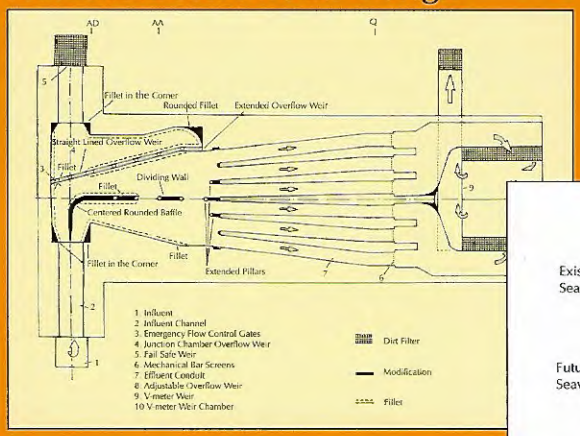
CIVIL ENGINEERING PRACTICE

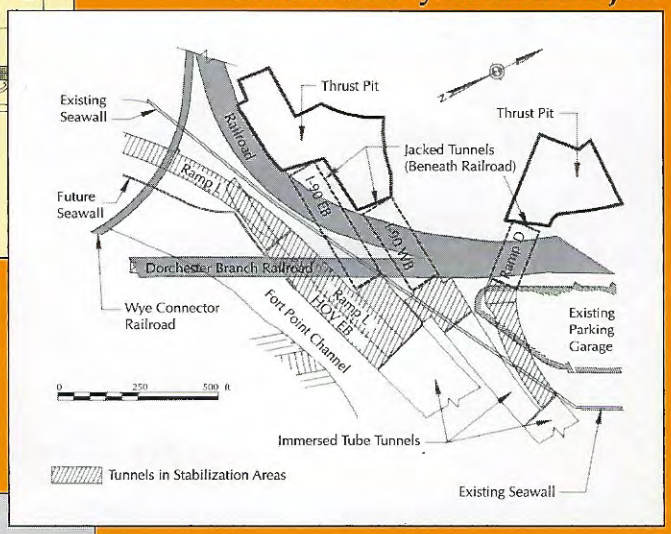
JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

FALL/WINTER 1999 VOLUME 14, NUMBER 2 ISSN: 0886–9685

Modeling Use on Nut Island Headworks Design

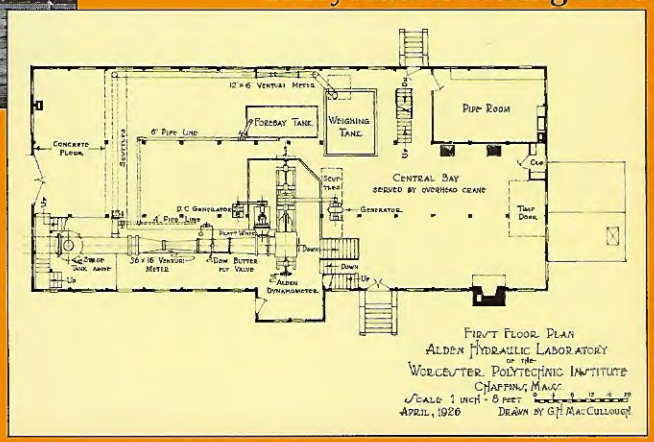


Design for Tunnel & Excavation Support on the Central Artery/Tunnel Project



The Historic New Bedford-Fairhaven Bridge

The Role & Contributions of Hydraulic Testing Labs



Special Section:

- **What Has the Finite Element Method Done for (or to) Geotechnical Engineering?**

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Erratum

The last sentence for Susan E. Murcott's author biography in the Spring/Summer 1999 issue (page 62) should read: *"She has been a research scientist in 1993 and 1995 at the International Institute for Applied Systems Analysis in Laxenburg, Austria; a fellow in 1994 and 1999 at the Salzburg Semina in Salzburg, Austria; and has undertaken research and field projects in Eastern Europe, Mexico, Brazil, China and South Asia."*

The Role & Contributions of Hydraulic Testing Labs: Part II, World War I to World War II

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

At the beginning of the twentieth century, the theory relating to hydropower was advancing. R.D. Johnson published his analytical treatment on surge tanks in 1908.[1] Many colleges started including courses on hydropower. At Worcester Polytechnic Institute (WPI), student assignments relating to power at the Alden Hydraulic Laboratory (AHL) at Chaffinville were extensive. For example, a list of experiments for mechanical engineering students for the period from 1904 to 1909 is shown in Table 1.

Charles M. Allen, Head of the Alden Hydraulic Laboratory since 1906, also became active in the "politics" of hydraulics in the early part of the twentieth century. On January 14, 1914, Clemens Herschel sent the Boston Society of Civil Engineers (BSCE) a letter recommending that a national testing flume be built in Washington, DC, and be operated by the US Bureau of Standards. In a March 9 letter to Herschel, Allen indicated that he had started the ball rolling in favor of the Government Testing Flume. Similarly, in the late 1910s, when there were 7 million horsepower of developed water power and an estimated 30 million to 60 million of undeveloped horsepower, Allen lobbied six influential members of Congress to increase the appropriations for the construction of river and stream gauging stations. The argument was made that information from these gauging stations was required to build new hydroelectric stations that would decrease the use of more expensive coal stations and, thereby, preserve dwindling coal reserves. (Along the same lines, it is interesting to note that Allen served as a

TABLE 1.
Experiments Undertaken by Mechanical Engineering Students

Boiler Inspection	Torsion
Efficiency of Screws	Valve Setting-Vacuum Pumps
Efficiency of Chain Hoists	Calorimeters
Platform Scales	Friction of Flow
Pump Sketching	Belt Testing
Safety Valve & Gage Testing	Pump Testing
Clack Valve	Steam Engine Testing
Tension Tests of Steel Tests of Cast Iron	Boiler Tests
Engine Clearance	Gas Engine Tests
Indicator Testing	Compound Steam Engine Tests
Steam Traps	Walpole Pumping Test
Boiler & Engine Room Piping	Cross Bending of Wood
Inspection of Hydraulics Laboratory	Current Meter & Pitometer Tests
Pitometer Tests at Chaffinville	Pipeline Friction, Venturi & Weir Tests
Friction in Pipe Line	Bending Tests of Steel Beams
Current Meter	Compression of Steel Columns
Pitometer Tests	Water Wheel Tests
Governor	Structural Steel Tests
Pelton Wheel	Compression of Wood, Long & Short Columns
Belt Losses	Valve Setting-Corliss Engine
Power Tests	Steam Engine Testing
Friction of Oils	Overshot Wheel
Shearing Bolts & Rivets	

field engineer for the Fuel Administration during World War I, made a number of inspections of manufacturers, and recommended coal conservation measures that were of great value to the Commonwealth of Massachusetts.)

