. The species evaluated are all of commercial and/or recreational value and must be protected at water intakes. The results of the studies have varied by species and device. Species' response ranges from no response to strong repulsion with both sound and strobe lights. The results are used by ARL's clients to determine whether these fish-repelling technologies are appropriate for their various water intakes. This general expertise has led to numerous relicensing/National Pollutant Discharge Elimination System (NPDES) studies, investigations on mechanical screening, consultation with state and Federal regulatory agencies, and evaluation of alternative fish protection devices.

Environmental field studies related to fish diversion and passage have become a major ARL activity. Overall, field studies, as in the first days of ARL's history, promote current knowledge of ongoing activities and stimulate the creative instincts of the staff.

## Changes: Past to Present

What will the next century bring to ARL? When C.M. Allen started working at the Hydraulic Testing Laboratory, he probably could not begin to envision the changes that would take place during his tenure, and certainly not during the first century of the laboratory's existence. The giant changes that occurred in the first century can be summarized as follows:

- school-operated, nonprofit laboratory to private, for-profit consulting company;
- basic teaching laboratory to applied research laboratory;
- slide rule to personal computer;
- hydraulics to computational fluid mechanics;
- hand-acquired data to computerized data acquisition systems;
- still photography to motion pictures and video;
- hand-drawn curve sheets to computer-printed graphs;
- hand-made drawings to computer-generated design drawings;
- mercury hand-held thermometers to data acquisition systems with thermocouples;
- manometers and gauges to pressure transducers;
- current meters to hot wire anemometers, magnetic current meters and laser doppler anemometers;
- Venturi meters to ultrasonic flowmeters;
- calibration throat Reynolds numbers from 2.5 million to 8 million;
- a fewflowmeter calibrations per year to two or three every day;
- travel by horse and buggy to jet aircraft; and,
- handshakes to seal contracts to formal signed contracts with commercial terms and conditions.

Conservation is now a way of life, and many utilities can financially benefit by their sponsorship of better usage of electricity. More efficient lights and more efficient variable speed motors and controllers will control demand. The impact of electric cars may be dramatic on the power industry if the cost, battery-storage and recharging issues can be resolved so that these vehicles can become as convenient as the present motor car with their internal combustion engines.

It is certain that the nuclear power industry will undergo fundamental changes as plants age. Recently, Yankee Rowe in western Massachusetts was decommissioned, and the plant faces disassembling and storage of its nuclear components. Cost estimates have doubled or quadrupled from early estimates. Compounding decommission problems is the fact that the United States still has no storage facilities for spent nuclear power plant fuel components. Utilities will extend the operating life of all plants, particularly nuclear, to avoid large capital investments and decommissioning costs. Power oversupply is also rapidly dwindling. It seems certain that only a few new major steam power plants will be built in the

next 10 to 20 years, which will certainly be the case if the "free-wheeling" of power occurs between regions with excess and insufficient capacity. Gas turbines in modular units seem an effective way to meet demand.

## Predictions

There are a few things that might be predicted for the future of ARL and hydraulic testing laboratories in general. There will be many hydroelectric power plants coming up for relicensing. These plants will need studies, especially with respect to fish passage. With the addition of its environmental group, ARL is in a position to design, model test, install and field test equipment needed to conform to the ever-increasing environmental regulations regarding fish entrainment, deterrence and passage. New concepts for fish-friendly hydropower turbines will be explored. With ARL's floating test platform, on-site evaluation of fish response to sound or lights is possible.

As fossil fuel electric power plants age and the demand for electricity increases in this century, there will be studies required for condenser cooling water systems for replacement plants. Flue gas systems in these plants will undergo extensive modifications to clean exhaust gases before they exit to the atmosphere. Pump intake and other related traditional studies will continue because savvy engineers will still see the cost benefit of building and testing a hydraulic model before the enormous cost of the prototype is incurred. Models will still be used to correct design oversights. No matter how advanced mathematical modeling becomes, some complex flow-related phenomena will be intractable. These phenomena include sedimentation and scour, operation of pumps or intakes in various combinations with real and complex topographical or geometric conditions. Analytical models will become more advanced, will define the problem and will find solutions; however, final verification will require a physical model or depend on coefficients determined in a testing facility.

Fossil-fuel power plants will also be subjected to more laws and regulations regarding liquid and gaseous discharges. These plants will require the services of laboratories, such as ARL, to test and evaluate scrubbers, precipitators and other equipment to comply with environmental regulations. In addition to testing for dispersion of the contaminants, new and precise instrumentation capable of measuring parts per trillion will be used in the studies.

Whatever the fuel source, power plants in the future will need pumps, blowers, compressors, steam generators and maybe condensers. Hydro plants will never entirely disappear. Developing countries will still be building dams that require spillways and all the related testing and optimization.

Critical potable water problems will emerge during the twenty-first century as systems grow older, pollution sources increase and the population grows. Various new methods of recycling used water and super-low or no-water toilets will be invented. More cities will have storm overflow collection systems and large pumping systems.

Optimizing complex mechanical or mixing systems will still require physical modeling. Some issues related to reactor safety will be solved, but others will remain. Complex flow patterns and head losses related to insulation collecting on a strainer can now only be solved with the assistance of testing. In time, critical parameters and coefficients will be developed to allow analytical models to be calibrated.

Parts of the United States experiencing water shortages will be supplied by the parts of the country that have an excess of water using an extensive piping system that incorporates existing rivers and waterways. The design of such a massive system will utilize a multitude of scientific areas such as fluid mechanics, biology, zoology, chemistry, geology and many others due to the interaction of the different ecological systems.

This century should bring further large strides in computational fluid mechanics. More rigorous solutions to the three-dimensional Navier-Stokes equation should evolve during this time, and more solutions to complex fluid problems will be obtained with mathematical models. Computational fluid mechanics courses that are today's studies for graduate students will be considered simple to undergraduate students in the year 2094.

Physical models will still be utilized in the extremely complicated situations after exten-

sive computer optimization. Physical models will be constructed by robotic devices carving out huge slabs from some type of recycled or new plastic material. The robots will be controlled by high-speed computers with input data supplied on some sort of software by the customer. Physical changes in these models will be accomplished in a very short time. Scanning devices throughout the model would obtain temperatures, pressures, velocities, turbulence intensities and any other necessary data without touching the fluid.

Clients might see, at their desks and in real time, the model test or the results from the mathematical model. The test results will no longer be reported in a booklet. Instead, results may be software that the client will be able to view on a very thin portable hand-held screen.

Flowmeter calibrations will be performed using unobtrusive devices that would perform instantaneous complete velocity traverses at different sections of the flowmeter, integrate the traverse, obtain the readout and determine the coefficient of the meter. In fact, flowmeters of the future may be just a special piece of pipe fabricated from material that will act as a flow-integrating "chip."

Who really knows what the future generation holds for hydraulics testing laboratories? One thing for sure, current state-of-the-art equipment and visual documentation will look as antiquated as photographs from the early days of ARL look to us today. If the exponential growth of the past century is a good indicator, this new century should be an exciting and creative time.

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# Vibration Damage Claims: Ingredients for a Successful Investigation

*Following procedures to predict potential ground vibrations, selecting vibration damage criteria and evaluating damage probability are key to conducting a reasonably accurate investigation.*

PAUL L. KELLEY, STEVEN J. DELLORUSSO & CHARLES J. RUSSO

Many construction activities like blasting, pile driving, soil compaction and heavy equipment operation create ground vibrations. When the work is close to existing buildings, occupants will likely experience vibration levels in excess of typical ambient conditions, which often leads to an expectation of property damage and resulting insurance claims. It is almost always very difficult to determine the causal relationship of the construction activity to the alleged damage for the following reasons:

- The claimant's first inspection of the building coincides with heightened awareness created by the actual perception of noticeable or annoying vibration levels.
- The pre-construction conditions of the alleged damage are not documented.
- Vibration measurements are usually not made during construction activities, except for blasting operations.
- The alleged vibration damage is usually cosmetic and involves the cracking of brittle, fragile or aged components like roofing, flashings, plaster wall finishes and interior trim, masonry chimneys, rubble stone foundation walls and unreinforced concrete block walls.

The above conditions provide few hard facts to the engineer investigating damage claims. In the absence of hard facts — such as a pre-construction survey document — the engineer must undertake a multidimensional investigation of the interrelationship of circumstances in order to develop an opinion about the relationship between the construction vibrations and the alleged damage. In a court of law, this determination of cause must usually be expressed as "more probable than not."

A balanced, professional and multidimensional assessment of alleged vibration-re-

**TABLE 1.**
**Vibration Damage Guideline**

**A. Inspect the Damaged Property**
- 1. General Condition of Property
- 2. General Degree of Maintenance
- 3. Viewing of Damages
  - a. Proximity to Construction Operations
    - (1) Other Damaged Properties
- 4. Age of Damage
- 5. Credibility of Claimant
- 6. Comparative Fragility of Damaged/Undamaged Components
- 7. Identify Alternate Damage Hypotheses
- 8. Repair Concepts & Repair Costs

**B. Estimate Vibration Magnitude & Frequency**
- 1. Determine the Construction Parameters
  - a. Types of Equipment & Operations
  - b. Duration of Operations
  - c. Distances from Operations to Damaged Property
- 2. Determine the Site Conditions
  - a. Subsoil Conditions
  - b. Topography
  - c. Water Table
  - d. Alternate Vibration Sources
- 3. Evaluate the Maximum Potential Ground Vibration Magnitude
  - a. Published Data
  - b. Site-Specific Data
  - c. Human Perception

**C. Select Appropriate Damage Threshold Criteria**
- 1. Match to Structure Type
- 2. Match to Component Type
- 3. Identify Probabilistic Basis of Criteria

**D. Determine Vibration Damage Probability**

**E. Evaluate Alternate Damage Hypotheses**

**F. Compare & Rank All Damage Hypotheses**

lated damage claims should focus on the following three major steps:

- The prediction of potential vibration magnitudes using published data and site-specific data;
- The selection of the appropriate damage criteria; and,
- The determination of vibration damage probability.

## Background

An assessment of the causal relationship of damage to architectural materials (such as plaster and wallboard) and minor cracking to relatively brittle structural materials (like concrete and brick/block/stone masonry) to ground vibrations is extremely difficult to conduct. Its difficulty is primarily due to the unknowns associated with the pre-vibration stress conditions and the fact that vibrations develop additional stresses in systems already performing under pre-vibration stresses from normal environmental loadings and aging. The problem requires determining how much additional stress from vibration is necessary to trigger failure or cracking in a material that is potentially on the verge of doing so. Because of numerous unknown factors, this type of analysis is so complex that it is generally not feasible, and empirical damage assessment methodologies must be used.

The probability of damage to a structure from ground vibrations depends on many factors, including the type and condition of the structure, the soil conditions at the site, the energy imparted to the ground, the frequency of the ground vibrations and the distance from the vibration source to the structure under consideration. Predictions or evaluations of vibration damage can be made by comparing actual site measurements with established vibration damage threshold criteria.

## Guideline for a Vibration Damage Investigation

In the absence of a pre-construction survey document that can usually definitively answer the claim, there will always be one party unhappy with the report. A balanced, professional assessment will minimize the contro-

versy. Table 1 presents an investigation guideline that is based on the assessment of numerous vibration damage claims.

## Ground Vibrations From Construction Operations

Recorded vibrations, published vibration data and vibration damage criteria for structures are typically expressed as the peak vibration velocity — either measured directly or derived from an acceleration time–history record of the vibration. Vibration velocities are typically recorded in three orthogonal directions: longitudinal, transverse and vertical. Many regulatory standards and established vibration criteria relate vibrations in terms of peak particle velocity (PPV), which represents the single-direction peak. Sometimes data are reported as the peak vector sum (PVS), which is the maximum resultant of the three orthogonal components occurring at any given time. PVS will always be greater than PPV, but it is only slightly conservative to compare PVS data to PPV criteria.

Vibration measurements are typically made in the ground adjacent to the structure or on a ground-supported slab or the foundation wall. Since comparisons to the data are empirical, it is important that the site measurements be obtained in a similar fashion to the data that are the basis for the criteria. Vibration measurements on loose structural elements, even elevated floors, may amplify the ground vibration, distorting the comparison to empirical "ground" vibration criteria.

## Prediction of Vibration Magnitude

It is difficult to assess the probability that construction-induced vibrations are related to alleged damage in an adjacent structure because vibration measurements are not typically recorded during construction activities, except for blasting operations. In addition, if vibrations are recorded, they are usually not recorded at the structure with the alleged damage.

The engineer investigating the claim must use published vibration data and site-specific vibration data to evaluate the potential maximum ground vibrations at the subject structure. Site-specific data include vibration measurement records taken during construction and/or vibration records made during the investigative phase with similar construction equipment.

*Published Vibration Data.* Several sources have compiled attenuation charts for construction-induced vibrations from a variety of construction equipment. Figure 1 shows a typical attenuation chart that illustrates the vibration-to-distance relationships for different construction equipment.

Similar ground vibration attenuation charts are available for blasting operations. Figure 2 presents an attenuation chart that illustrates the relationship between ground vibration and scaled distance, which is a function of the distance from the blast to the subject structure and the pounds of explosives per delay from the blast shot. Blasting attenuation charts use the square root scaled distance ($ft/lb^{1/2}$) or cube root scaled distance ($ft/lb^{1/3}$); Figure 2 is a square root scaled distance attenuation chart.

As indicated in the investigation guideline, it is important to identify the types of equipment and operations, as well as the approximate distances from the operations to the damaged structure. If the claim is related to a blasting operation, the blaster's log should be available and should document the amount of explosives per delay and the location of the blast shot with respect to the closest structure. Using the log data, the engineer can then determine the relative distances from each blast location to the subject structure either by field measurements or scaling techniques.

Using available vibration data for construction operations and published attenuation charts, the engineer can evaluate the potential maximum ground vibrations. The engineer should also use best judgment as to the possible impact of the soil conditions at the site.

*Site-Specific Vibration Data.* When vibrations from construction equipment or blasting activities are recorded at a site, they are usually not recorded at the subject structure. However, all vibration records can be used to develop a site attenuation chart, which intrinsically accounts for soil conditions. Figure 3 represents an attenuation chart for two construction equipment operations used on a site upon which ground vibrations were recorded during construction

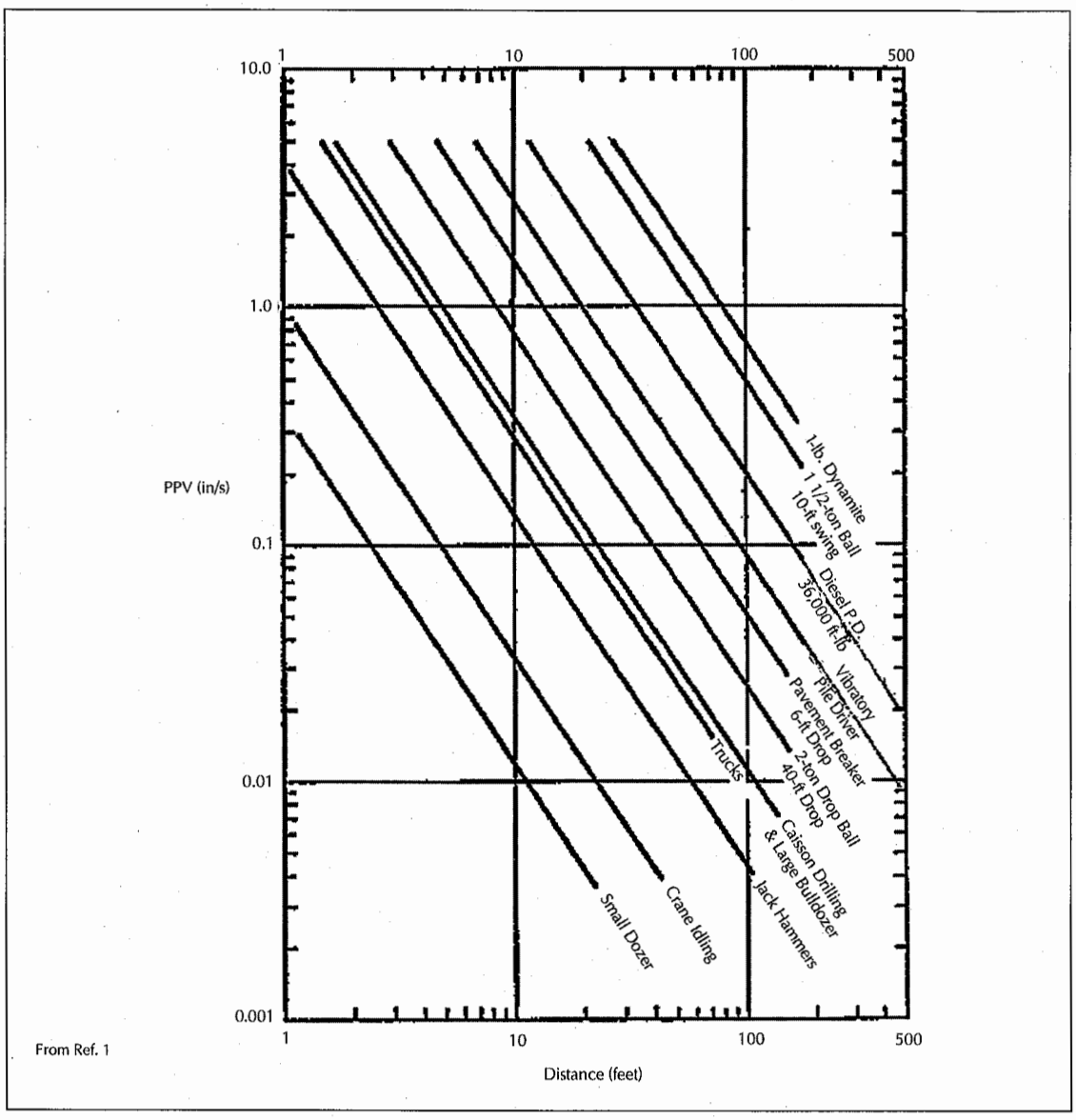


**FIGURE 1. The attenuation of ground vibrations from construction equipment.**

operations. The triangle-marked lines bound the test data from the vibratory compactor, and the square-marked line bound the test data from the ram-hoe. The remaining, unmarked lines are from other published construction vibration data (such as from Figure 1).