During World War I (1914-1918), the great increase in the cost of coal and the uncertainty of its delivery due to strikes had a marked effect on the desirability of water power development.2 Congress took comprehensive action and passed the Federal Water Power Act of 1920 (also amended in 1921). This act fundamentally altered hydropower development by placing the licensing of plants on navigable streams under the control of the Federal Power Commission. The function of the commission was "to provide for the improvement of navigation, the development of water power, and the use of the lands of the United States in relation thereto."

Also, the National Defense Act of 1916 authorized the federal government to construct dams for nitrate munitions plants. These plants were designed so that they could be easily transformed to produce fertilizer once the war was over. The Wilson Dam at Muscle Shoals on the Tennessee River was to provide power for one such plant. However, the dam was not completed until after World War I. (A great debate would last for 15 years until President Franklin Delano Roosevelt resolved the issue of what to do with publicly built plants that would be in competition with privately owned utility companies. Senator George Norris of Nebraska fought a "give-away" sale of this asset to Henry Ford or others. Because the facility was supposed to produce fertilizer, all proposals for the "sale" came before Norris' Committee on Agriculture and Forestry. In the meantime, no power was generated, and the Tennessee Valley languished in poverty generally without electrification until the mid-1930s.)

During this time, the notion of area-wide electrical grids, or "superpower zones," came into vogue. H.K. Barrows believed that comprehensive water development would alleviate electrical power shortages and interruptions.[2] A long-term overview of the growth in installed capacity for hydropower is shown in

Figure 1. Before World War I, power was generated primarily for lighting and manufacturing plants. Most manufacturers owned and operated their own power plants. However, by 1936, industry was purchasing over half of its power. While in 1914 the price of energy for domestic use averaged 8.2 cents per kilowatt hour, by 1936 the price had declined to about 4.5 cents per kilowatt hour, a decline of 45 percent over a period when inflation was approximately 40 percent. The economy of scale and the lowering of prices were accelerating the use of power, except during the worst depression years. There were few regional companies prior to World War I, and a 5 megawatt unit was considered large, and a 50 megawatt plant was confined to major load centers. With little grid interconnection, reliability required installed capacity 50 to 100 percent over peak winter load.

Early Measurement of Large Flows

In addition to the required technical information regarding river flows, the hydropower industry needed better flow measuring capabilities by the mid-1910s. From the time of the founding of the Hydraulic Laboratory at WPI, the population of the United States was growing at an average of 14 million people per year, and the use of water power was growing at an average rate of 288,000 horsepower per year. The maximum size of water turbines, however, was increasing exponentially at a rate of 1.7 times every five years. The largest turbine output 5,000 horsepower in 1895, 20,000 horsepower in the 1910s, 38,000 horsepower in 1920 and 70,000 horsepower in 1925. No longer was it possible to test such large turbines in the Holyoke, or other, test flumes to establish the flow rate to the gate setting relationship. Since flows had become too large to use weirs in many places, flows were determined by using current meters, pitot tubes, Venturi meters, the color velocity method and the moving screen method. The color velocity and moving screen methods are no longer used. The color velocity method basically consisted of visually observing the elapsed time between injecting a dye and observing it at a downstream point. By knowing the distance and the pipe area, the flow could be calculated. All of these methods

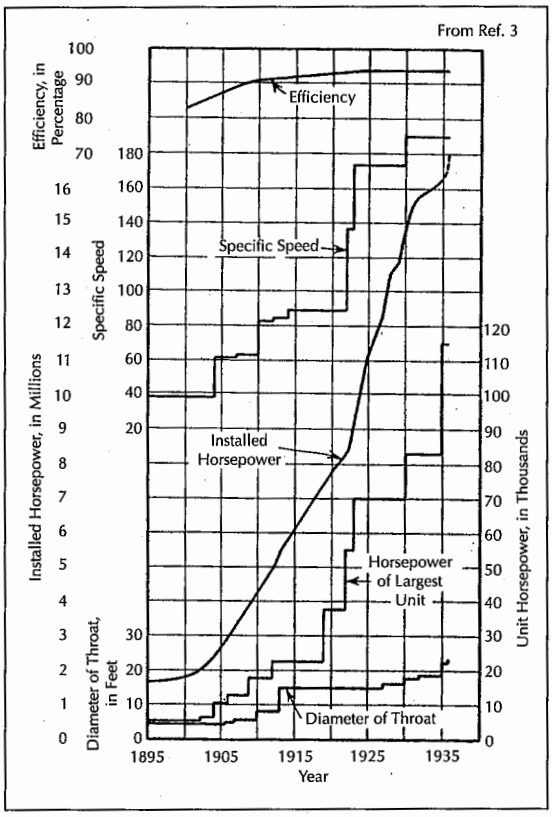


FIGURE 1. Installed capacity and progress of the hydraulic turbine in the United States from 1895 to 1936.

of large flow measurement had disadvantages either due to their high cost or because they resulted in inaccuracies greater than most engineers would tolerate, especially in turbine acceptance tests.