Using the site-specific attenuation lines and the approximate distances from the operations to the damaged structure, the engineer can evaluate the potential ground vibration from the construction operations to the subject structure.

For blasting operations, a site-specific attenuation chart is relatively simple to develop. A blaster's log is required to document the location of the blast shot on the site, the distance from the blast to the nearest structure, the amount of explosive per delay and the maximum ground vibration recorded at the nearest structure. In some instances, the blaster may use multiple seismographs to record simultaneous ground vibrations at numerous adjacent properties, not only the nearest properties, providing useful redundant data.

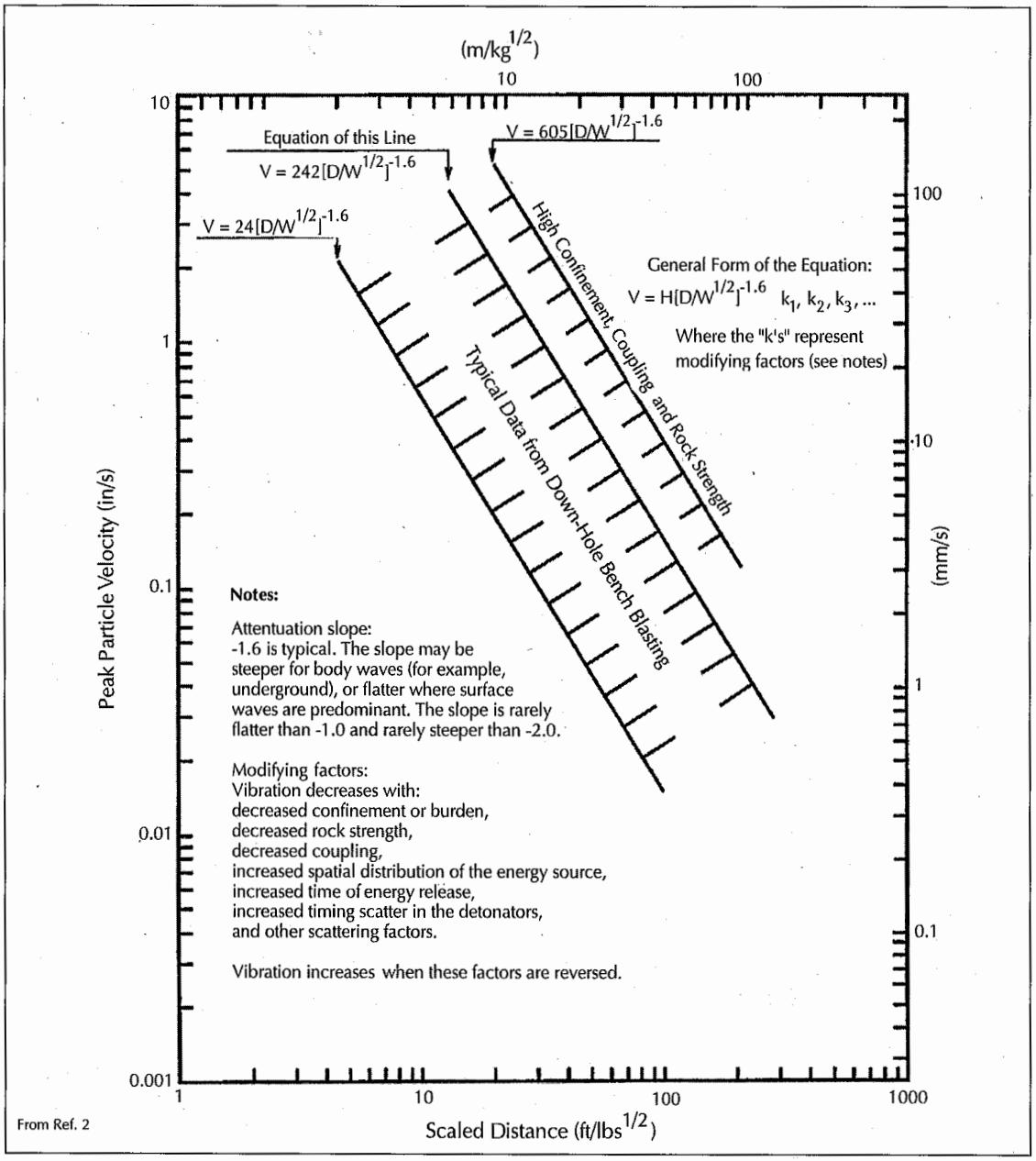


**FIGURE 2. The attenuation of ground vibrations from blasting.**

Using all the available vibration data, it is possible to estimate a site-specific attenuation and to determine upper and lower bounds. However, the data can be quite scattered if the blast contained variable degrees of confinement, and better predictive equations might require the segregation of data from different types of confinement (confined blast versus open-face blasts).

## Selecting the Appropriate Damage Criteria

*Classifications of Vibration Damage.* It is important to note the relevant types of damage before using the various damage criteria. Most of the criteria use terms such as *threshold damage, minor damage* and *major damage.* A good, recent source of vibration-damage criteria

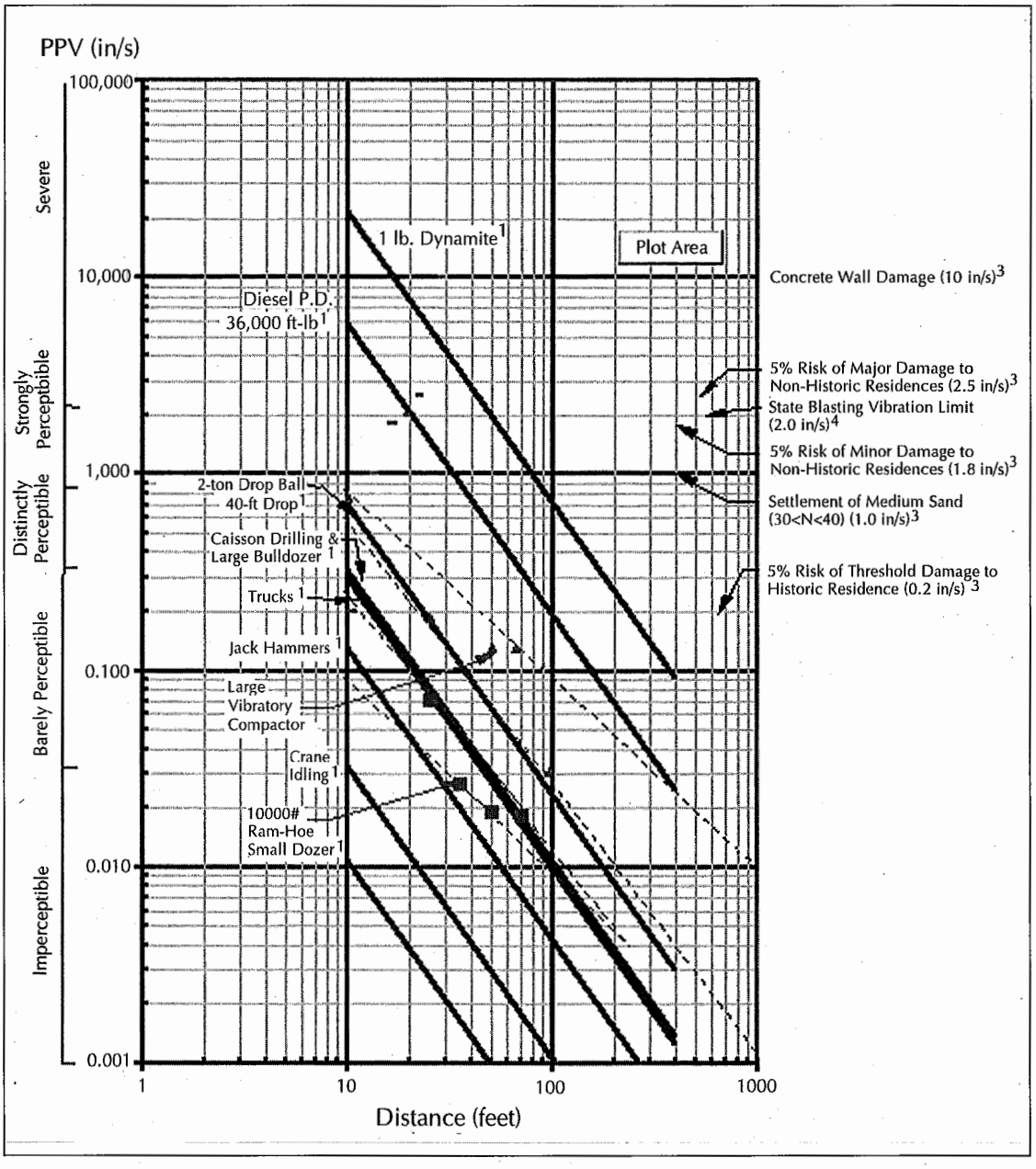


**FIGURE 3. Ground vibrations from various construction equipment.**

and probability is *Blast Vibration Monitoring and Control*, by Charles H. Dowding.[3] In general, the following definitions apply to these terms:

- Threshold damage: loosening of paint, small plaster cracks at joints between elements, lengthening of old cracks.
- Minor damage: loosening and falling of plaster, cracks in masonry around openings near partitions, hairline to 3-millimeter (0 to 0.125-inch) cracks, falling of loose mortar.
- Major damage: cracks of more than 3 millimeters (0.125 inches), rupture of opening vaults (arched roof chambers), fall of masonry (*e.g.*, chimneys), load support ability affected.

**TABLE 2.**
**Swiss Standard SN 640312 Criteria for Construction Vibrations**

| Structural Category | PPV (in./s) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | **I — Industrial** (Reinforced Concrete & Steel Construction Such as Industrial & Commercial Buildings) | | **II — Concrete Framed** (Buildings With Foundation Walls & Floors in Concrete With Walls in Masonry or Concrete) | | **III — Masonry or Wood Framed** (Buildings With Foundations & Basement Floors of Concrete Construction) | | **IV — Vulnerable to Vibrations** (Buildings That Are Especially Sensitive or Worthy of Protection) | |
| Frequency | Steady-State | Transient | Steady-State | Transient | Steady-State | Transient | Steady-State | Transient |
| 10-30 | 0.5 | 1.2 | 0.3 | 0.7 | 0.2 | 0.5 | 0.12 | 0.3 |
| 30-60 | 0.5-0.7 | 1.2-1.6 | 0.3-0.5 | 0.7-1.0 | 0.2-0.3 | 0.5-0.7 | 0.12-0.2 | 0.3-0.5 |

*U.S. Bureau of Mines (USBM) Report of Investigation 8507/1980.* This study provides an often-quoted standard damage threshold.[5] The USBM damage criteria are based on studies of first cracking in residential structures subjected to repeated blasting. The criteria are velocity based, with different allowable velocity levels over different frequency ranges to account for the likely effects of structural resonance at low frequencies. The variable velocity criteria are 0.2 inches per second at 1 herz, increasing linearly to a plateau of 0.75 inches per second (0.5 inches per second in plaster versus drywall) between 4 and 15 herz, and increasing linearly again to a 2.0 inches per second plateau starting at 40 herz. The criteria are limited because the focus is on residential buildings and because the primary basis is on the first cracking of interior plaster and drywall finishes ("minor" damage). The USBM criteria do not account for other types of construction systems that are likely in commercial construction, nor do the criteria address the age of the structure and prior deterioration of the building systems.

*Swiss Standard SN 640312.* A more useful approach is to categorize damage threshold criteria by the type and age of construction. One standard of this type is the "New Swiss Standard for Vibrations in Buildings," as presented by J.F. Wiss in 1980.[1] The Swiss standard is acknowledged to be conservative in deference to the importance placed on property in Switzerland. It most likely represents something on the order of a 1 to 5 percent probability of "minor" damage, based on comparisons of Dowding's damage term definitions, the Siskind blasting damage probability plots, the USBM criteria and the Swiss standard. The Swiss standard lists four building classes ranging from a Building Class IV ("very sensitive to vibrations") to Building Class I ("buildings in steel or reinforced concrete"). Table 2 presents the building classes and associated vibration criteria for both steady-state and transient vibration sources.

To use the standard, the closest comparable building class to the subject structure must be assessed to determine which criteria apply; residential structures are not specifically identified. It is interesting to note that the criteria for "steady-state" vibrations in the Swiss standard are approximately one-half of those prescribed in the same standard for impulse sources like blasting (*i.e.*, Building Class III, steady-state limiting PPV to 0.2 inches per second and transient limiting PPV to 0.5 to 0.7 inches per second). This reduction addresses the greater potential for resonance and fatigue that exists with the multiple period impulses associated with "steady-state" vibrations. This difference implies that the USBM criteria for blasting should be adjusted downward if applied to steady-state vibrations.

The Swiss standard, other similar standards and standards for particular vibration sources are presented in *Vibration Generated by Traffic and Building Construction Activities* by Roger Holmberg, *et al.*[6] One useful damage criterion noted in Holmberg is by Forssblad. The

Forssblad values are for vibratory compactors. Vibratory compaction creates steady-state vibrations, with similar frequencies to railroad traffic (10 to 40 herz). The Forssblad values indicate a 0.20 inches per second limit for "risk for cracking in normal [European] residential buildings with plastered walls and ceilings" and indicate a 0.40 inches per second limit for "risk for damage to normal residential buildings (no plastered walls and ceilings)."

*Structural Damage Criteria.* The above criteria generally apply to damage to architectural finishes. For the new cracking of structural walls, data from *Dynamic Strains in Concrete and Masonry Walls* by Crawford and Ward indicate a damage threshold of 3.0 inches per second for mortar and masonry (rubble) walls.[7] Data reported by Dowding indicate a damage threshold of 10.0 inches per second for concrete walls.[3] These figures are for blasting vibrations and should presumably be reduced by half for application to steady-state sources. The halving of the standard for steady-state vibrations is indirectly supported by criteria reported by Akins and Dixon.[8] Akins and Dixon address the protection of young concrete at sites where simultaneous steady-state construction activities continue. Akins and Dixon also recommend limiting vibrations in 24-hour-old concrete to 0.2 inches per second, in 1-day- to 7-day-old concrete to 0.4 inches per second, and in 8-day and older concrete to 4 inches per second (about half of 10 inches per second).

*Vibration-Induced Settlements.* Concerning soil settlement from vibration, studies by others reported in Dowding indicate that very loose (5< N, blow count <10) cohesionless soils (sands and gravels, with no cementing soils like clay or silt) subjected to vibratory pile driving (steady-state type source) consolidate at vibration magnitudes of 0.17 inches per second.[3] Medium-dense sands (30< N <40), under similar steady-state vibration, were shown to consolidate at vibrations of 1.0 inch per second. These soil densification data are from isolated testing, and the values are probably more indicative of a high probability of occurrence (greater than 50 percent) than the low (5 in 100) probability of the other criteria.

For blasting operations, research reported by Dowding indicates that measurable settlement occurred in loose sands from 11-pound, single-delay blasts outward to a range of 59 feet; for medium-dense sands the range is 46 feet.[3] Using the square root scaled distance technique and the attenuation chart for average peak particle velocities from blasting events produced by the International Society of Explosives Engineers,[2] this 11-pound blast corresponds to a scaled distance of 18 $ft/lb^{1/2}$ and an average peak particle velocity of 1.6 inches per second for loose sands; for medium-dense sands, the same technique and attenuation chart gives a scaled distance of 14 $ft/lb^{1/2}$ and an average peak particle velocity of 2.3 inches per second.