The first breakthrough in measuring large flows was described in a lengthy 1916 paper by B.F. Groat presented to the American Society of Civil Engineers (ASCE) and entitled "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators."[4] This technique became known as the salt dilution method of flow measurement, and is generally the same as the tracer (often dye) dilution method that is used today. In this method, a known concentration of salt solution is injected into the flow stream. After complete mixing in the flow, a sample is extracted. By knowing the initial and final salt concentrations, the flow rate can be determined. Allen used this technique for acceptance tests conducted on a

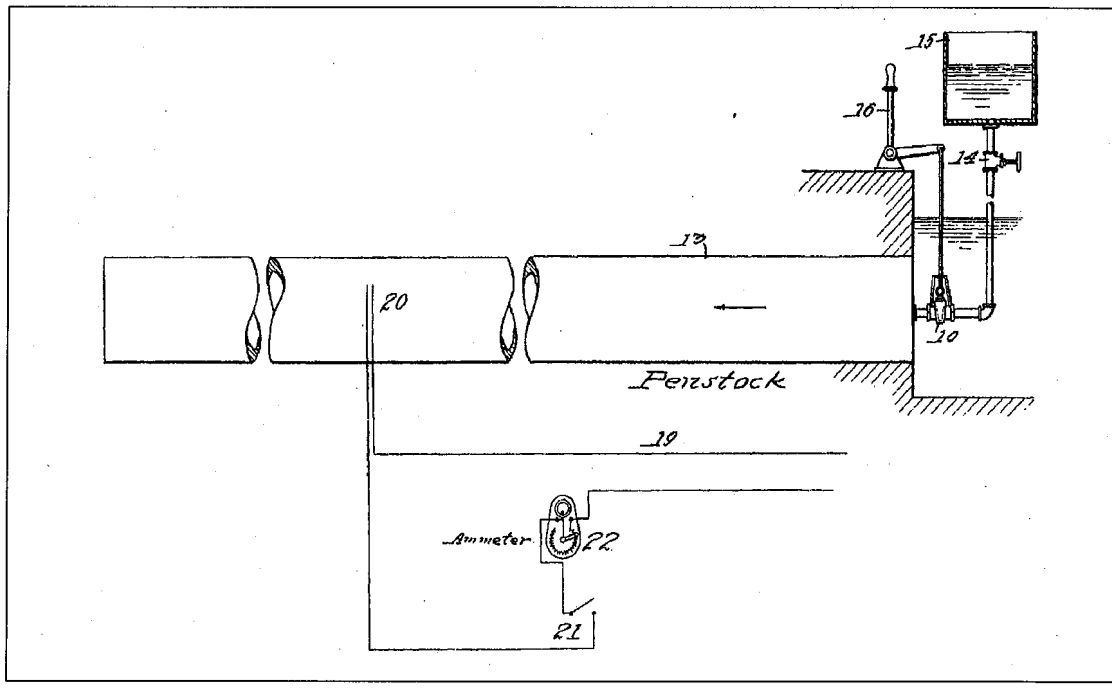


FIGURE 2. Allen's early salt velocity patent drawing (1926).

3,540-horsepower turbine for a power company in Hiram, Maine, on September 25 to 28, 1917. Allen had first proposed doing pitot tube traverses in the 11-foot diameter penstock to obtain the flow rate. However, because the power company would only pay for the turbines based on the results from the tests, the manufacturer only consented to the salt dilution method.

Allen secured the services of Professor Fredric Bennet, Jr., of the WPI Chemistry Department to perform the necessary chemical work for the tests. One ton of salt per test was used in a 15- to 20-minute run. Many salt-water samples were collected during each run. The salt concentration was obtained by evaporating a water sample using a hot plate, and then weighing the remaining dry salt. At the end of the test, the manufacturer was paid the agreed price for the turbine, based on achieving the 89 percent efficiency as specified by the power company.

It is pertinent to note that Groat's extensive paper dealt in detail on errors and methods that are taken for granted today.[4] Allen probably had numerous ideas based on his experience with Groat's salt dilution method. Multiple distribution points, multiple sampling points, mixing and other factors were all considered.

Salt Velocity Method

In 1916, the same year that Groat gave his paper on salt dilution, Allen began experimenting with a similar technique, which was later called the salt velocity method. This technique was based on the fact that salt increases the conductivity of water. If you could accurately measure the time it takes a salt cloud to go from the injection point to a measuring station, and you could measure the volume between these two points, you could determine the volumetric flow rate in the penstock. The time at which the salt cloud reached the measuring station could be determined by continuously measuring the conductivity of the water.

It was not until five years later that the first salt velocity tests were conducted at the laboratory. During the initial testing, the injection station was located at the intake of the penstock into the laboratory. A turbine was set at different gate openings. The electrode that would measure the conductivity of the water was connected to an ammeter and consisted of two copper strips 2 inches wide separated by wooden

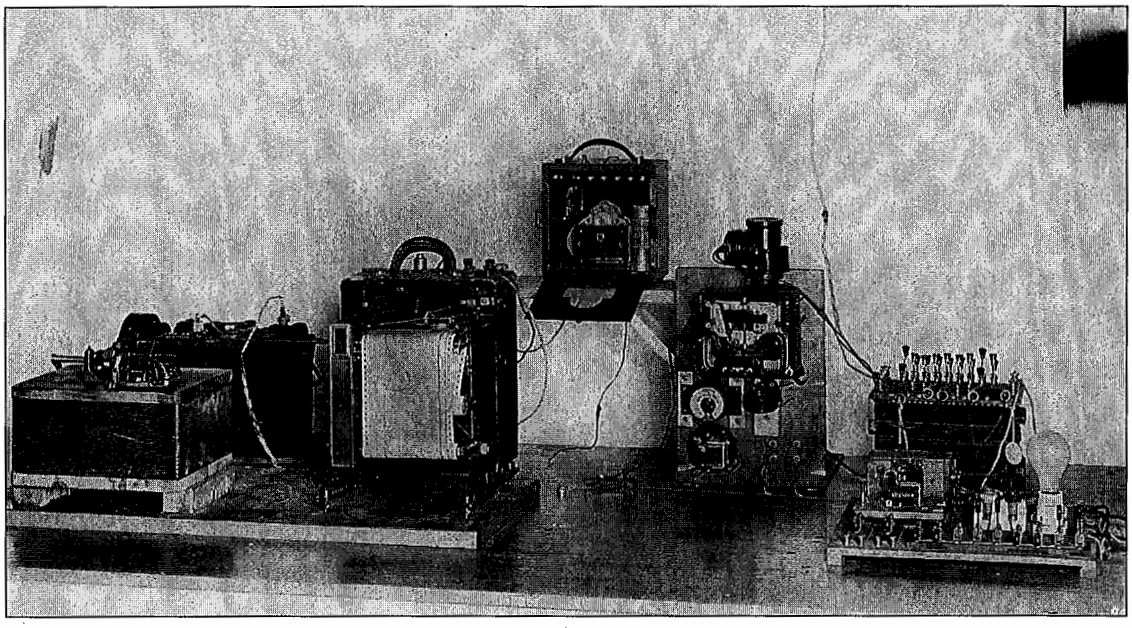


FIGURE 3. The recording apparatus used for Allen's salt velocity method.

blocks. The flow rates were measured with a 12-foot weir located in the basement of the laboratory.

The first test consisted of throwing a bag of salt in the intake, starting a stop watch, running down to the electrode and stopping the watch at the indication of the first appearance of the salt. This crude injection technique was refined twice. First, a hinged box containing a quantity of salt was lowered to the mouth of the intake and the salt was released by using a string attached to the hinged cover. The second refinement, which ultimately was implemented for all salt velocity tests, consisted of a quick-acting valve located at the intake centerline and connected by a 2-inch pipe to a mixing tank elevated above the surface of the pond. Allen's 1926 patent drawing reflects these improvements (see Figure 2).

Soon after the first few tests, a telephone line was installed between the pond and the laboratory that replaced the operator who had to sprint from the intake. Timing was also changed to include both the time of the initial and final indication of the salt passing by the electrode. Figure 3 shows the apparatus used for recording the test data.

About 400 charges of salt were used in 30 runs at different gate settings of the turbine during these initial tests. The results of each individual test were within 1 percent of the weir measurements. In addition to the weir, the color velocity was used to check the flow rate. In this method, a red aniline dye was injected at the penstock inlet, and the dye was observed in an open-ended 2-inch glass pipe containing a white background. By taking the mean time between the initial and final appearance of the color, these results correlated very closely with the results obtained by the salt velocity method.

The first field tests using the salt velocity method were performed in 1920 by Professor S.M. Woodward of the State University of Iowa for measuring dredge pipe velocity in flood prevention work near Dayton, Ohio. Allen's first test was for a hydropower plant in Berlin, New Hampshire, on September 10 to 15, 1921. The tests were conducted on two units located on 13-foot diameter wood stave pipes 1,400 feet long. In addition to the salt velocity tests, current meters and the color velocity method were also employed during some of the tests. After the tests, which Allen termed successful, he returned to the laboratory and continued to work on improvements to the technique. (Even after hundreds of tests had been conducted by Allen and his successors [Professors Leslie J. Hooper and Lawrence C. Neale], the routine was al-

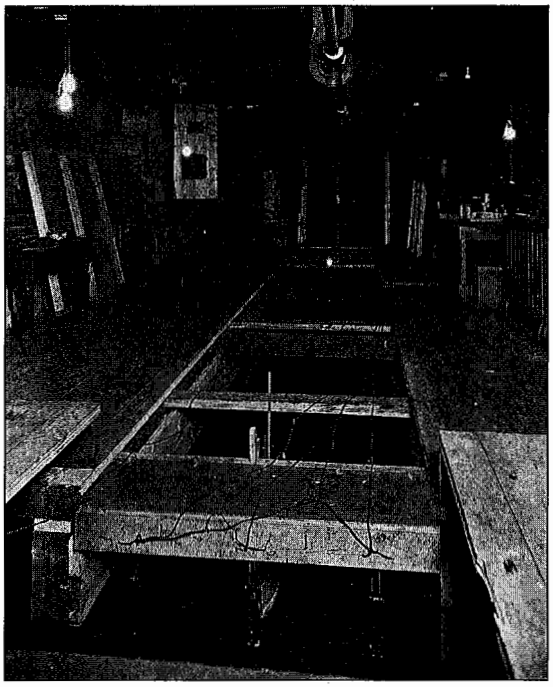


FIGURE 4. Measuring open channel flow using the salt velocity method.

ways the same after each test — namely, to return to the laboratory and try to improve the technique. To that end, test lines were almost always available at the laboratory for experimentation during the five decades that followed Allen's development of the method.)

Allen also experimented with using the salt velocity method in open channel flow (see Figure 4). A clock pendulum indicated time and multiple injection points were used.

A formal presentation of salt velocity work, "The Salt Velocity Method of Water Measurement," was presented in the 1923 American Society of Mechanical Engineers (ASME) *Transactions* by Allen and his brother-in-law Edwin A. Taylor, who had helped in the original testing and development.[5] An example of efficiency test results conducted by Allen and Taylor using the salt velocity method is shown in Figure 5. Coincidentally, another paper appeared in the same publication, entitled "The Gibson Method and Apparatus for Measuring the Flow of Water in Closed Conduits," by Norman R. Gibson, which outlined the impulse-momentum principle.[6] Both methods were successful and were later adopted as official methods endorsed by ASME and other worldwide technical organizations for measuring flow in large pipes.

Allen's creative skills, as demonstrated by the salt velocity method of flow measurement, and his other inventions, along with all his testing experience, teaching, and common sense, enabled him to formulate what he called *fundamentals* that he observed important in engineering practice:

- If you stay with a problem long enough, you will get an answer. It may not be the one you expected, but the chances are it will be the truth.
- The truth is the only thing that does not vary.
- If you really want to learn anything from an experiment, change only one condition at a time.
- Never hesitate to try a hunch. If it turns out OK, the theoretical chap will tell you why.
- If practice and theory do not agree, investigate the theory.
- Every engineer has the right to use his or her brain.
- One cannot work long with natural laws without having great respect for the "powers that be."