*Buried Structures.* Building structures are not the only structures where vibration damage is a concern. Underground or buried structures — such as building foundations, pipelines, culverts — should also be evaluated. Because underground structures are restrained against free response to vibration excitation, they can withstand particle velocities of much greater magnitude (at least 6 to 8 inches per second) than above ground structures without damage.[9] This restraint tends to limit dynamic displacements of the buried structural elements, which cause cracking strains in the structure, to the ground strains in the surrounding soils as opposed to the unrestrained response of above-ground structures that can selectively amplify ground-based excitation. However, any permanent deformation due to the settlement of the surrounding soils is a separate concern.

*Environmental Vibrations.* Many investigators believe that it is improbable that sub-0.5 inch per second vibrations can be the cause of property damage because many other environmental factors create strain levels consistent with this level of vibration and should be viewed as a "more probable" cause for cracking. *Blasting Vibrations and Their Effect on Structures* (USBM Bulletin 656) reports that studies of vibrations from normal household activities — such as walking, jumping, closing doors and operating laundry equipment like clothes dryers and washers — can create measurable vibrations sometimes approaching damaging levels, but the influence is generally very local.[10] For example, walking is reported to cre-

ate vertical vibration velocity magnitudes in the range of 0.02 to 0.20 inches per second, jumping can create vertical vibration velocity magnitudes in the range of 0.23 to 5.0 inches per second and closing doors can create vertical vibration velocity magnitudes in the range of 0.01 to 0.06 inches per second.

*Human Perception Levels.* One major complication in the assessment of vibration damage claims is the incompatibility of the vibration intensity as perceived and reported by human witnesses to the damage criteria. It is commonly accepted that the threshold of human perception is 0.01 to 0.02 inches per second, and that steady-state exposure to vibrations greater than 0.10 inches per second causes tiredness and annoyance. With the human perception threshold one to two orders of magnitude lower than the damage criteria, and annoyance levels almost one order of magnitude lower, it is not unusual for damage claims to be accompanied by truthful testimonials of "shocking" vibration levels.

Prior to the development of a geological instrumentation network to monitor and record ground motion, the intensity of seismic events was assessed on the basis of human perception and observed building damage. The most commonly used reaction-based measure of intensity is the Modified Mercalli Intensity Scale (MMIS). The MMIS has twelve categories for ground motion intensity based on human perception and observed building damage. Table 3 is a reproduction of the MMIS with category ratings and the associated descriptions of the reactions. In addition, Table 3 presents a correlation of the MMIS to ground velocity that can be useful in trying to apply human descriptions to the puzzle of vibration-intensity information.

## Determining Vibration Damage Probability

The probability of damage to a structure from ground vibrations depends on many factors, including the type and condition of the structure, the soil conditions at the site, the energy imparted to the ground, the frequency of the ground vibrations and the distance from the vibration source to the structure under consideration. Predictions or evaluations of vibration damage can be made by comparing actual site measurements with established vibration damage threshold criteria.

In addition, it must be understood that exceeding the vibration damage threshold levels does not confirm that the vibration is the cause of the damage, but only that the vibration is a measurably probable cause.

*Published Damage Probability.* Charles H. Dowding's book *Blast Vibration Monitoring and Control* contains a discussion of a probabilistic study by the USBM on the relationship between ground vibrations and the development of the three damage classifications discussed earlier.[3] Figure 4 represents the results of the damage probability analysis.

The probability chart can be used to evaluate the likelihood of damage for a selected damage criteria, as well as the assessment of damage probability for a given damage classification. For example, for a given blasting operation, the probability chart indicates that a 5 in 100 chance of damage requires 0.5 inches per second for "threshold" damage, 1.8 inches per second for "minor" damage and 2.5 inches per second for "major" damage. Almost four times the "threshold" damage level is required for "minor" damage. For a "more probable than not" chance of blasting damage (greater than 50 percent), the corresponding blast damage vibrations are 2.5, 4.8 and 5.9 inches per second, respectively.

*Evaluation of Vibration Damage Probability.* Using the predicted maximum ground vibrations, the damage probability chart and visual observations of the alleged damage, an engineer can assess the likelihood that there is a relationship between the alleged damage and the construction operations. The assessment is typically expressed terms of "improbable," "probable" and "more probable than not."

## Conclusions

It is difficult to make an assessment of the causal relationship between alleged damage to architectural materials and structural components and construction-induced vibrations. The investigation usually lacks documentation of pre-existing conditions and vibration measurements at the subject property over the course of construction operations. The investigating engineer must undertake a

**TABLE 3.**
**Modified Mercali Intensity Scale**

| Equivalent Magnitude[11] (Eastern North America Scale) | Equivalent Peak Ground Velocity[12] (in./s) | Modified Mercali Intensity Scale | Description of Reaction & Damage From Vibration |
|---|---|---|---|
| 1.0-3.0 | 0.06 | I | Not felt except by a very few under especially favorable conditions. |
| 3.0-3.9 | 0.11 | II | Felt only by a few persons at rest, especially on upper floors or buildings. |
| | 0.22 | III | Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated. Hanging objects swing. |
| 4.0-4.9 | 0.45 | IV | Felt indoors by many; outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticebly. Hanging objects swing. |
| | 0.90 | V | Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop. |
| 5.0-5.9 | 1.80 | VI | Felt by all; many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight. |
| | 3.60 | VII | Damage negligible in buildings of good design & construction; slight to moderate in well built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken. |
| 6.0-6.9 | 7.20 | VIII | Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. |
| | 14.40 | IX | Damage considerable in specially designed structures; well designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. |
| 7.0+ | | X | Some well built wooden structures destroyed. Most masonry & frame structures destroyed with foundations. Rails bent. |
| | | XI | Few, if any, (masonry) structures remain standing. Bridges destroyed. Rails bent greatly. |
| | | XII | Damage total. Lines of sight & level are distorted. Objects thrown into the air. |

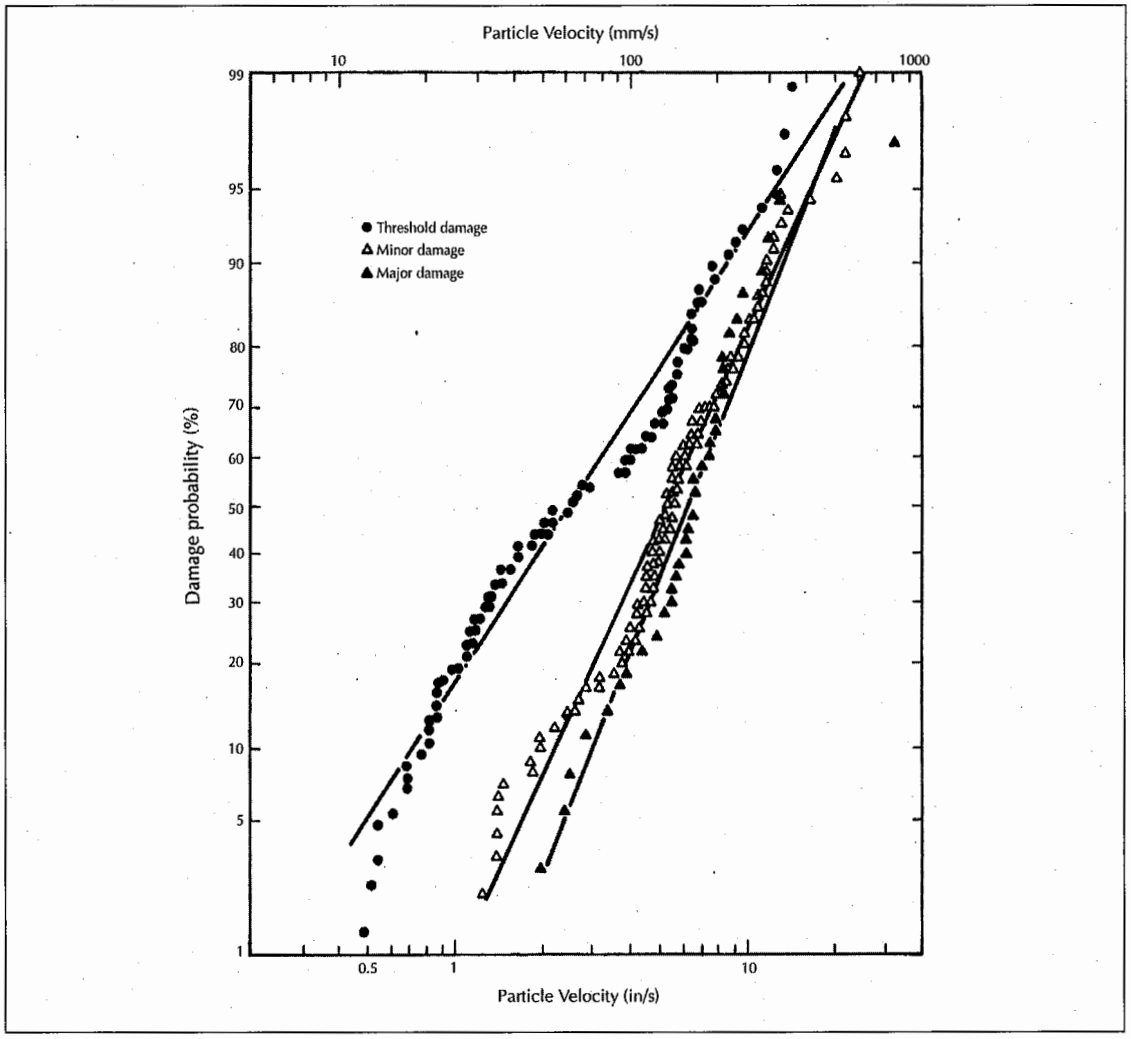


**FIGURE 4. Damage probability from ground vibrations.**

multidimensional assessment to evaluate the likelihood of the relationship of construction vibrations to damage.

The guideline for investigation and methodology for the prediction of potential ground vibrations (see Table 1), the selection of vibration damage criteria and the evaluation of vibration damage probability provide the investigator with a balanced and professional approach to assist clients in assessing the validity of claims.

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# Measures to Minimize the Effects of a Deep Excavation on Two Adjacent Office Buildings: The Abutters' Perspective

*For a large-scale excavation project, effective communication provides the foundation to lessen the impact on adjacent structures.*

LEWIS EDGERS, RICHARD HENIGE, JR., THOMAS L. WEINMANN & KENNETH B. WIESNER

The main feature of Section C11A1 of the Boston Central Artery/Third Harbor Tunnel (CA/T) Project consists of a 630-meter (2,065-foot) long, three- to five-lane cut-and-cover highway tunnel for the northbound central artery (CANB), which is overlain by a bus transitway tunnel. Along this excavation route, the intersection of Atlantic Avenue and Summer Street (Dewey Square) presents a number of challenges in addition to a dense array of utilities typical of many urban intersections (see Figure 1). The excavation for the CANB tunnel alignment along Atlantic Avenue reaches its greatest depth — approximately 34 meters (110 feet) — in order to pass beneath the Massachusetts Bay Transit Authority (MBTA) Red Line, an active subway line carrying more than 300 trains per day beneath Summer Street. Major structures located at this intersection include:

- South Station Headhouse, a terminal for commuter rail service, at the southeast corner. The headhouse is a five-story stone structure built in 1899 and supported by 7.6- to 12.2-meter (25- to 40-foot) long timber piles.
- MBTA Red Line Station at South Station, crossing the alignment beneath Summer Street with access kiosks at four corners and a number of below-grade pedestrian tunnels. The subway station at this intersection has now been underpinned, a major undertaking.

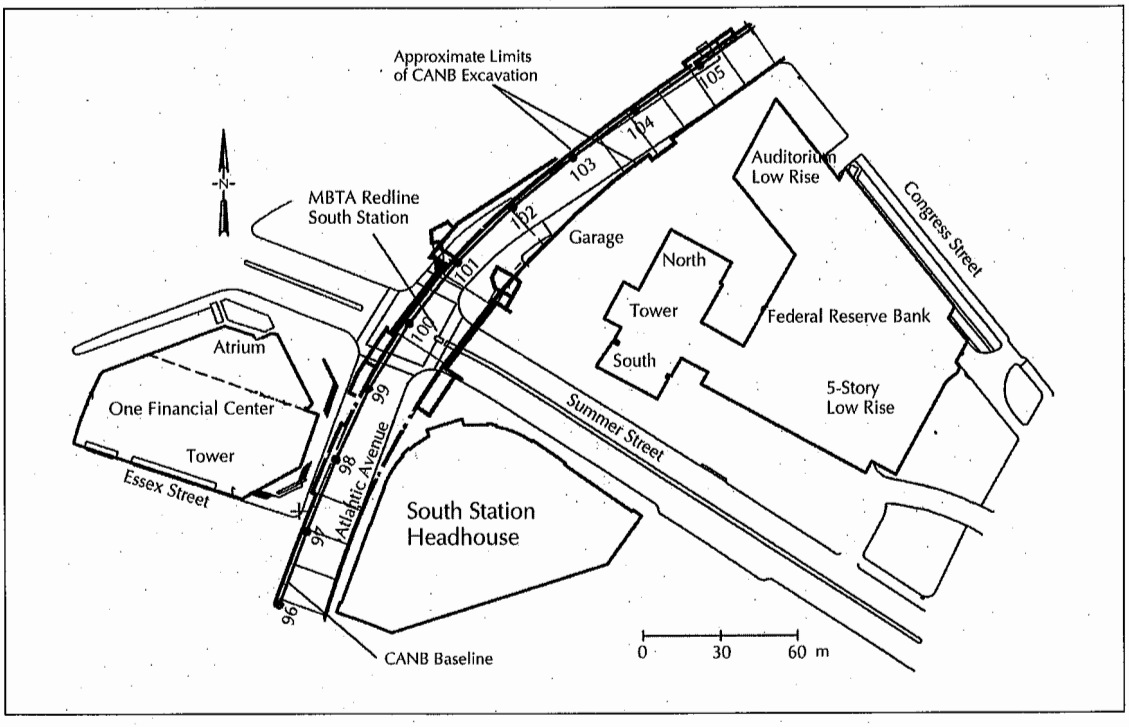


**FIGURE 1. The location of the Federal Reserve Bank and One Financial Center.**

- Federal Reserve Bank of Boston (FRB), a 33-story office tower at the northeast corner.
- One Financial Center (OFC), a 46-story office tower at the southwest corner.
- Dewey Square vehicular tunnel (not shown on Figure 1), located west of the CANB alignment.

Since excavation-induced ground movements, even if small, may have large effects on large office buildings, measures were developed to minimize the effects of excavations for the CA/T Project on the FRB and OFC.

## Description of the FRB & OFC Building Complexes

*Federal Reserve Bank (FRB).* The FRB complex consists of a number of structures built from 1972 to 1976. The tower structure is supported by two reinforced concrete mats (north and south tower mats), bearing at elevation 22.27 meters (73 feet), CA/T Project datum. For reference, the ground surface elevation is about 34.47 meters (113 feet). The complex's four-story auditorium is a steel-framed structure, which is supported over a two-level common basement that extends beyond the buildings to the site limits bounded by Atlantic Avenue, Summer Street, Dorchester Avenue and Congress Street. The half of the basement near Atlantic Avenue has two levels and is supported by spread footings founded at elevations ranging from 23.73 to 23.27 meters (77.8 to 76.3 feet). The first basement slab is a two-way concrete flat slab supported on steel columns, while the second basement slab is a mesh-reinforced slab on grade. The rear half of the complex consists of a five-story low-rise office structure underlain by one basement and supported by reinforced concrete drilled piers. This low-rise structure in the rear half of the FRB complex is beyond the zone of influence of the CA/T Project excavation and will not be considered further. All the FRB foundation mats, spread footings and drilled piers bear on glacial till.

Figure 2 illustrates the subsurface conditions beneath the FRB complex near the CA/T Project alignment. The tower mats, as well as the garage and auditorium footing foundations, bear in the T2 cohesive till that consists of a very dense mix of gray silt and fine to coarse

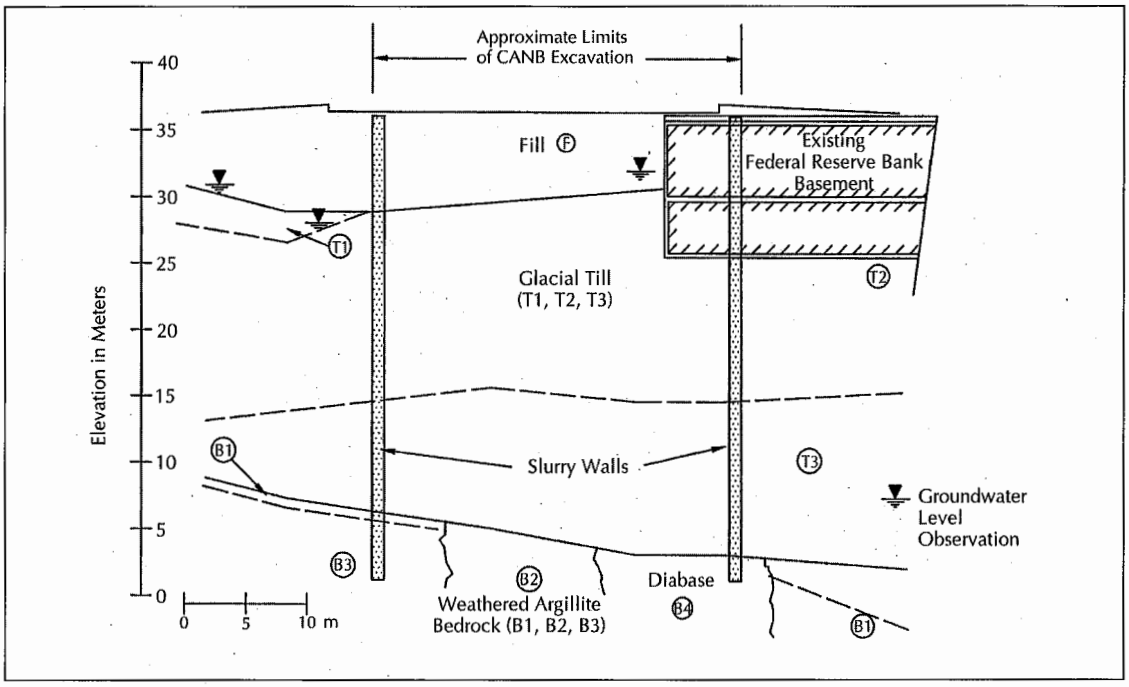


**FIGURE 2. Subsurface conditions at the FRB (station 101+40 CANB).**

sand, little fine gravel, little clay and occasional cobbles and boulders. The T2 till is underlain by a very dense granular till (T3) and bedrock. The bedrock ranges from completely weathered, very soft argillite (B1) to slightly to moderately weathered hard argillite (B3) and a moderately hard to soft, slightly to severely weathered diabase (B4).