Hydraulic Modeling & Major Historic Events

Hydraulic models were not yet commonly accepted in engineering practice in the United States, but river modeling was used in Europe. In 1918, B.F. Groat extensively discussed similitude and advocated hydraulic scale modeling.[7] Groat used a model having proper similitude for designing submerged baffles to improve flow conditions at a hydro intake while ice was deflected. Since "seeing is believing," Groat's favorable model-prototype comparison of flow patterns on the St. Lawrence added to the modeling momentum. The prototype data were recorded by John R. Freeman in 1904.[7] It is pertinent to note Groat's correct insight on factors that can influence similitude. He discussed viscous effects, surface tension, roughness and other factors for free surface models.

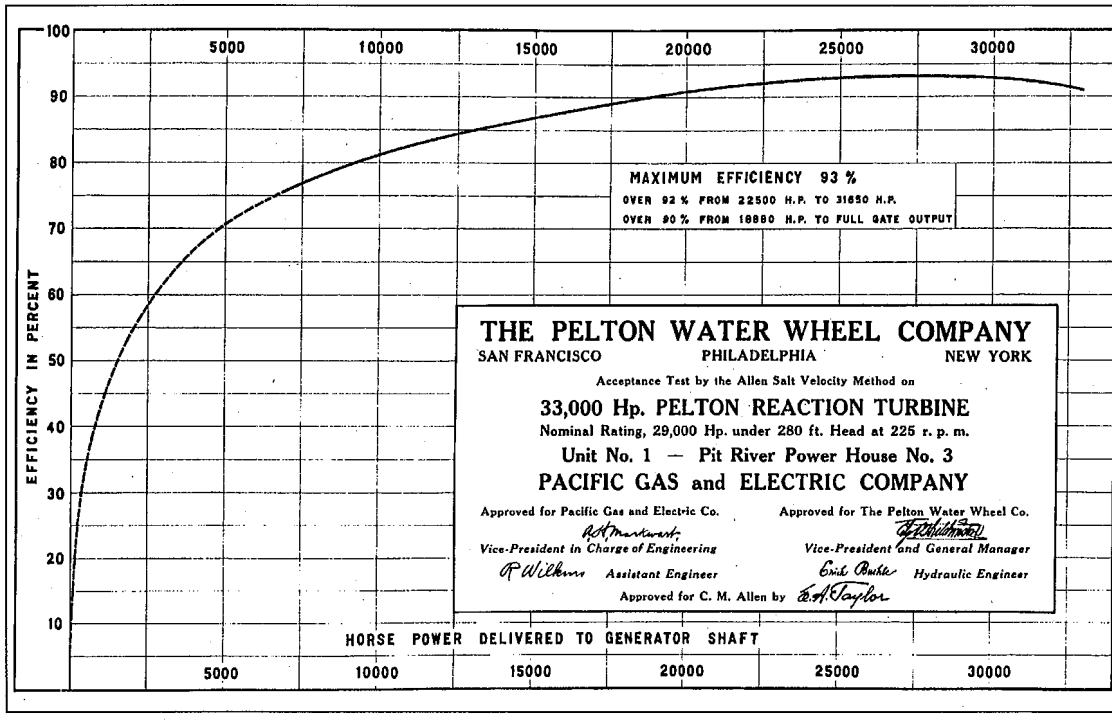


FIGURE 5. Acceptance test data using Allen's salt velocity method.

The Alden dynamometer at AHL (part of the original testing equipment at the lab) continued to be used for many other activities. Allen's 1919 files showed the dynamometer (see Figure 6) being used to test the efficiency of a gear box. Apparently, the bearings for the worm gear failed during the test or at some other time.

During the period from 1910 to 1920, major flood control works were being undertaken in the United States, prompting innovation in all phases of civil engineering, hydrological analysis, equipment design and model testing. The severe Ohio River flood of 1913 resulted in a system of flood control dams by the Miami Conservancy District in Ohio.[8] This agency developed the basic concept of the hydraulic jump pool (stilling basin) for energy dissipation, lending further credence to scale model testing. Also, major water supply projects were being developed for metropolitan areas. Models were used for the spillway for the Boonton Dam in New Jersey and models were contemplated for the Schoharie developments of the Catskill water system.

Farming in the early 1900s, and earlier, was a struggle. Rural areas were the last to be electrified, "modern" soil conservation methods were either not readily accepted or known, and large-scale efficiency had not yet started. However, during World War I there was extensive farm expansion to supply food for the war effort and export. This need combined with somewhat higher than average rainfall in marginal farm land in areas of the West and Midwest resulted in farming in areas never before having sod plowed. After the war and through the 1920s, these areas would "hang on" as adequate moisture levels continued on the Great Plains.

World War I fundamentally changed the US government and changed how most people viewed the role of the federal government. The national government became very much involved with business. The railroads were nationalized, and the War Industries Board, War Food Administration, War Finance Corporation and other such agencies made people tolerant of major new government activities as well as the centralization of authority.

During this time, other major civil-hydraulic works were completed. The Panama Canal opened in 1914. The need to combat malaria and yellow fever during its construction

FIGURE 6. Gear box testing at the WPI Hydraulic Laboratory.

helped foster large-scale, concerted public health projects. In addition the worldwide influenza epidemic that killed 20 million (one half million in the United States) pushed public health agendas to the forefront.

Sea-borne traffic between New England states and other eastern states was very extensive. Finished products were going south, and coal and other raw materials were coming north along Cape Cod through Nantucket Sound. Fog, shoals and treacherous currents led to many ship losses. In 1870, Clemens Herschel analyzed the situation and recommended a sea level canal without locks. After 250 years of surveys and false starts, the Cape Cod Canal was completed in July 1914. Because of high currents, narrowness and a poor approach from Buzzards Bay, the original canal would be widened, deepened and improved in the 1930s. At that time, the present-day car bridges and vertical lift counterweighted railroad bridge were completed.

The post-World War I years became known as "The Roaring Twenties." The United States was not interested in the League of Nations (which had been created after World War I in order to avoid another world war), but a return to normalcy. For the Alden Hydraulic Laboratory, this decade began one of growth and one in which field testing became the consulting domain of individuals (for example, Allen) and laboratory testing was concerned with modeling and flow meter calibration. Going into the 1920s, the staff at the laboratory was quite small, with most dedicated to student instruction.

Floods

Although the "The Roaring Twenties" was coined to describe the social aspects of the American scene following World War I, a similar name could be applied to one natural hydraulic phenomenon that touched thousands of Americans during the 1920s — floods. Al-

though there had been floods in the past, the floods of that decade seemed more severe and impacted many lives in the United States.

To help communities, especially along the Mississippi River, the US Congress passed legislation in 1917 to build levees. This legislation provided that at least one-third of the cost of levees built by the US government be borne by the local authority. Although the legislation was instrumental in helping build numerous flood protecting levees, it had two drawbacks. The first drawback was its funding requirement. The more populous cities, towns and districts had the most funds and, therefore, could afford to pay the one-third duty imposed by the act. Secondly, while the levees constructed protected those areas, the flood waters also rose higher and flooded areas that were unprotected by levees (some of these areas had not previously been affected).

In 1922, parts of the 1,240,050 square mile area of the Mississippi River watershed were hit with large floods that caused considerable damage. Larger floods were experienced in the same watershed in 1927 when 20,000 square miles were flooded, leaving 380 dead and 700,000 homeless. (By comparison, the floods of June to August of 1993 in roughly the same areas caused 50 deaths and left 70,000 homeless. The week of November 3, 1927, also saw the greatest flood in New England since 1869.)

Partly as the result of floods such as these, and from his own work and the work of others, John R. Freeman began to promote hydraulic model testing and the construction of a national hydraulic laboratory. In his 1922 address at the ASCE national convention in New Hampshire, he discussed the need for such a laboratory.[9] A technical paper written in 1924 outlined what equipment and buildings would be required for a national laboratory. Freeman even included 65 different hydraulic problems that the national laboratory could pursue.[10]

In the same year, Freeman visited numerous European laboratories to learn about their methods and techniques. He persuaded the leaders of these laboratories to write technical descriptions of their facilities and the methods used in their testing. All of this information was gathered and published, first in German and then enlarged in 1929 in English, in an 868-page book entitled *Hydraulic Laboratory Practice.*[11] Included in this book was a straightforward discussion on similitude. With Freeman's stature, the profession started to pay attention. Also, the great cost saving associated with modeling was evident. Many millions had been spent in a haphazard manner (some would say) on flood protection along the Mississippi River. At this time there was still limited experience with the performance of new large projects, and many engineers were concerned with the various aspects of river modeling.

Freeman was so convinced that the United States should have a national laboratory, and adopt modeling techniques such as those used in European laboratories, that he established traveling scholarships in 1924, 1925 and 1926 to study these overseas laboratories. (Seven decades after their establishment, Freeman Scholarships are still available for studies of hydraulic-related topics.)

Physical Hydraulic Modeling & Laboratory Expansion

In the 1924 *Transactions of the ASCE,* when John Freeman was pleading the economic need and value of a national laboratory, B.F. Groat, formerly a professor at the University of Minnesota and at the time a consulting engineer in Philadelphia, firmly stated his support.[10] Groat described the results of a 1:100 scale model that was used for a temporary stone closure across the South Sault Channel of the St. Lawrence River. The stone was sized from the study, and the model was vindicated. Groat even described a model-prototype comparison of behavior when the project superintendent, having no large stone, started the work with smaller stone that washed away. Groat proposed: "One use for a laboratory . . . is the publication of the physical laws by which correct interpretations may be placed . . . relative to the action of the full size river. Many such laws are now known to a limited number of engineers . . . The National Hydraulic Laboratory (should) prove these laws and publish the information."

Allen was also attuned to the future need of model testing. In the early 1920s, he had Gleason H. McCullough, a future head of WPI's Mechanical Engineering Department, draw up plans for a new and larger main laboratory

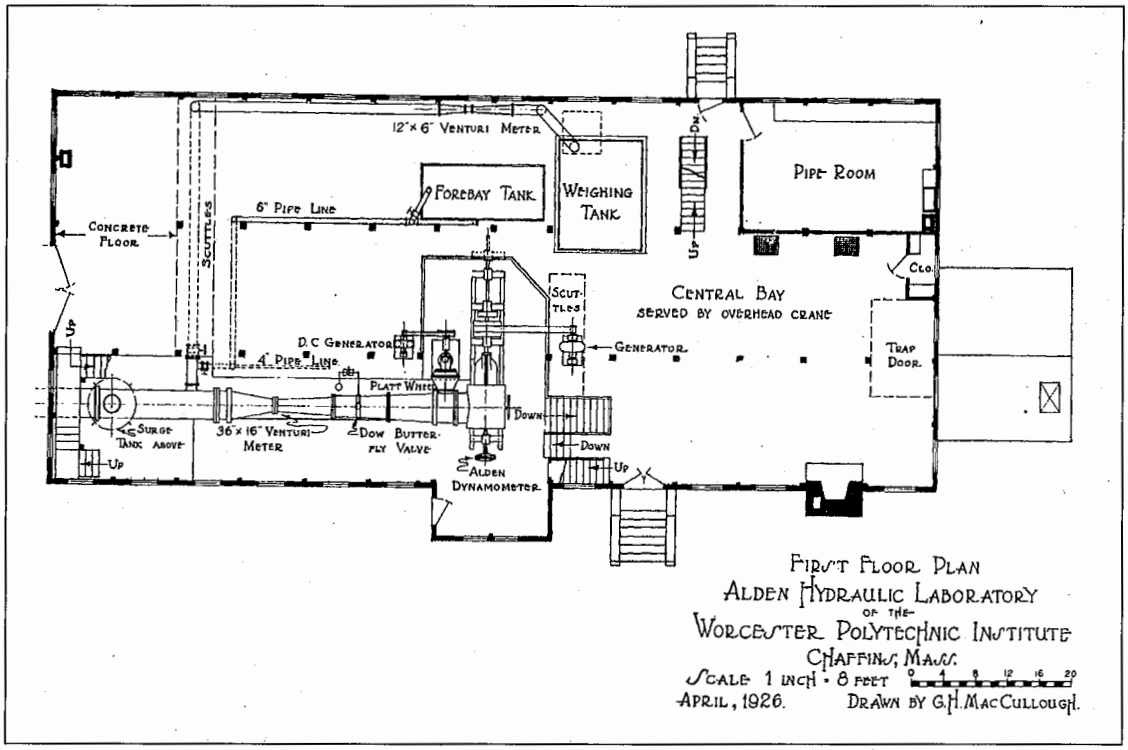


FIGURE 7. Layout of the first floor of the Alden Hydraulic Laboratory (1926).