*One Financial Center (OFC).* Built in 1981, the OFC building consists of a 46-story high-rise structure with an adjoining atrium. Both structures are underlain by a two-story basement that is used as a garage and that also contains maintenance and utility areas. The tower structural system is a steel tube frame constructed of heavy rolled steel columns and beams. The columns are socketed into a nominal 1.83-meter (6-foot) thick reinforced concrete mat that is locally thickened at the interior columns to 2.6 meters (8.5 feet). The mat bears approximately at elevation 23.79 meters (78 feet). For reference, the ground surface elevation is about 34.47 meters (113 feet). The atrium structure is on the northern side of the site adjacent to Summer Street. It is a glass-enclosed space truss, approximately 26.8 meters (88 feet) high, and it is supported by individual spread footings and a continuous perimeter wall footing. A 15-centimeter (6-inch) slab on grade serves as the basement floor, which is contiguous with the mat surface beneath the tower structure.

Figure 3 illustrates the subsurface conditions beneath the OFC building near the CA/T Project alignment. The tower mat and atrium footings bear in the T2 cohesive till, similar to the till beneath the FRB. The T2 till is underlain mostly by the slightly to moderately weathered hard argillite (B3), with only a thin layer of the completely weathered very soft argillite (B1).

## Design of the CA/T Project Adjacent to the FRB & OFC

Work on the CA/T Project adjacent to the FRB and OFC buildings extends from approximately CANB station 97 (near Essex Street) to approximately CANB station 106+70 (near Congress Street). Most of this work (up to station 104+87) is being completed as part of CA/T Project Contract C11A1. The remainder of the work up to Congress Street will be built as part of contract C17A1 (but was designed as part of the C11A1 effort). The MBTA bus transitway (located above and generally within the CANB alignment) will be con-

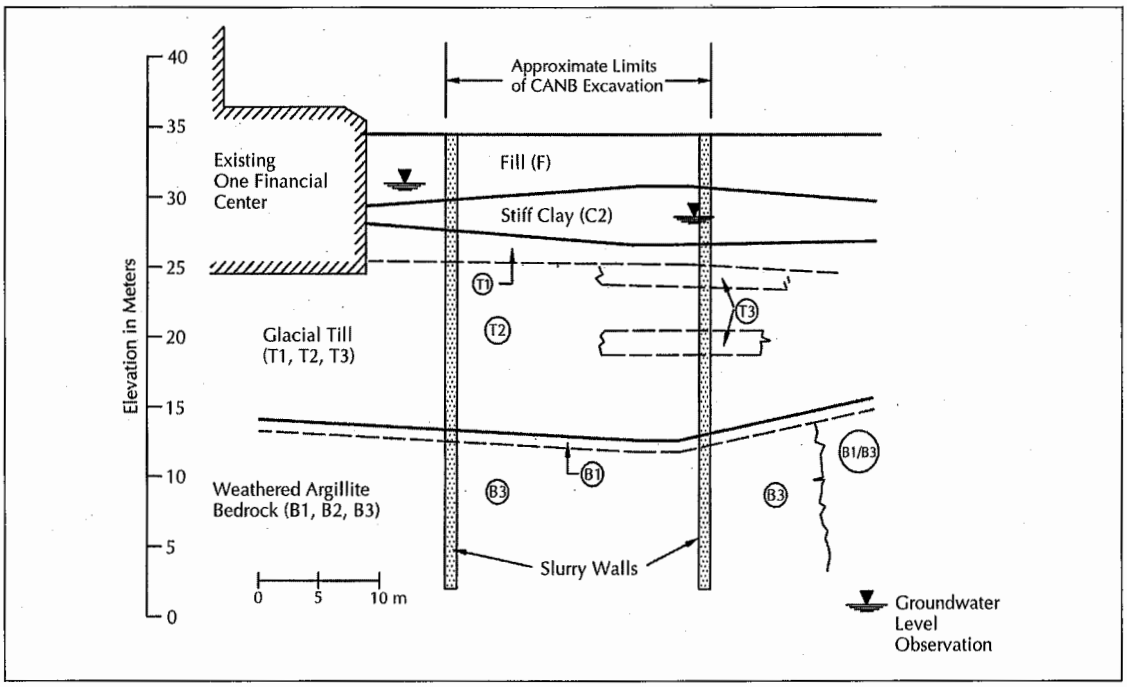


**FIGURE 3. Subsurface conditions at OFC (station 97+00 CANB).**

structed as the CANB tunnel is being backfilled and supported above its roof slab.

The CANB C11A1 tunnel alignment reaches its maximum excavation depth of about 33.55 meters (110 feet) below ground surface as it crosses beneath the Red Line at Summer Street, and the depth gradually decreases to about 30.5 meters (100 feet) near Congress Street. The alignment stays generally within the Atlantic Avenue right of way except for an approximately 46-meter (150-foot) length between stations 101 and 102+50 that intersects the corner of the FRB garage. This portion of the FRB garage has been demolished, resupported by temporary columns to support adjacent sections of the garage and will be reconstructed after the CANB tunnel has been completed. The alignment passes within about 6 meters (20 feet) of OFC.

The design cross sections of Figures 2 and 3 show the support walls and CANB tunnel limits in relation to the FRB and OFC buildings, respectively. The excavations extend more than 21.4 meters (70 feet) below the foundation levels of both buildings. The support walls are soldier pile tremie concrete (SPTC), nominally 1.07 meters (3.5 feet) thick, excavated using slurry techniques. The SPTC walls extend into till or bedrock and will become part of the permanent structure. They are supported during excavation by wales and cross-lot struts, with strut spacings varying from about 3.05 to 3.66 meters (10 to 12 feet) vertically and 4.88 to 6.1 meters (16 to 20 feet) horizontally. It is the contractor's responsibility to design the internal bracing system for the excavation support walls. The contract documents have provided excavation support system design criteria (including lateral earth pressure diagrams) and have suggested construction sequences for cut-and-cover tunnel construction that is intended to restrain lateral wall movement to the maximum extent possible.

The initial studies of the site conducted by the CA/T Project's area geotechnical consultant (AGC) for this contract included preliminary estimates of excavation-induced ground movement along the C11A1 alignment.[1] Calculations were made using the semi-empirical approach developed by Clough and O'Rourke[2] and by the finite element computer program SOILSTRUCT for a generalized glacial till profile. The semi-empirical approach consisted of first estimating the maximum horizontal wall movement (about 0.15 percent of

**TABLE 1.**
**Summary of Preliminary Estimated Displacements of the FRB and OFC Resulting from Cut-and-Cover Tunnel Construction**

| Structure | FRB Low Rise | FRB High Rise | OFC |
|---|---|---|---|
| Distance From Excavated Support Wall | | | |
| Front of Structure (m) | 0 | 25.6 | 7 |
| Back of Structure (m) | 39 | 58.6 | 68 |
| Excavation Depth (m) | 28.4-32.6 | 31.4 | 32.7-33.2 |
| Estimated Excavation-Induced Movement | | | |
| Empirical Methods | | | |
| Maximum Lateral Wall Movement (cm) | 3.8-6.4 | 3.8-6.4 | 3.8-6.4 |
| Settlement at Front of Structure (cm) | 2.8-4.6 | 1.0-1.8 | 2.5-4.1 |
| Settlement at Back of Structure (cm) | 0.3 | 0.0 | 0.0 |
| Lateral Movement at Front of Structure (cm) | 2.8-4.6 | 1.0-1.8 | 2.5-4.1 |
| Lateral Movement at Back of Structure (cm) | 0.3 | 0.0 | 0.0 |
| Maximum Horizontal Strain (×0.001) | 0.7-1.1 | 0.7-1.2 | 0.6-1.0 |
| Finite Element Method | | | |
| Maximum Lateral Wall Movement (cm) | 2.3-3.3 | 2.3-3.3 | 2.5-3.6 |
| Settlement at Front of Structure (cm) | 0.5-0.8 | 0.5-0.8 | 1.0-2.0 |
| Settlement at Back of Structure (cm) | 0.3-0.5 | 0.3-0.5 | 0.3 |
| Lateral Movement at Front of Structure (cm) | 2.0-2.8 | 1.3-1.8 | 1.5-2.3 |
| Lateral Movement at Back of Structure (cm) | 0.8-1.0 | 0.3-0.5 | 0.3 |
| Maximum Horizontal Strain (×0.001) | 1.7-2.4 | 0.6-0.8 | 0.2-0.3 |
| Estimated Maximum Angular Distortion | | | |
| Empirical Methods (×0.001) | 0.7-1.1 | 0.7-1.2 | 0.6-1.0 |
| Finite Element Method (×0.001) | 0.2-0.3 | 0.2-0.3 | 0.3-0.4 |
| Allowable Angular Distortion (×0.001) | 2.0 | 2.0 | 2.0 |
| Building Damage Potential | Slight | Very Slight | Slight |

From Ref. 3.

the excavation depth for these predominantly glacial till profiles) and then estimating the magnitude and distribution of horizontal and vertical (assumed equal to horizontal) ground movements away from the excavation. Both methods assumed that building displacements will be equal to free-field ground movement at the foundation bearing levels.

Table 1 summarizes the preliminary estimates by the section designers of the cut-and-cover excavation-induced ground movements of OFC and FRB using the two methods described above.[3] These estimates did not include consolidation settlement due to groundwater lowering.

Table 1 also compares the corresponding excavation-induced angular distortion compared with the allowable angular distortion of 0.002, based on CA/T Project design criteria. Finally, Table 1 presents the building damage potential resulting from the estimated angular distortions and horizontal strains. These potentials are estimated from the well known relationships established by Boscardin and Cording.[4] The damage potential was estimated to be slight (for the FRB auditorium low rise and OFC) to very slight (the FRB high rise), which suggests that the effects of the CANB excavation on the FRB and OFC may be small.

## Communication Between the Abutters & the CA/T Project

The management of the FRB and OFC complexes assembled their own teams of consultants to work with the CA/T Project management consultant to develop measures to minimize the effects of the C11A1 excavation on their buildings. Formed during the early stages of the C11A1 design in 1992 and 1993, these teams developed a number of tactics that have been employed in this effort. The CA/T Project management consultant has regularly transmitted for FRB and OFC consultant review key documents such as:

- Draft contract plans and specifications as the bid documents were developed over a number of years;
- CA/T Project section design consultant reports;
- CA/T Project contract addenda;
- Contractor submittals such as support-of-excavation design and value engineering proposals; and,
- All relevant instrumentation data.

Some specialized work within the buildings (such as the required structural modifications in the FRB garage) has been contracted directly by the building owners, with reimbursement by the project. A most important tactic has been the establishment of regular meetings attended by the FRB and OFC consultants, the CA/T Project management consultant and section design consultant (SDC), as well as contractor representatives whenever appropriate. These working group meetings occur every one or two months (more frequently if needed), and, depending on the commonality of the agenda, the FRB and OFC meetings have been held either separately or combined. At various times, project representation has included design, instrumentation, construction and management personnel.

These meetings have led to a number of measures to minimize the effects of the CA/T Project excavation on the FRB and OFC buildings. These measures include:

- Additional ground movement and structural analyses of the FRB and OFC.
- Additions to the CA/T Project instrumentation program.
- Special C11A1 contract provisions such as strict limits on the longitudinal extent and time duration of unsupported excavation between bracing levels and a contingency plan that includes additional bracing levels and tieback support that may be implemented if instrument response values are reached. (The instrumentation specification includes threshold and limiting values for all FRB and OFC instruments that may trigger responses ranging from more frequent readings to modifications of construction procedures and, in the extreme, to the implementation of the specified contingency plan.)

## Additional Ground Movement & Structural Analyses

The CA/T Project management consultant and the SDC decided to perform site-specific analyses of the FRB and OFC because of the approximate and generalized nature of the preliminary analyses, the lack of consideration of soil-structure interaction effects, and the size and importance of the FRB and OFC buildings. The building damage potential relationships established by Boscardin and Cording do not apply to large modern buildings.[4] Because they are highly indeterminate, some large modern buildings are sensitive to even small differential foundation movements. On the other hand, the benefits of structural creep and relaxation on building response are unknown. There are virtually no data available on the response of large structures like the FRB and OFC to excavation-induced ground movements.

*FRB Analyses.* The site-specific FRB analyses evaluated the structural response to excavation-induced ground movements at three cross sections as follows:[5,6]

- CANB station 102+00 — through the south tower of the high rise to the slurry wall.
- CANB station 103+55 — through the north tower of the high rise to the slurry wall.
- CANB station 105+50 — through the auditorium and basement electrical area to the slurry wall.

The SDC first computed the free-field soil movements using SOILSTRUCT and the soil properties recommended in the AGC report, assuming that the groundwater would be maintained at preconstruction levels.[1] The finite element program ANSYS was then used to evaluate soil/structure interaction effects. ANSYS includes a non-linear gap interface element with friction along the interface. Figure 4 shows typical foundation displacements computed by these soil-structure interaction analyses. A comparison of the soil-only and soil/structure analyses indicates that the foundation footings and tower mat have minimal impact on vertical displacements but that the tower mat greatly smooths out the horizontal displacements. A number of structural models were then developed to evaluate the effects of these foundation movements on the FRB structures. These included a two-dimensional plane strain model with lumped column and beam properties as well as a comprehensive three-dimensional model for the north tower. These analyses concluded that:[5,6]

- The foundation movements will not compromise the safety of either the foundation or the superstructure.
- The two-story below-grade garage structure may experience some degree of structural distress when subjected to the estimated ground movements. This distress may consist of cracking, which may develop in the first and second basement slabs; additional cracking may appear in the foundation walls at abutting corners. There is marginal strength in the connection between the steel column and the two-way flat concrete slab at the first basement level to develop the bending-type loads imposed by the laterally displaced foundation and column base.
- The high-rise structure may experience some minor problems due to the displacement of the north tower mat. Cracking of the first basement and ground level slabs may occur.
- The deflection of the upper stories of the tower due to the base movements is within acceptable limits.
- The auditorium structure can sustain the movements; however, there is the possibility of localized distress developing in certain regions that may cause cracking of slab, wall and architectural elements.

As a result of these studies, a number of the connections between the steel column and concrete slab at the first basement level have been reinforced. These studies have also served as a basis for the FRB instrumentation program, with particular emphasis on the possible structural distress of the two-story below-grade garage when subjected to the estimated horizontal ground movements.

*OFC Analyses.* The site-specific OFC analyses evaluated the structural response to excavation-induced ground movements at three cross sections as follows:[7]

- CANB station 97+25 — through the south side of the high rise adjacent to Essex Street.
- CANB station 98+50 — through the north side of the high rise.
- CANB station 98+60 — through the atrium area.

As in the FRB analyses, the SDC first computed the free-field soil movements using SOILSTRUCT and the soil properties recommended in the ACG report, assuming that the groundwater would be maintained at preconstruction levels.[1] The SDC then used the finite element program ANSYS to evaluate soil/structure interaction effects. As noted in the FRB analyses, the OFC tower mat foundation also significantly smooths out the excavation-induced horizontal displacements; however, it has little effect on the vertical displacements. In the atrium, the computed vertical and horizontal displacements of the individual spread footings parallel the soil movement, indicating that the atrium structure (with its spread footings and simple framing) exerts a minimum restraining influence on the soil medium.

The SDC then used the GTSRUDL structural engineering software to evaluate the response of the OFC mat, tower superstructure and atrium to the computed excavation-induced movements. These analyses concluded that:[7]

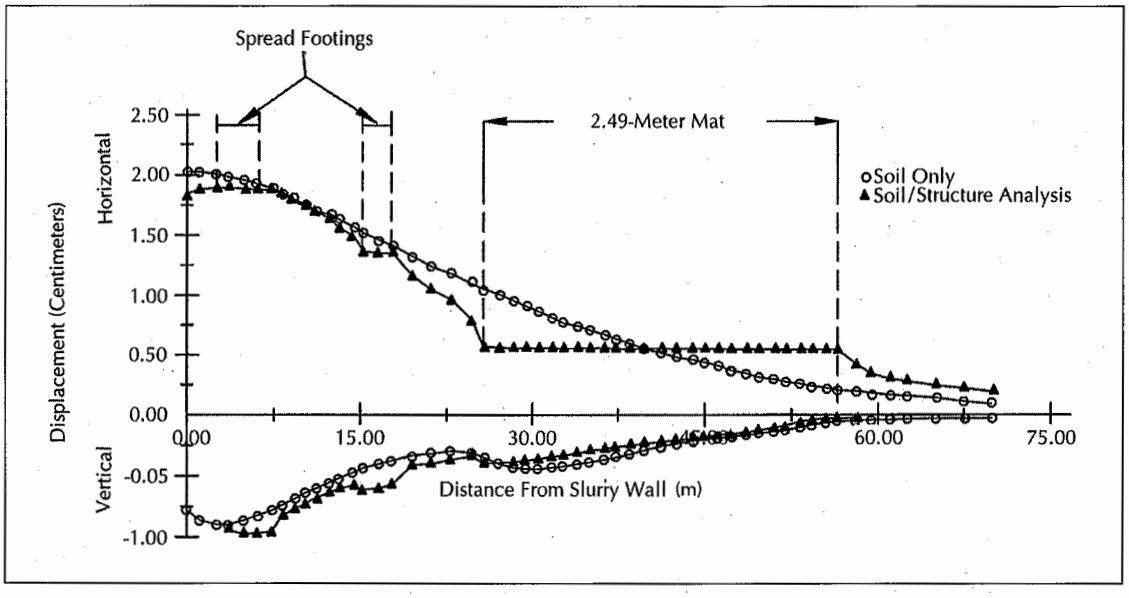


**FIGURE 4. Foundation soil interaction for the FRB.**

- The computed soil movements under the OFC building will not compromise the safety of either the foundation or the superstructure.
- The mat has sufficient capacity to carry the moments induced by the vertical displacements. However, some flexural cracking may occur.
- Some tower floor girders may experience additional excavation-induced stresses up to 27,580 kN/m$^2$ (4 ksi). However, these additional stresses will not pose any threat to the structural integrity of the structure.
- A bolt failure or concrete bearing seat failure may occur where the steel framing of the atrium connects to the foundation wall.
- The excavation-induced stresses in the atrium superstructure are acceptable.
- Some distress of the building's architectural components may occur, resulting in the cracking of wall and floor finishes. This cracking will most likely occur at the interface of the atrium and high-rise tower.

These studies have served as a basis for the OFC instrumentation program, with particular emphasis on the effects of vertical ground movements on the OFC structure.

## Instrumentation Programs

CA/T Project designers established comprehensive instrumentation programs to monitor the performance of the excavation support system and the detailed response of the FRB and OFC buildings and their foundations to the CA/T Project construction. The instrumentation programs also includes a management plan for collecting, reducing, processing, plotting, reporting and interpreting the monitoring data.

Figure 5 presents the locations of instrumentation installed in the FRB building. These instruments include:

- deformation monitoring points (DMPs) in the lower garage for the precision survey of horizontal (29 points) and vertical (65 points) movements;
- thirty-three convergence gage anchorage points (CGAPs) to measure relative horizontal movements of select garage columns, the exterior garage wall and main tower mats;
- three inclinometers (INC) to measure horizontal garage and ground movements;
- four seismographs (SM) to measure vibrations in representative and/or critical locations;
- two hundred and eighty vibrating wire strain gages (VWSG) backed up by 280

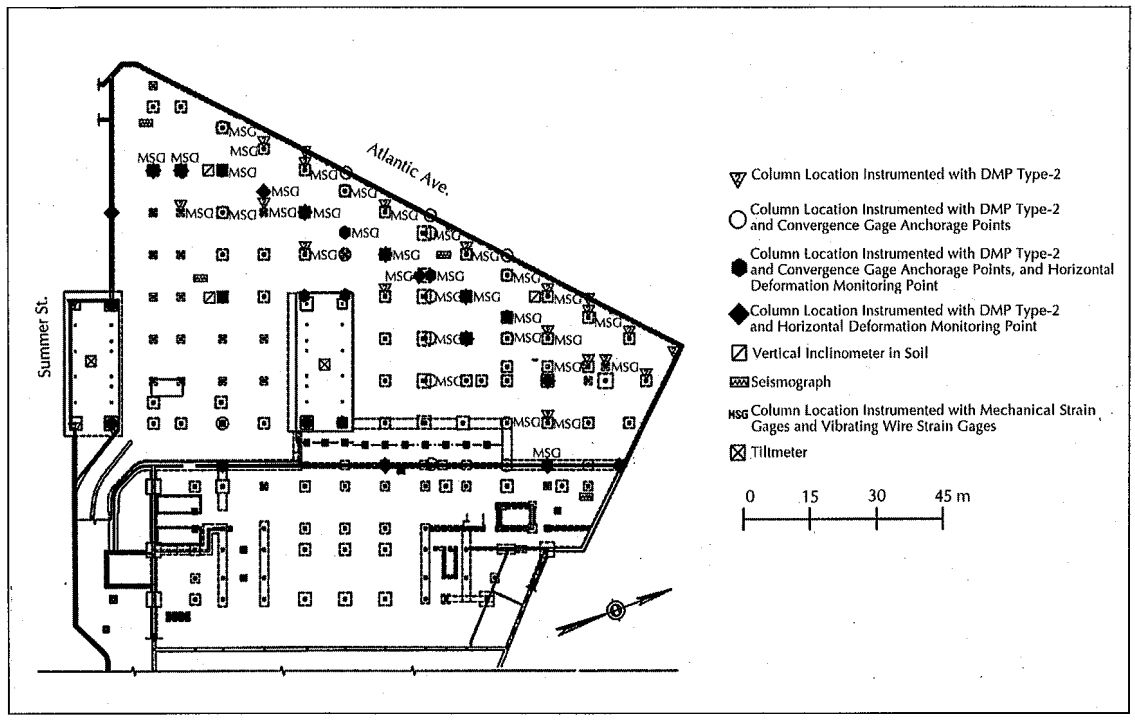


**FIGURE 5. Instrument locations for the FRB.**

pairs of mechanical strain gages (MSG) to measure bending strains at select column locations in the basement levels just above and below the B1 floor slab; and,

- two tiltmeters to measure bidirectional tilt of the tower mat foundations.

Figure 6 presents the locations of instruments installed at OFC. These instruments include:

- seven DMPs in the lower garage for precision survey of vertical movements;
- six CGAPs to measure relative horizontal movements of select garage columns;
- six shear displacement gages (SDGs) to measure the relative horizontal movement between the foundation mat and the atrium slab on grade;
- five single-position borehole extensometers (SPBXs) to measure the settlement of the foundation mat;
- one dial gage (DG) to measure the relative horizontal movement between the beam and exterior wall at column grid location J-1 in the lower garage; and,
- twenty-one tiltmeters (TMs) to measure bidirectional tilts of the foundation mat.

Also, 72 VWSGs (backed up by 72 pairs of MSGs) were installed to measure strains at select beams and columns throughout the structure.

After the FRB and OFC instruments were installed, significant construction activities included:

- utility relocations;
- Red Line stairway and escalator excavations;
- the demolition of part of the FRB garage that intersects the CA/T Project alignment; and,
- slurry wall construction.

These activities have provided baseline data and an opportunity to evaluate the performance of the instrumentation system.

CA/T Project excavation adjacent to the FRB and OFC, as well as the building instrumentation systems, were designed under the assumption that there would be only a small amount of groundwater lowering in the area

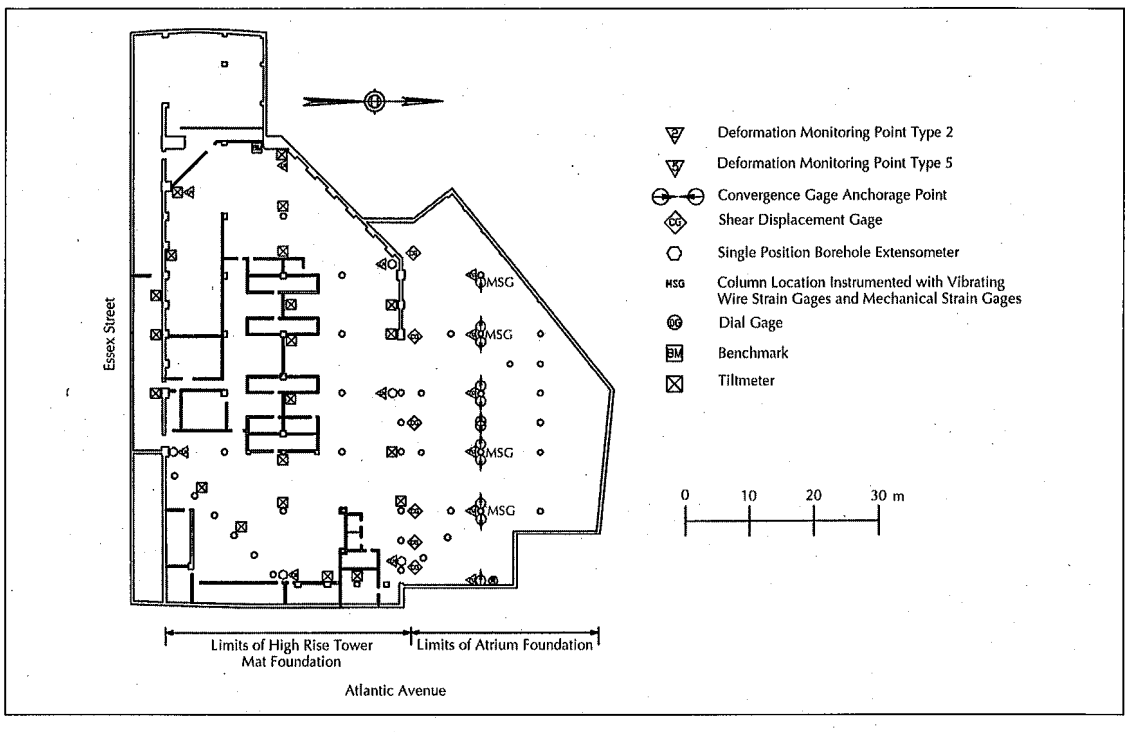


**FIGURE 6. Instrument locations at OFC.**

during construction. However, the underpinning of the MBTA Red Line South Station subway station required substantial construction dewatering, lowering the piezometric surface to below elevation 6 meters (20 feet) within the CA/T Project alignment. The initial baseline measurements — especially when the groundwater was lowered for the Red Line underpinning — suggested that there was a need for additional instruments. At the FRB, these instruments have included DMPs for vertical survey, additional CGAPs in the garage and piezometers installed below the garage. At OFC, additional SPBXs and one multiple position borehole extensometer (MPBX) have been installed to measure the settlement of the foundation mat and to serve as deep benchmarks for the vertical survey. Also, an extensive array of vertical DMPs has been added to replace the TMs that performed poorly because of moisture from the OFC garage floor wash water.

Representative data are presented in Figures 7 through 10. The representative FRB inclinometer data presented in Figure 7 reveals a four-year record of excellent repeatability and very small horizontal movements accumulated to date. Representative FRB tiltmeter data in Figure 8 shows mostly the effects of seasonal variations with little effect caused by the adjacent construction activities. Figure 9, an OFC DMP settlement profile, illustrates the precision achieved by these high-order DMP surveys. Representative OFC strain gage data in Figure 10 depicts the strain gage response of the structure to both construction activities and seasonal effects. Computer-controlled systems continuously monitor the strain gages at both buildings and will trigger an electronic 24-hour alarm that notifies a list of key individuals if readings exceed contract-specified response values. These data selections illustrate the quality of data that the instrumentation has provided during the main CA/T Project excavation.

## Discussion

The groundwater lowering in Dewey Square for the Red Line underpinning has significantly affected the building and instrument response to the CA/T Project construction. The FRB and OFC have settled, as illustrated by Figure 9, mostly due to the compression of the underlying soils caused by groundwater

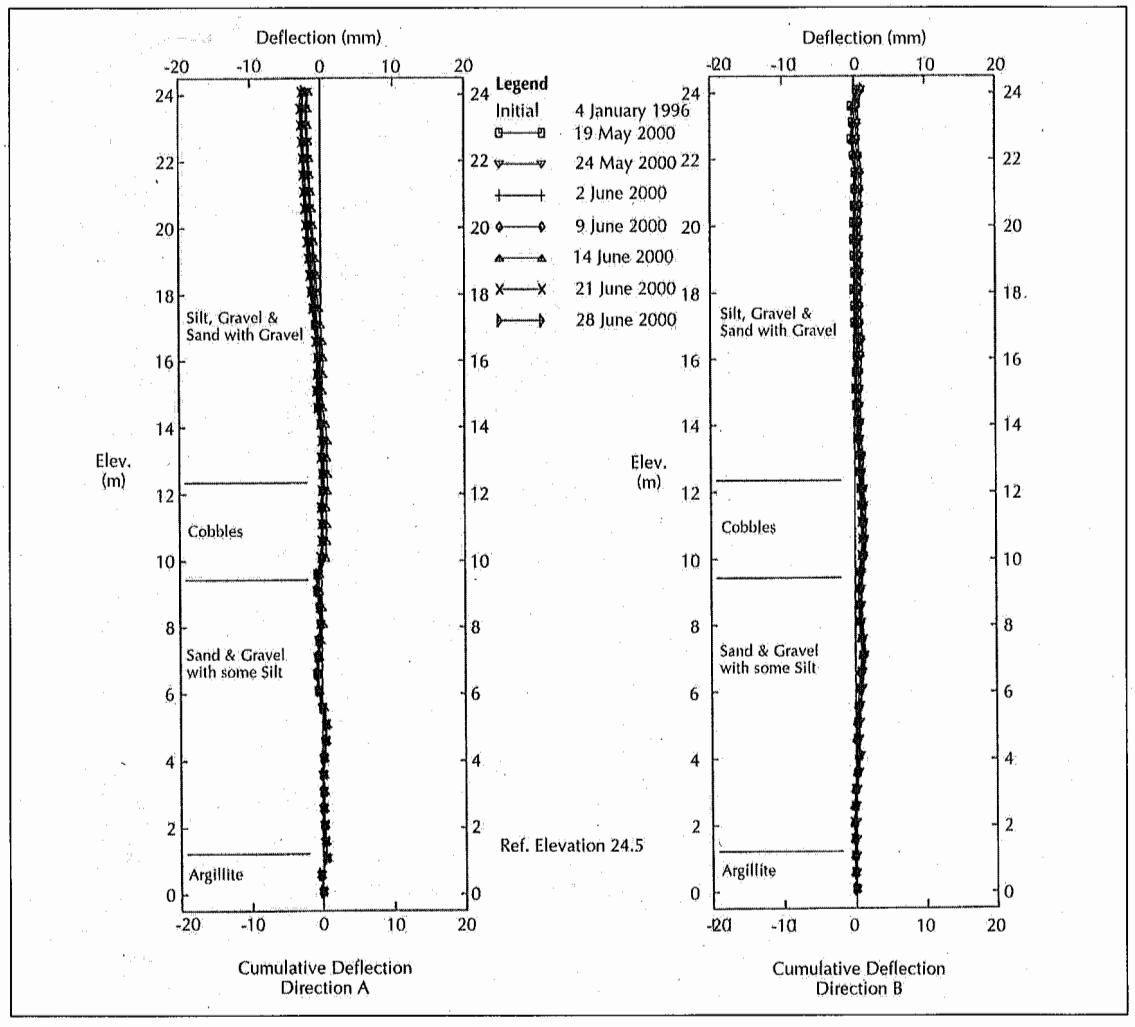


**FIGURE 7. FRB inclinometer data (from inclinometer 85705).**

lowering. Measured horizontal movements have been small — much less than predicted — due to the reduced hydrostatic pressures on the very stiff CA/T Project bracing system. For example, the FRB inclinometer data (see Figure 7) show very small horizontal movements. These movements are as small as the repeatability of the inclinometer measuring system. Although not depicted, the CGAP data also show very small horizontal movements. Seasonal variations can be a large portion of the instrument response values, as illustrated by the FRB tiltmeter data presented in Figure 8. These variations must be accounted for when interpreting all data for the effects of the CA/T Project excavation. A more detailed discussion of the instrumentation data, including comparisons with the predicted movements, is beyond the scope of this article.