building using the existing building as a starting point. In 1924, thirty years after the start of AHL, Allen saw that it was time to expand and renovate. In the summer of 1924, the foundation for a new facility (now Building 2) was erected around the existing Hydraulic Testing Station. In the fall of that year, Allen took Alden (the founder of the laboratory) out to see the new foundation and hoped to convince him to furnish funding for the new building. Alden agreed to the funding, as a provision of his will. Using the old building as a staging, a new building was built during the next year and a half at a cost of $42,000. The dedication of the facility was on May 7, 1926. The building represented the last direct gift from Alden to the laboratory since he died four months later on September 13, 1926.

The new main laboratory building at that time was 110 feet long by 45 feet wide, with two towers — one for head measuring water columns and the second tower for a surge tank. Other existing buildings included the low-head lab, a small office building, a storehouse and a wood shed. The total floor space of all buildings was approximately 19,000 square feet. The layout of the first floor of the Alden Hydraulic Laboratory in April 1926 is shown in Figure 7. The surrounding land included over 100 acres of woods. An aerial view of the area is shown in Figure 8.

In 1922, a resolution was introduced in the US Senate for a national hydraulic laboratory, but the US Army Corps of Engineers blocked it. Not until 1928, when a great flood occurred on the Mississippi with tremendous loss of life and property, did the Corps of Engineers (directed by Congress) recommend (as a self-protective move) the establishment of a hydraulic laboratory under its own auspices.

Feverish Modeling & Consulting

The Alden Hydraulic Laboratory was a point of interest to many engineers during this period. In his 1923 report to the president of WPI, Allen mentioned that "during the past year we have had as visitors all of the Chief Engineers of all the Water Wheel Companies, as well as a large number of consulting hydraulic engineers not only of this country but from Canada,

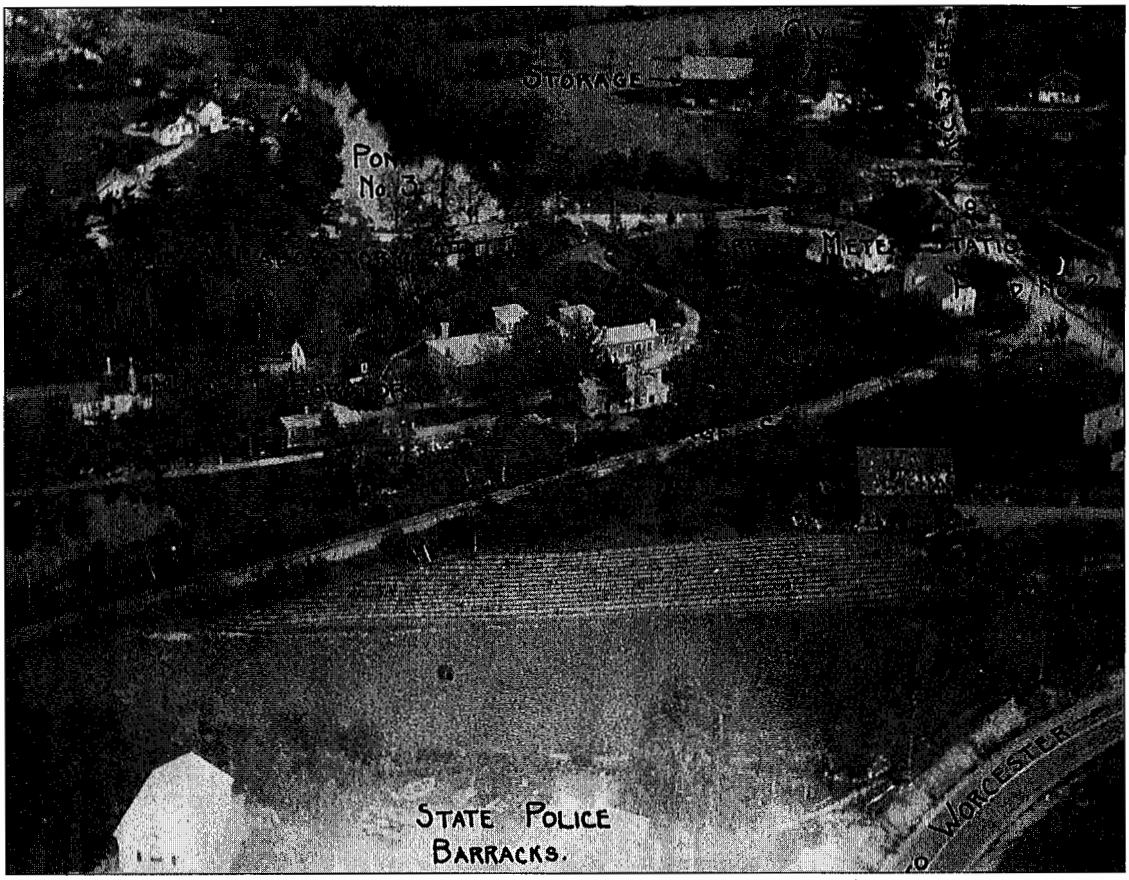


FIGURE 8. An aerial view of the laboratory and surrounding area (1930).

England, Sweden, etc." These visits enabled Allen and the visitors to discus various aspects of hydraulic engineering, thereby keeping abreast of current national and international practices. These practices included the use of various materials for simulating roughness and shape. In one of Allen's spillway models, for example, equal portions of sawdust and cement were used for easy shaping using carpenter's tools.