The abutter/project communication process during the design period led to the valuable development of analysis methods, monitoring methods and contract provisions. This process has been extremely important during construction as instrument data are gathered, contractor issues develop and construction procedures are modified based on conditions in the field. For example, abutter/project communication facilitated the project response to the Dewey Square groundwater lowering. This response included adding instruments to the monitoring program and developing follow-up analyses that have shown that the combined effects

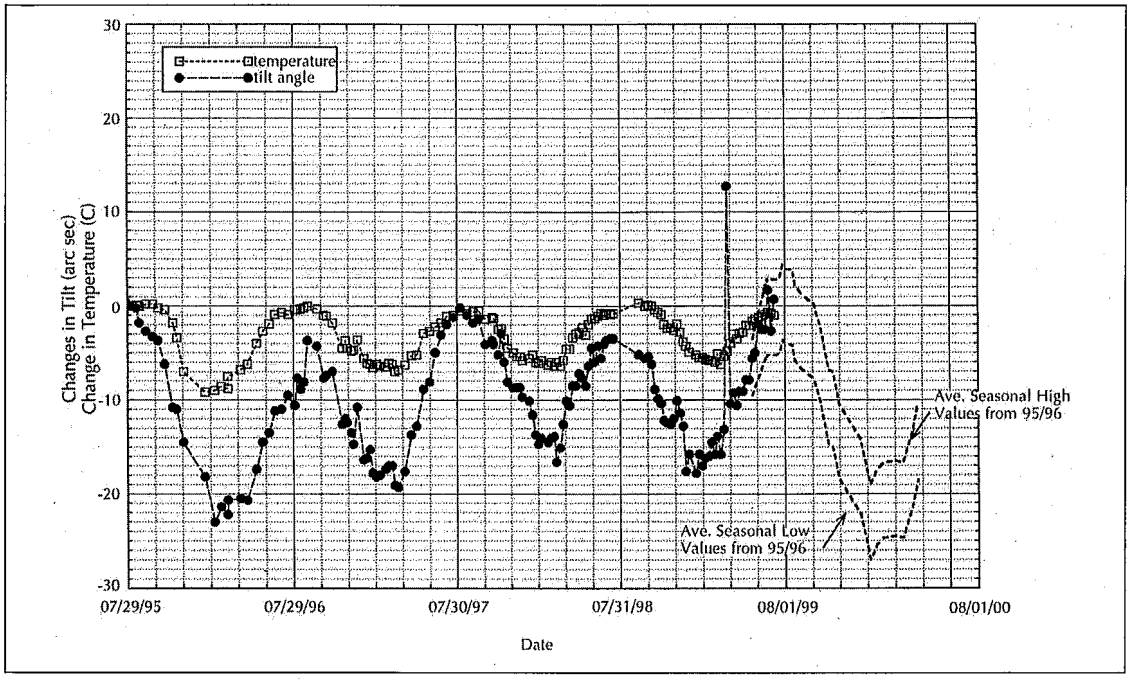


**FIGURE 8. FRB tiltmeter data (from tiltmeter 51859, y-axis).**

of ground movements caused by the excavation and groundwater lowering would likely cause comparable or less impact on the FRB and OFC buildings than what was anticipated for the assumed design condition of CA/T Project tunnel excavation without dewatering.

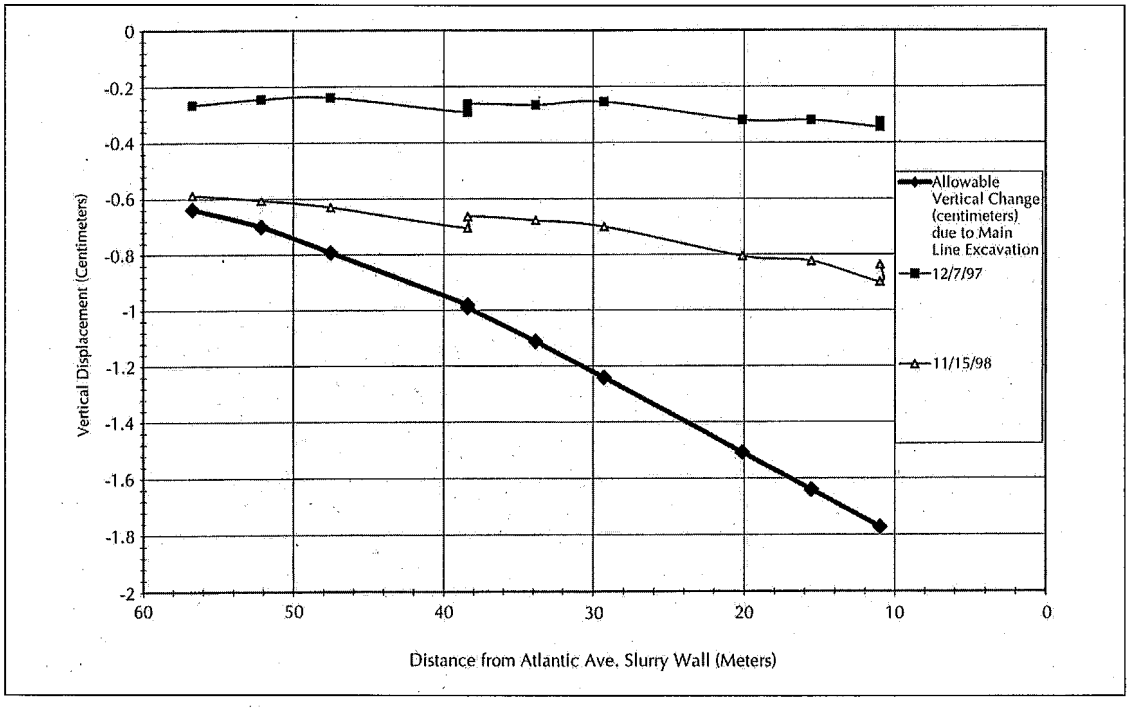


**FIGURE 9. OFC DMP settlement profile (column line E).**

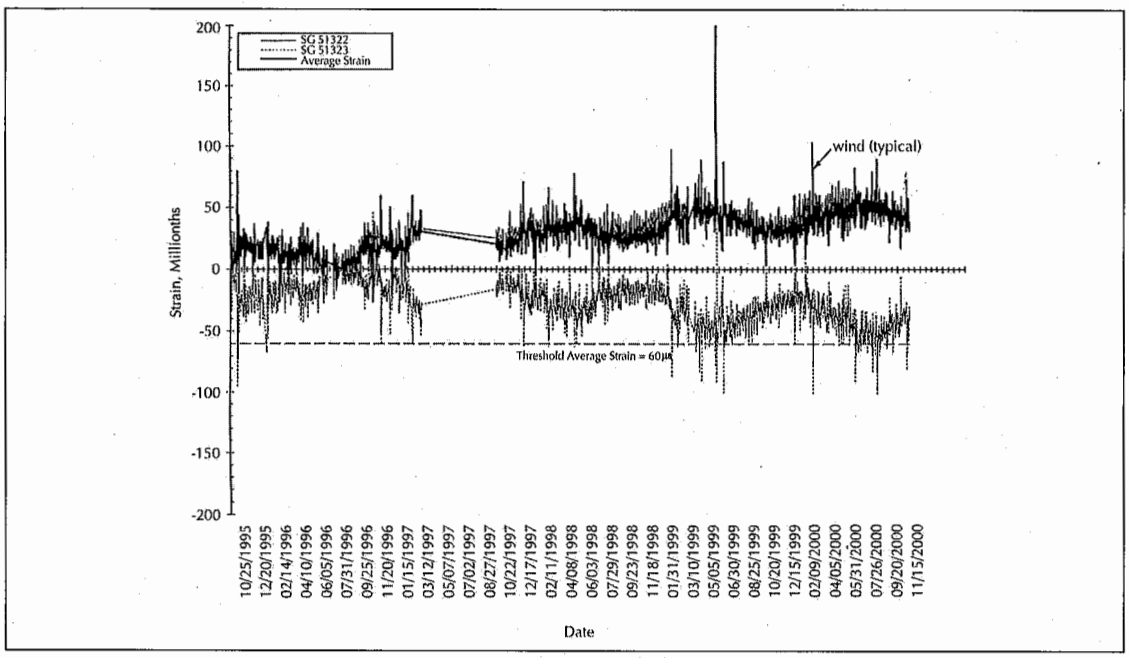


**FIGURE 10. OFC strain gage data (plate girder along the E-line).**

This benefit is due to the reduced hydrostatic pressures on the excavation support system.

## Summary & Conclusions

Measures were developed to minimize the effects of an excavation for the CA/T Project on the FRB and OFC. Protective measures included:

- abutter review of excavation support design, contract documents, contingency plans and contractor submittals;
- extensive ground movement and structural response analyses;
- geotechnical and structural instrumentation systems (hardware, data processing, management, etc.); and,
- structural modifications.

These measures were developed by a cooperative effort between the abutter consultants and the CA/T Project management consultant. In a project of this complexity, size and duration, clear communication between the interested parties is essential. Regular working group meetings have been particularly important for establishing procedures for transmitting information, for abutter review of design and construction, and for maintaining continuity of effort. The process can benefit other large and complex projects with significant potential abutter impact. It requires that the abutter and project have appropriate technical expertise, and that they cooperate and communicate clearly.

NOTES — *The Bechtel/Parsons-Brinckerhof (B/PB) joint venture serves as management consultant for the Massachusetts Turnpike Authority (MTA) in the CA/T Project work. Geotechnical Engineers, Inc. (GEI), served as the area geotechnical consultant and a joint venture of Seelye Stevenson/Deleuw Cather, in association with Haley & Aldrich, Inc., (SS/DC/H&A) served as designers of the CA/T Project work adjacent to the FRB and OFC buildings. This description of the CA/T Project is based on the final geotechnical engineering report, the final design summary report and the contract drawings and specifications for contract section C11A1. Consultant teams for the RFB and OFC consisted of Lewis Edgers (geotechnical), Thomas L. Weinmann of CTL, Inc. (instrumentation), and Kenneth B. Wiesner of LeMessurier Consultants (structural) for the FRB; and Lewis Edgers (geotechnical), Thomas L. Weinmann of CTL, Inc. (instrumentation), and Richard Henige of LeMessurier Consultants (structural) for OFC.*

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# Making the Most of Transportation Infrastructure: MBTA's South Station Intermodal Transportation Center

*Getting many different agencies to work together was instrumental in revitalizing an old railway terminal and making in into a model of integration for various modes of transport.*

LAURENCE W. SHUMWAY

Transportation infrastructure is a vital contributor to a city's economic well-being and vitality. What began in Boston with an underutilized railroad station has been transformed into a unique transportation complex. The opening of the South Station Transportation Center/Bus Terminal Complex in October 1995 marked what is likely the most significant point in the history of South Station since the original groundbreaking over one hundred years ago. While the opening signified more than the revival of a grand railroad passenger station, it also signified the completion of a transformation of this historic relic of transportation infrastructure based on a new approach to public transit — the concept of *intermodal transportation*. Thanks to the commitment of a multitude of federal, state and local agencies, Boston's South Station Transportation Center Project brought together diverse transportation modes to more effectively and efficiently serve the city's thriving metropolitan area well into the next century. The South Station Transportation Center Project combined intercity and commuter railroad operations, intercity and regional bus operations, rapid transit, an airport link connection to Logan Airport and automobiles in a single location that is close to major regional highways and at the very edge of Boston's central business district.

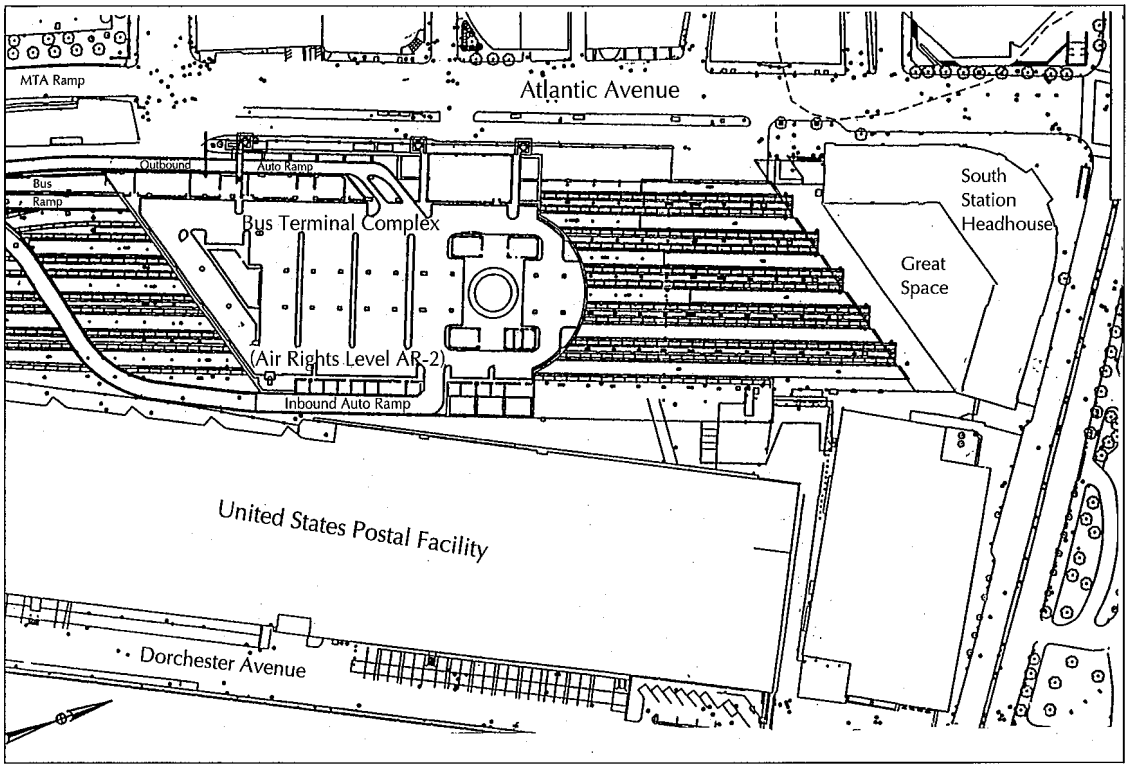


**FIGURE 1. Plan of the MBTA South Station Intermodal Transportation Center.**

Beginning with the implementation of this transformation as a part of the Northeast Corridor Improvement Project (NECIP), the South Station Transportation Center is the best example of bringing many elements of a major city's transportation infrastructure together, combining a multitude of transportation modes to more effectively serve the transportation needs of a metropolitan area. Implementation of the project involved diversified institutional and political entities, funds from several federal agencies and physical plans responsive not only to specific transportation needs but also providing a framework for future major commercial air-rights development at a new gateway to Boston.

The Massachusetts Bay Transportation Authority (MBTA) led this multi-agency project that included the Federal Railroad Administration (FRA), the Federal Transit Administration (FTA), the Federal Highway Administration (FHWA), the Massachusetts Executive Office of Transportation and Construction (EOTC), the Massachusetts Highway Department (MHD) and the Boston Redevelopment Authority (BRA).

## Background

Until reconstruction of South Station began in June 1984, its history was similar in many ways to numerous other major railroad terminals across the country. When it opened in 1899, South Station — with its 28 tracks — occupied the entire block between Atlantic and Dorchester avenues in the southeast corner of downtown Boston (see Figure 1). The station, with its neo-classical facade, was one of the largest in the country and it set a standard for numerous stations that followed. In 1918, South Station was the busiest train station in the country, handling 38 million passengers. However, railroad passenger demand began to fall sharply at South Station in the late 1940s as part of a general decline nationwide in rail ridership. The station fell into disrepair, and major portions of the facility were demolished in the 1960s and 1970s to make way for "urban renewal."

However, the future existence of South Station was assured when the small portion of the headhouse with its curving granite facade was placed in the National Register of Historic Places in 1975. The FRA (as part of NECIP), the MBTA (which acquired the property and was designated lead agency) and the BRA (which retained development rights above the transportation center) began planning the South Station Transportation Center in the late 1970s. Conceptual and preliminary designs were completed by the FRA's consultant team in 1979. In 1980, after negotiations and the execution of two multi-agency agreements and the transfer of the station property from the BRA to the MBTA, the FRA design team began design development for the rail facilities. In addition, two separate consultant teams for the MBTA began design development of the subway station modernization and the bus terminal facilities, and a consultant team to the BRA began feasibility and design studies for air-rights commercial development.

## Transportation Elements

*Railroad.* A realigned track plan and reconstructed concourse were the key elements in the design of the rehabilitated railroad facilities. The tracks were shifted to the west to align with Atlantic Avenue and lengthened so that there would be a direct relationship to the station concourse. Pairs of tracks were served by high-level platforms, providing full accessibility. The two edges of each platform were treated with tactile warning strips. The new concourse was on-axis with the headhouse main entrance from Dewey Square. Its geometry was developed from the symmetry of the headhouse so that the original station's awkward joining of headhouse and concourse was corrected. The glass wall between the concourse and the trackheads was angled, creating an exciting three-quarter view of trains in the train room.

South Station's historic landmark headhouse formed the anchor of the new South Station Transportation Center. The headhouse was designed to be the focal point of patron access to this multimodal facility. The MBTA's goal was to achieve the transformation of the headhouse into a high-quality, well maintained, mixed-use facility. The six-level headhouse was rehabilitated and expanded by the construction of a new west wing. The basement, first mezzanine and second floors were designated for transportation and public uses (approximately 68,800 square feet) and ancillary retail/concession services (approximately 40,430 square feet). The third through fifth floors were developed as Class A office space (approximately 73,360 square feet).

The existing elevator core had been reconstructed to make the upper level commercial space more attractive and rentable as office space. To complete the original symmetry of the headhouse facade, the new west wing containing retail and commercial office space was built and the east wing was extended to accommodate a new fire stair. Both of these wings were constructed using details composed primarily of precast units that mimic the historic facades in the neoclassical style of the original architecture.

*Rapid Transit.* Another important element serving the rail concourse level was a pair of escalators and stairs to the MBTA's Red Line subway station, providing direct access to and from the rapid transit system. Under separate contracts, the Red Line station was modernized and platforms were lengthened to accommodate six-car trains to significantly increase the capacity of the station.

*Bus.* Phase II of the South Station project involved the construction of a new consolidated bus terminal and parking on air rights above the railroad tracks and platforms. The work consisted of the construction of two structural decks (including railroad ventilation, bus terminal operations facilities and parking), construction of roadway ramps from the local street network for access to the bus terminal and parking (with provisions for direct connections to the highway network), construction of a service facility to serve the headhouse and railroad operations and construction of the Atlantic Avenue frontage. This new frontage construction extended from the end of the new west wing of the headhouse to Kneeland Street and included a bus terminal lobby entrance.

Prior to the completion of the project, bus operations in Boston were dispersed through-

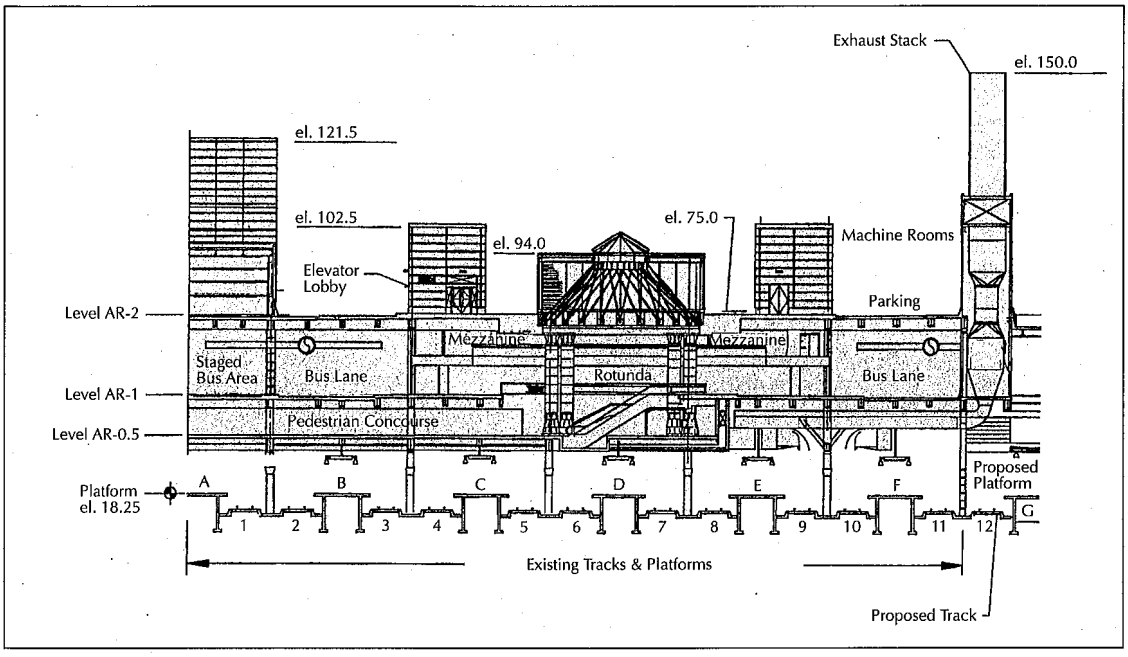


**FIGURE 2. Section through the building concourse.**

out the city. Privately owned bus companies provided commuter and intercity service to the suburbs of Boston and the towns and cities in the Northeast, with connections to the remainder of the United States and Canada. The old Greyhound and Peter Pan/Trailways terminals and a South Station commuter bus facility functioned as the Boston termini for most of the private carriers. A small number of private commuter bus companies operated from curbside locations scattered around downtown Boston.

The design of the bus terminal facility achieved the goals of:

- Creating an attractive and inviting image;
- Affording clarity and ease of movement for patrons;
- Providing a central location for many different bus operations servicing the city; and,
- Delivering adequate and discrete space for those bus operations.

An attractive and inviting image for the bus terminal was created at street level by a modest yet visible entry lobby. Special consideration was given to security by providing clear open spaces in and around the lobby and prominently positioning security operations for high visibility. A drop-off curb length was designed at the street in the vicinity of the lobby. A domed skylight covered the terminal's two-story rotunda, serving as the focal point for ticketing, baggage-handling and services for bus patrons. Throughout the terminal, sophisticated, durable interior finishes were utilized to provide a more easily maintained appearance. Clarity and ease of movement for bus patrons was achieved by zoning air rights Level AR-0.5 for pedestrian use (see Figure 2).

Adequate space for bus operations was provided by zoning air-rights Level AR-1 exclusively for bus circulation. Within the site constraints, bus docking spaces were maximized. Project construction Phase IIA provided 23 saw-tooth docking positions, four pull-through positions and two positions for the bus connection to Logan Airport (the Airport Link) to accommodate the ten private companies utilizing the new facility. Current estimates have approximately 12,000 patrons using the facility daily.

*Parking.* The parking level at air-rights Level AR-2, including parking spaces for 225

vehicles and a package express facility, was served by ramps to and from Kneeland Street. Additional ramp connections directly to the adjacent highway system, as well as the new Ted Williams Tunnel to Logan Airport, have been anticipated and will be constructed as part of the MHD's Central Artery/Tunnel (CA/T) Project. The parking level not only provides much-needed parking to serve the transportation center, but also serves as the "roof" covering the bus operations and it has been designed to support construction activities anticipated for the future development of air rights.

## Project Cooperation

At the outset of the conceptual design process, the South Station project was perceived as a multiphased project. At each phase, the preferred design concept was defined by a set of broad, interrelated criteria. Later, and throughout the project's development, individual attention was focused on particular elements of the complex, including the headhouse with its new west wing, the bus terminal, access ramps and the local street network, and future air-rights development. With the design development of each of these elements, key design issues — including pedestrian circulation, architectural treatment, railroad ventilation, special systems, utility considerations and proposed utilities — were addressed.

By agreement of the parties, the FRA (under the NECIP Project) was responsible for the bidding and construction of the railroad elements, and the MBTA was responsible for the bidding and construction of the Red Line modernization and construction of the bus terminal complex. Although numerous technical and funding issues were encountered, the parties of interest had the common objective of moving forward with completion of construction of the entire transportation center since it was designed to meet the diversified needs of a wide range of users.

## Project Coordination

*Interagency Coordination.* Coordination of the overlapping — and occasionally competing — requirements of the FRA, the MBTA and the BRA was the key to the ultimate successful completion of the transportation center. Beginning in the earliest stages of the project's development, each agency and its consultant team endeavored to advance coordinated designs for each respective agency's program. This effort, which required good-faith negotiations and compromise by the parties on a number of design and technical issues, led to the eventual designs meeting the needs of all of the parties.

*Establishment of Overall Project Goals & Objectives.* In order to realize the fullest potential of the combination of transportation elements at a single facility, the following design objectives were established jointly by the participating agencies:

- To provide facilities that meet the operational and support needs of the intercity, commuter and local transportation operators and users with the maximum efficiency.
- To provide an environment that meets contemporary standards for passenger safety, comfort and enjoyment.
- To provide an overall setting, massing and image that enhances the historic headhouse.
- To provide convenient interchange between the various transportation modes and metropolitan Boston.
- To provide transportation facilities that complement and encourage both state and city plans for commercial development at South Station and the surrounding area.
- To provide barrier-free access for the physically challenged.
- To provide a distinct identity for each transportation mode and operator.
- To provide maximum frontage along Atlantic Avenue for the competing automobile pick-up/drop-off needs of the rail and bus carriers, and potential air-rights development.
- To provide clear, efficient access between the local street and highway systems as well as between the bus and automobile parking levels.
- To provide a network of ramps for automobiles, buses and package express

trucks that allows for direct access from the air-rights levels to the highway system and that can be implemented in multiple phases.

## The Design Process

*Pedestrian Circulation.* The preferred design concept recognized the entry from the MBTA subway as the most heavily used intermodal connection. The new passenger tunnel provided access from the Red Line station directly into the lowest level of the headhouse. Within the existing headhouse, escalators from the Red Line station delivered patrons on-axis to the edge of the "Great Space" concourse. The connection was clearly organized and sufficiently sized to accommodate pedestrian traffic with minimal disruption.

The pedestrian path from both the Red Line transit station and the street continued directly into the Great Space. The advantage of bus patrons participating in the life of the Great Space was obvious. However, there was concern about the capacity of the Great Space to accommodate peak period activities, as well as the effects of diverting pedestrian activity away from Atlantic Avenue. Early in the design development process, it became apparent that the concept of the transportation center would make very significant changes to the fundamental character and function of the Great Space concourse (the area of the transportation center complex that is the natural as well as physical focal point). The area changed essentially from a railroad station waiting area to the primary activity center of this major intermodal transportation center. The key project elements and issues that were considered during the development of the transportation center concept included:

- Train gates and railroad passenger access;
- Pedestrian circulation issues;
- Pedestrian movement safety issues;
- Inherent clarity of movement;
- Display of directional information;
- Pedestrian security;
- Movement of special needs persons;
- Activation of public spaces;
- Bench seating; and,
- Retail kiosks.

*Clearance Requirements.* The basic structural grid supporting air-rights construction was dictated by the railroad track/platform configuration established by the FRA in concert with the MBTA. The structural support for the first air-rights level had to be designed without full-height bracing or shear wall elements in order to maintain the required clearances for railroad operations. Furthermore, for adequate train room ventilation a maximum clearance had to be maintained between the top of trains and the first air-rights level.

The structural system that was developed for the required lateral stability of the structure is a two-way, fully welded, moment-resisting frame. The use of a composite structural deck system utilizing a longitudinally post-tensioned cast-in-place deck on precast, prestressed planks responded to the concerns for differential deflection, while also addressing constructability issues (including time of erection, shielding of terminal operations and performance as an intermediate working platform).

*Railroad Operations.* Because the bus terminal complex was to be constructed on air rights above the diesel locomotive operations, early studies were undertaken to develop a design strategy that would address the problem of train ventilation in the simplest and most cost-effective manner without major compromises to either bus or train operations. Obviously, ventilation requirements were directly related to the operating modes of MBTA commuter rail and Amtrak service, schedules and equipment. In response, the design solutions developed to address the concerns for both initial and operating costs for ventilation combined adjustments to the railroad operations along with modifications to the size and shape of the building complex.

Current rail traffic into South Station consists of a combination of MBTA and Amtrak revenue trains as well as a number of non-revenue train movements for maintenance and storage purposes. The MBTA, recognizing the problems associated with collecting diesel emissions along the entire length of eleven or more tracks, determined that normal MBTA operation might be best arranged with locomotives on the outbound, or southerly end, of each

train. This approach was made with the realization that this situation could change if the MBTA's South Side locomotive fleet were equipped with electric traction power capability. The other basic approaches were that the MBTA air-rights development would only be constructed over the length that a nine-car train occupied (leaving the locomotive uncovered), and that trains shorter than nine cars would be stopped so that locomotives were not under the air-rights development. An additional stipulation in the development of railroad ventilation alternative strategies was that Amtrak would operate only on station tracks 8, 9 and 10. This decision was used to further refine ventilation alternatives.

*Design Innovations.* The South Station project produced many engineering achievements as the result of the project's design requirements:

- Specifications for curved trapezoidal box girders now can be more effectively interpreted better for radii significantly less than 300 feet;
- The structural framing system provides engineers with a constructed example of how air-rights construction can be effectively accomplished within constrained dimensions over railroads and expressway facilities; and,
- Air-rights development over diesel-locomotive operations utilizing a practical ventilation system provides new opportunities for intermodal transportation, the expansion of rail transportation and public/private air-rights development.

Accommodations for vehicular access to the project air-rights bus terminal complex was severely constrained by the site and its surroundings. Design constraints included the inability to control the location and configuration of the ramp's substructure support system due to the proximity of MHD's CA/T Project, vertical clearance requirements and geometry mandating a minimum radius of 120 feet. Steel trapezoidal box girders were utilized as the primary structural element for the viaduct roadways. AASHTO specifications and normal practice had previously only addressed steel trapezoidal box girders with radii of 300 feet or greater. The engineers' innovation and judgment reconciled the intent of AASHTO with MHD's CA/T Project design criteria and the specific requirements of the South Station project.

A unique framed-in capbeam and merging box girder system was designed in response to the restricted substructure envelope allocated to the ramp structure. It consisted of two vertical webs having separate top and bottom flange connection plates with a bolted full-width flange plate to create a box shape. The steel fabricators built scale models to assess their ability to fabricate and assemble this "one-of-a-kind" structure. The transportation of the box girders required careful consideration because the severe curvature limited the length of each box girder segment that could be practically transported. The design, fabrication and construction of these unique steel trapezoidal box girders and the framed-in capbeam had never been done on a site with such significant constraints. Their design extended the limits of existing technology.

The ability of the structure to achieve the required lateral stability was severely limited by the existing railroad tracks and structural grid. The structural support for the first air-rights level had to be designed without full-height bracing or shear wall elements. Furthermore, for proper train room ventilation, a maximum height between the ground level and the first air-rights level had to be maintained. The new structural system was original and innovative. The lateral resisting system was developed as a two-way, fully welded, moment-resisting frame. The use of a composite structural deck system, utilizing a longitudinally post- tensioned, cast-in-place deck on precast prestressed deck planks responded to concerns of differential deflection, while addressing constructability issues (including time of erection, shielding of terminal operations and performance as an immediate working platform). The concrete for the precast deck planks was designed to resist the corrosive diesel exhaust from below and chlorides from above.

Bus traffic, truck traffic and accommodations for future air-rights construction loadings were unusual factors that had to be taken into

account in designing the structural system for the building. The South Station building structure was truly unique because AASHTO specifications had to be interpreted for application on the air-rights levels in conjunction with applicable AISC specifications. This combination expanded the application of these existing technologies.

While train stations with air rights and low ceilings have been built in Canada and the United States, none of these facilities have been completely successful in venting diesel locomotive exhaust. Engineers had to develop a method of capturing a 70-foot-per-second diesel engine exhaust jet within a relatively low ventilation slot velocity while minimizing pressure losses and fan power requirements and fitting the total ventilation system within an 11.67-foot-high space directly over the railroad clearance envelope. The concept of the new ventilation duct system utilized the kinetic energy of the engine exhaust jet in combination with the suction of the ventilation fans to capture essentially 100 percent of the diesel exhaust fumes. The new ventilation duct system included a 34-inch wide slotted duct above and along the tracks, and a curved and contracting passage leading through a nozzle and orifice into a manifold, and exhausted through fan ducts.

Using the train room's basic dimensions, configuration and established pollutant concentration criteria, a mathematical model was developed utilizing finite element techniques. The computer model evaluated various design and railroad operation alternatives ranging from standard solutions to innovative "uniform slot ventilation systems." These systems exhibited the greatest promise and exhibited additional benefits as the hood was curved and made deeper. A physical model of the proposed ventilation system was constructed to validate the mathematical modeling findings.

## Conclusions

The South Station Transportation Center Project utilized the initiative of NECIP as well as initiatives of the FTA to secure sizable additional federal, state and local funds to bring to fruition a unique multimodal transportation center. Successful completion and inauguration of this facility has made a substantial contribution to solving some of the critical issues that face any large urban area today:

- Preservation of historic resources and the urban fabric;
- Reduction of energy consumption and air pollution;
- Improvement of basic transportation systems; and,
- Continued revitalization of the urban core.

Because the South Station project makes available simple direct connections between the various modes of public transportation, it will foster greater, more efficient use of these systems by the traveling public.

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# 20/20 Vision: The Engineering & Construction Industry in the 21st Century

*While venturing to predict the future is a risky proposition, there are certain issues and trends developing now that will definitely have an impact on how the engineering and construction industry will do business in the next 20 years.*

HENRY L. MICHEL

Now that we are ending the second millennium by dragging the engineering and construction industry kicking and screaming into the twenty-first century, it is time to look at ourselves and decide what we want to be when we really grow up. The biggest change that we will confront is that we will have to provide "products" rather than projects and the product will have to be structured to meet performance requirements from the cradle to the grave. True life-cycle performance will have to encompass all "product" aspects from sustainable development to ultimate re-use or disposal — be it a building, a highway or a generator set. To do so will require a total delivery system that cuts across all disciplines within our industry and that also directly involves all other industries that contribute to the final product. This delivery system will include not only the traditional team members — such as designers, builders, lawyers and insurers — but also the manufacturers of the construction equipment as well as the entire range of construction materials manufacturers. We will also see the entry of a whole new range of non-conventional advanced materials that are self-assembling, self-growing and self-healing. These materials will be designed by teams working in cyberspace, built and assembled in highly automated environments, and linked together electronically from inception to ultimate disposal.

What changes will help us get there? Throughout history there has been no shortage of prognosticators. Here is a sample of what some of them have had to say about the "future":

- In 1899, Charles Duell, then head of the U.S. Patent Office, said, "Everything that can be invented, has been invented." He also recommended that the office be abolished.
- In 1943, Tom Watson, Chairman of IBM, said, "I think there is a world market for about five computers."
- In 1977, one year after Steve Jobs founded Apple Computer, Ken Olsen, the President of Digital Equipment, said, "There is no reason for any individual to have a computer in their home."
- And, in 1981, Bill Gates, then CEO of Microsoft, said, "640k [of RAM] ought to be enough for anybody."

So much for the science and art of forecasting future needs.

Everyone aspires to 20/20 vision, even if one may need a bit of correction to achieve it. Let us see what our 20/20 vision for the engineering and construction industry in the year 2020 might look like. What changes are occurring, and how will they impact our activities?

In order to identify the possible changes that we might see confronting our industry between now and the year 2020, we need to look at the industry from three perspectives:

- The significant changes that have impacted our industry over the last 20 years.
- The implications of those changes on our future, and the challenges they engender.
- The possible changes that might occur during the coming 20 years and what we should try to do to bring them about.

But whatever we do, we better get ready for a major restructuring of our industry — one that will be revolutionary rather than evolutionary.

## Significant Changes

Let us list some significant changes that took place during the past 20 years that have influenced the engineering and construction industry:

- The continuing depletion of the world's natural resources and resultant awareness of the need to consider sustainable development.
- The transformation of the design profession from a labor-intensive to a capital-intensive sector of the industry through the application of computer and space technologies — in particular, computer-aided drafting and design (CADD), global positioning systems (GPSs) and geographic information systems (GISs).
- The introduction of new construction materials, plastics, composites, fiber optics, etc.
- The better understanding of the need for, and value of, applied research.
- The emergence of environmental and hazardous waste clean-up as a top national priority.
- The evolving changes in the packaging and delivering of projects: design-bid-build, design-build, turnkey, construction management, program management, etc.
- The growing importance of women and minorities in the workforce.
- The rise of the consumer movement and public activism and the need for community participation.
- The growth of a litigious climate.
- The shrinking of the globe due to communications and information advances.
- The emergence of a "global economy" with the introduction of global competitors.
- The renewed emphasis on trade barrier reductions — NAFTA, European Union, APEC, World Trade Organization.
- Increasing pressure on governmental budgets in the United States and worldwide causing reductions in public investments and a trend towards more privatization of public facilities.

## Future Implications

What are the implications for the future and can we meet those challenges? Here are some issues we might face:

- The entry of defense contractors and accounting firms into our industry as part of their diversification strategies.
- The introduction of performance-based procurement, requiring delivery systems that are faster and more finance-driven (such as design-build-operate-maintain, build-operate-transfer, partnering, etc.),

which continues the trend to more privatization, or "contracting-out."

- The introduction of new building materials and systems, as well as greater use of recyclable materials.
- The greater use of automation and pre-manufacturing that is more capital intensive, leading to further industry consolidation into fewer but larger and more diversified companies.
- The transformation of the construction industry through computer-enhanced construction methods as in architect Frank Gehry's "experience music project" in Seattle that integrate three-dimensional computer-aided design (first developed to design airplanes) into the construction process.
- The use of digital design-build techniques (sometimes called 4dcad), where three-dimensional computer-aided designs are joined by the fourth dimension of time.
- The ability to tap distant low-cost markets. For example, the use of third world labor for design as well as for pre-manufacturing building components.
- The need for new skills, including a broader ability to address not only technical but also economic, environmental, societal, political, legal and aesthetic concerns.
- The globalization of the industry and the need to recognize barriers such as language, cultures, standards and certification.
- The advent of a truly balanced environmental ethic resulting in multinational cooperation to deal with global ecosystems, pollution reduction, resource conservation and sustainable development.
- The need to create new scientific breakthroughs to deal with hazardous and non-hazardous waste management (to recognize that waste is an out-of-place resource).
- The need to restore mutual trust in our society and introduce serious tort reform.
- The need to bring back the credibility of engineers and constructors. (For example, we need a television show like *LA Engineer*.)

## Future Changes

What are some of the possible changes that might occur during the coming 20 years and how will we meet those challenges? The greatest single change is one that started more than 30 years ago: the growth in the global construction market versus the U.S. domestic market. Thirty years ago, the U.S. market represented about half the world's total. In 1999, it had shrunk to 18 percent. Now 18 percent of a $3.6 trillion market is still a huge workload. However, our domestic market is static and the global market is expected to grow at an annual rate of 5 percent. Therefore, five years from now the U.S. share will be down to 15 percent of the total. The implications are clear: with more than 80 percent of the market elsewhere, many of the major breakthroughs are likely to occur outside of the United States. And we had better shed our parochialism and learn from those "foreigners."

Not surprising, the world's entire population growth, estimated at 100 million per year over the next 20 years by the World Health Organization, will mostly occur outside the United States. In fact, this development will occur outside of all the developed nations of the world. If you cannot get your arms around 100 million people, think of building a new San Francisco every 3.5 days for the next 20 years — and that does not include all the many unmet needs of less developed nations.

Those changes and growing needs present an immense challenge to our industry and, in fact, to our whole society. A total of $3.6 trillion in construction per year means a lot of natural resources, a lot of money and a lot of talent.

Natural resources have to be converted into the building materials and equipment that make up a completed facility. We will need literally hundreds of millions of tons of building materials and from where will those materials come? Every year the U.S. construction industry deposits over 100 million tons of construction debris in landfills; probably another 100 million tons or more are dumped elsewhere. How long can we continue to throw away those materials? Many European countries — as well as Japan, Korea and Taiwan — have established target dates by which they will eliminate construction debris disposal by employing re-use and recycling technologies and by finding new uses for those materials. Their target dates for 100 percent material recovery range from the years 2002 to 2010.

In addition, the European Union has instituted regulations — now voluntary, but ulti-

mately mandatory — that relate to source control, where the manufacturer or producer becomes responsible for the cost, or the act, of product disposal at the end of its useful life. Initially, those regulations have been directed at the automobile industry but they will eventually apply to all manufacturing industries including construction. Here is one example of what would happen with those regulations in force:

> You buy a Mercedes Benz, drive it for a couple of years, and then give it to your kid to take to college. A year later it is totaled on the way home for winter vacation. At that point it once again becomes the property of Mercedes Benz, which is responsible for disposing of the wreck. You paid for that disposal cost because it had been included in your original purchase price, but now it is the manufacturer's responsibility to dispose of the wreck and the manufacturer will want to keep dismantling and disposal costs to a minimum. This burden will create a huge wave of innovation in materials — many non-conventional, combining chemical, biological, biomolecular and other cross-fertilized sciences to create components that can be transformed into other substances during the recycling and reformatting process. BMW has a demonstration car that can be disassembled in 23 minutes, reducing the disposal labor cost significantly.

The thrust of recycling of paper and of beverage containers with which we are familiar will be followed by that assault on the automobile and other manufacturing industries and, in the end, on all industries, even the conservative, non-conservationist, construction industry. Doing so will give real meaning to the concept of sustainable development.

## Paying the Piper

What about money? How do you pay for works that will require $3.6 trillion, or more, every year? The global construction market breaks down into major categories:

- 36 percent residential;
- 36 percent non-residential; and,
- 28 percent civil works, or physical infrastructure.

Traditionally, residential construction has been partly funded from public sources and mostly from private sources. Non-residential construction, with the exception of government offices and secondary educational facilities, is mostly funded by the private sector. Civil works have generally been provided by taxpayer funds or user funds (which is another form of taxation levied by governments).

With limitations on the availability of taxpayer dollars, a growing proportion of the world's construction bills will have to be paid by non-government sources — by the private sector. That brings on the golden rule: whomever has the gold, will make the rules. One result is to move to value-based delivery systems, with whole-life or life-cycle cost as the driver. The time value of money and the operating and maintenance costs will generally outweigh the initial design and construction costs by factors of 20 to 30. Early completion to achieve early investment recovery, coupled with ease of maintenance and controllable costs of operation will drive project considerations. The value-added delivery systems will have to incorporate all those factors. The client of the future will no longer be the chief engineer but will be the chief financial officer.

Private-sourced funding has traditionally been project oriented. But, in addition to funding discrete projects as we do today, in the future we will also see more project bundling — "mutual funds" of projects, not of companies or of industries. They will be either captive funds created by insurers, pension trust fund managers, retirement fund managers, privatized social security programs, etc., or they will be open to individual investors (in other words, open to the public). There are huge pools of money looking for reliable returns on the investment. This bundling of projects also will help to balance the risks. E-links will simplify the process and also help generate other forms of innovative financing.

## Getting It Done

Where we are going to find the talent to do it all? The western world's population is aging. Fewer people are entering our industry than

are leaving. More of our technically educated people are moving into other careers. With growing needs to produce more and better constructed facilities, we will have to do things differently. One important aspect is that we better realize that all industries — including ours — now have to perform in a truly borderless environment: with no physical boundaries, no political borders and no cross-industry barriers. No ways exist to constrain unlimited flow of information, not only within a project but also within an industry, across industry sectors and literally beyond this planet to extraterrestrial bodies. The detailed mapping of the planet earth by a NASA space shuttle last February is a good, but probably primitive, example of things to come. A better example of things is the development of the "E-Era," the electronic communications that have made it possible to create an automated 24/7 design office, located wherever talent is available and affordable. For example, Indian and Philippino engineers, architects and planners will design international projects in their home countries that are checked, monitored and managed by supervisors in Boston. The Worldwide Web, Internet, intranets and other electronic communications and control systems will radically change the pre-construction as well as the construction and operation and maintenance phases as we know them today.

For the construction part, the solutions will be different. A shrinking work force in certain countries cannot be offset by farming out everything to another country. But replacing some of your traditional workforce with pre-manufacturing, with automation, robotics, electronic and e-commerce, etc., cannot only reduce your dependence on that shrinking labor market, it can also increase your productivity and competitiveness. Pre-manufacturing reduces on-site labor and can also tap low-cost labor in other parts of the world. Automation, such as satellite-controlled construction equipment, eliminates the need for equipment operators. Robotics can be used to build entire buildings with a minimum of manpower (a modular 12-story reinforced concrete structure was built recently in Japan using only robots and computer controlled construction equipment). New technologies permit a paperless project all the way from conceptual design to a good set of as-built drawings, with the concomitant reduction of office and field staff. E-commerce will simplify and expedite the whole procurement cycle by providing access to the entire world for the best subcontractors, to the most reliable and least costly materials delivery, to readily available construction machinery, etc. In other words, e-commerce enables you to leverage your resources and will help you deliver projects with less manpower and at a lower price.

So let us get ready for an electronically straddled borderless marketplace where only the fittest will survive. And the fittest are those who are willing and able to embrace change.

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HENRY L. MICHEL *earned an international reputation as a leader and innovator over the course of his 50-year career as a civil engineer. A partner at Parsons Brinckerhoff (PB) since 1969, Michel served as PB's first President and Chief Executive Officer from 1979 to 1990 and as Chairman from 1990 to 1994. During his tenure, PB expanded from a 500-person firm with six overseas offices to a nearly 4,000-person firm with more than 100 offices in 30 countries around the world. He graduated from Columbia University in 1949, and was a Senior Lecturer at the Massachusetts Institute of Technology, Industry Professor of Construction Management at Polytechnic University of Brooklyn in New York and a guest lecturer at many other universities. He was a founding member of the Civil Engineering Research Foundation (CERF), serving as its Chair from 1989 to 1996. He was the first recipient of the CERF Henry L. Michel Annual Award for Industry Advancement of Research. On may 23, 2001, Henry Michel died at home in Manhattan at the age of 76.*

# The Discovery of Pluto

*Engineers can count their blessings that they could retire their slide rules long ago; however, it might be worthwhile to take a look backward and see where the real blessing lies.*

BRIAN BRENNER

I had the pleasure of visiting the Lowell Observatory in Flagstaff, Arizona. The observatory is perched atop a hill in Flagstaff, a small city at an elevation of about 7,000 feet above sea level. When the observatory was first built, Flagstaff was much smaller, with fewer street lights and less nighttime illumination. Today, the city has grown, and the glare is too bright for any meaningful astronomical observation. The work has been relocated to a distant, darker butte out of town in the desert. But the original observatory with its old telescopes and facilities remains for tourists to visit. It is a beautiful site, with a pleasant campus surrounded by a high-altitude conifer forest. This is the site where astronomers discovered the last planet in our solar system — Pluto. The methods they used in the 1930s seem primitive today. A comparison between the past and present helps illustrate the great advances in technology that we have experienced and now take for granted.

The discovery of Pluto took place in two phases. It all began with the suspicion by astronomers that there must be a ninth planet because the solar orbits of the first eight planets were not quite right. The application of Kepler's Law predicted certain types of orbits, and the measured orbits of the known eight planets did not match the predictions. It was expected that another planet was exerting gravity that influenced the orbits of the inner eight planets, leading to the search for "Planet X," the ninth planet.

In the first phase of exploration, mathematicians performed thousands of calculations to try to determine the mass and location of the missing planet. The volumes of the original calculations are on display at the observatory. These relatively simple trigonometric and algebraic calculations can be performed today using a spreadsheet or MathCad in perhaps a few minutes. In the 1930s, it did not take a few minutes; rather, it took months of painstaking, manual arithmetic, with careful pencil markings, slide rules and volume after volume of calculations and checking. There were no computers or expectations that the calculations could be done any other way than longhand.

This phase in the discovery process seemed painful enough to me. At some point, it was determined that the calculations were complete, and it was time for the next phase to occur. Based on the predicted locations, astronomers set out film plates exposed to telescopic images of the night sky to find the new planet. The exposures of each plate were separated by two or three days. The idea was that because a planet would move differently than a star, its image could be identified by comparing images from

different nights and looking for a point of light that did not match up as a star would. The process of comparing the images was done in something called a blink comparator. The observatory had set up a display of the original device so we could observe the method. The way it worked was that you would first see a quick blink of the film plate on the left, and then a quick blink of the film plate on the right. Staring into the device, the picture would flash back and forth — blink of light after blink of light. You had to find the one point of light out of hundreds that did not match on both film plates. Gratefully, the display we looked at had convenient arrows showing the point of light that was determined to be Planet X — Pluto. With the arrow placed on the map of stars, it was easy to see that one tiny point of light that did not quite line up the way the other hundreds of points did and, therefore, must be a planet. The convenient arrows were not, however, available for the poor gentleman (instead of "gentleman," I was thinking of a word here that starts with "b") that had to sit for one year with the blink comparator, flashing hundreds of film plates, back and forth, back and forth, trying to find a needle in the starstack. Today, we could digitize the images and have a computer compare them and determine any differences in a matter of moments. A year of frustrating, boring, tedious work today would be completed in seconds.

Seeing the exhibit, I thought about how different our lives are today, and how our expectations have changed. It is not just in discovering Pluto, but in every aspect of technology and how we apply it. I was preparing to teach a freshman introduction to civil engineering class. I planned to present a lecture on the strength of materials. A colleague loaned me an old textbook written in 1951. He found this book at a rummage sale and purchased it for a quarter. It was musty but readable, with good examples and still-relevant themes. The book presented a series of simplified derivations. For example, there was a problem on the axial loading of two bars with different cross-sectional areas and different moduli of elasticity. The layout and solution of each problem was predicated on the assumption that you could not use a computer. In 1951, while computers had been invented, they were top-secret, punch-carding, room-filling machines that had less computational power than the personal computer I now use for work. It was not for another decade or so before civil engineers could start to imagine practical computer applications like STRUDL and COGO. So, the text was carefully developed with this in mind. There was even a helpful chapter in the back to assist you with your slide rule.

All sorts of things were done this way without computers, whether it was discovering Pluto or designing monumental suspension bridges. Today, how this past work was done seems like monumental drudgery. We have been liberated by the incredible computation power at our fingertips. This new tool has become second nature, and now the old work-arounds and methods fade into history, to end up as exhibits about finding Pluto, and as collections of dusty old slide rules. Considering what people spent their lives doing then, the way we are able to work today is so amazing. Yet in the work-arounds and the necessity of the old methods, there was a certain grace and embedded level of technical excellence. The manual work was so hard and tedious that there was little margin of error for misapplication. In comparison, there are times when I wonder about how we use and appreciate the technology with which we have been blessed today.

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