The model activities at AHL involving power production and flood control were so numerous in the 1920s that, of the 26 most important power projects in the United States listed in the magazine *Power* in January 1930, nine had been modeled at the laboratory. The total installed capacity of the 26 projects was 2.2 million horsepower. The nine projects studied at AHL had an installed capacity of 1.1 million horsepower. In the late 1920s, rivers flowing (by proxy) at the laboratory included the Androscoggin, Penobscot, Kennebec, Presumpscot, Deerfield, Connecticut, Westfield, Ware, Hudson, Susquehanna, Osage, Columbia, St. Lawrence and St. Maurice. During this period, Allen was also continuously refining his salt velocity flow measurement technique.

A power company in New England had studies conducted for the Fifteen Mile Fall power site (also called Comerford) on the Connecticut River (see Figure 9) and on the Davis Bridge Dam power project on the Deerfield River, a tributary of the Connecticut. A power company in New York had canal model studies done on the Spiers Falls site on the Hudson River. At this time, physical models were either tested in open channels, in the low head laboratory or in the basement of the main laboratory. The Davis Bridge project involved a novel spillway (previously used in Europe) that was called a "morning glory" because of its shape. As far as can be determined from existing files,

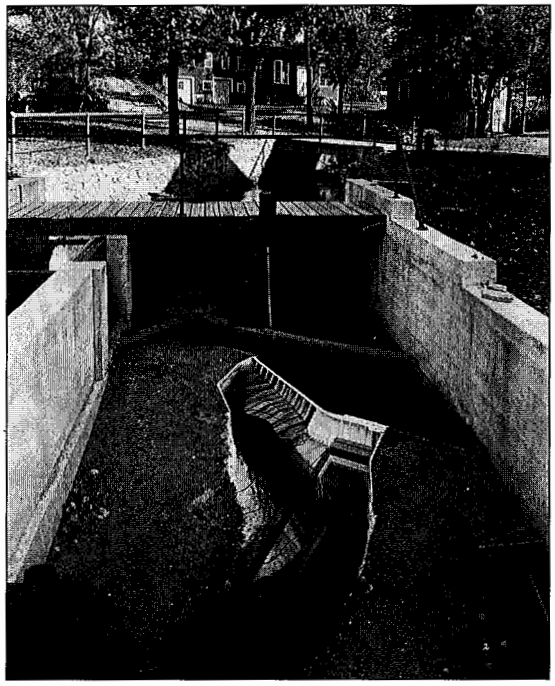


FIGURE 9. Fifteen Mile Fall model.

the 1922 testing of the 1:36 geometric scale Davis Bridge "morning glory" intake was the earliest hydraulic model test at AHL. This shaft spillway was designed for 27,000 cfs and would have to pass this maximum flood without any entraining air that would reduce capacity. A rendition of the Davis Bridge Project is shown in Figure 10, and the capacity of the spillway based on the 1:36 scale hydraulic model is shown in Figure 11.

The Davis Bridge Project was a record 200-foot high hydraulic fill dam on the Deerfield River at Davis Bridge (now called Harriman Station) in Whitingham, Vermont.[4] The hydroelectric plant was constructed from 1922 to 1924 and had many special features including a 13,000-foot long, 14-foot diameter power tunnel with a surge tank. The plant had a total capacity of 38.5 megawatts, for three units operating with a head of 340 feet.

The first Davis Bridge intake tests simulated the intake crest and tunnel, which was located in a deep pool that did not have any channel effects. The actual channel shape was designed so that part of the spillway was fronted by deep water while the other half had a special channel. Model roughness similitude was a concern in the 1:36 model. These tests indicated satisfactory flow capacity with limited air entrainment. Subatmospheric pressures were meas-

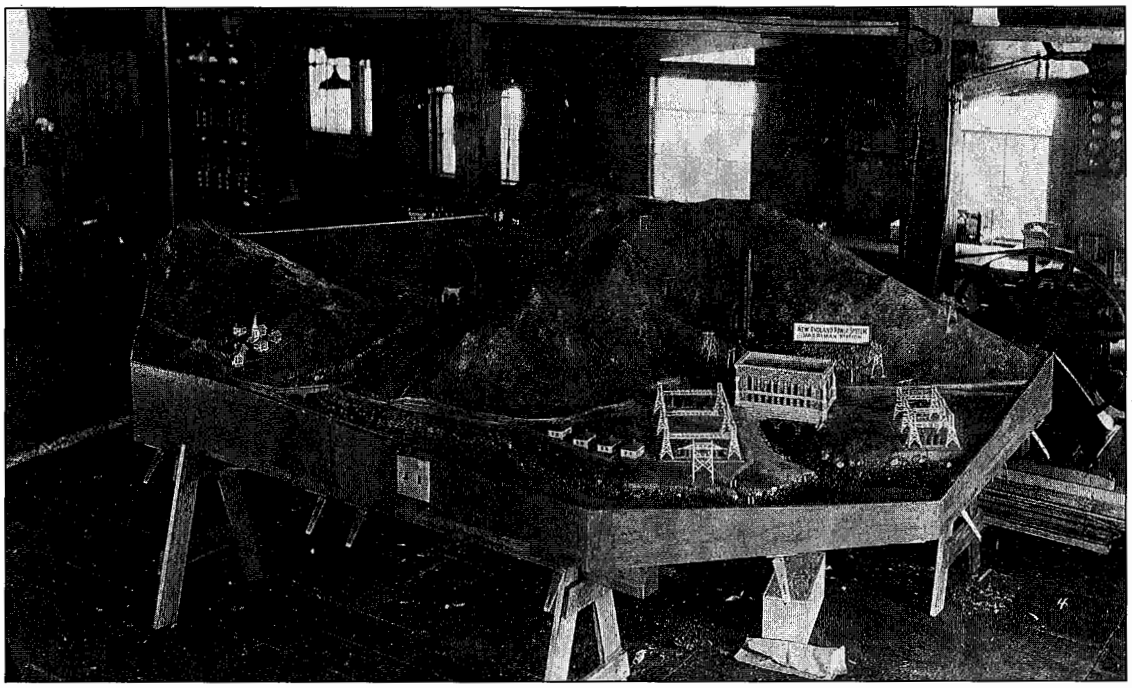
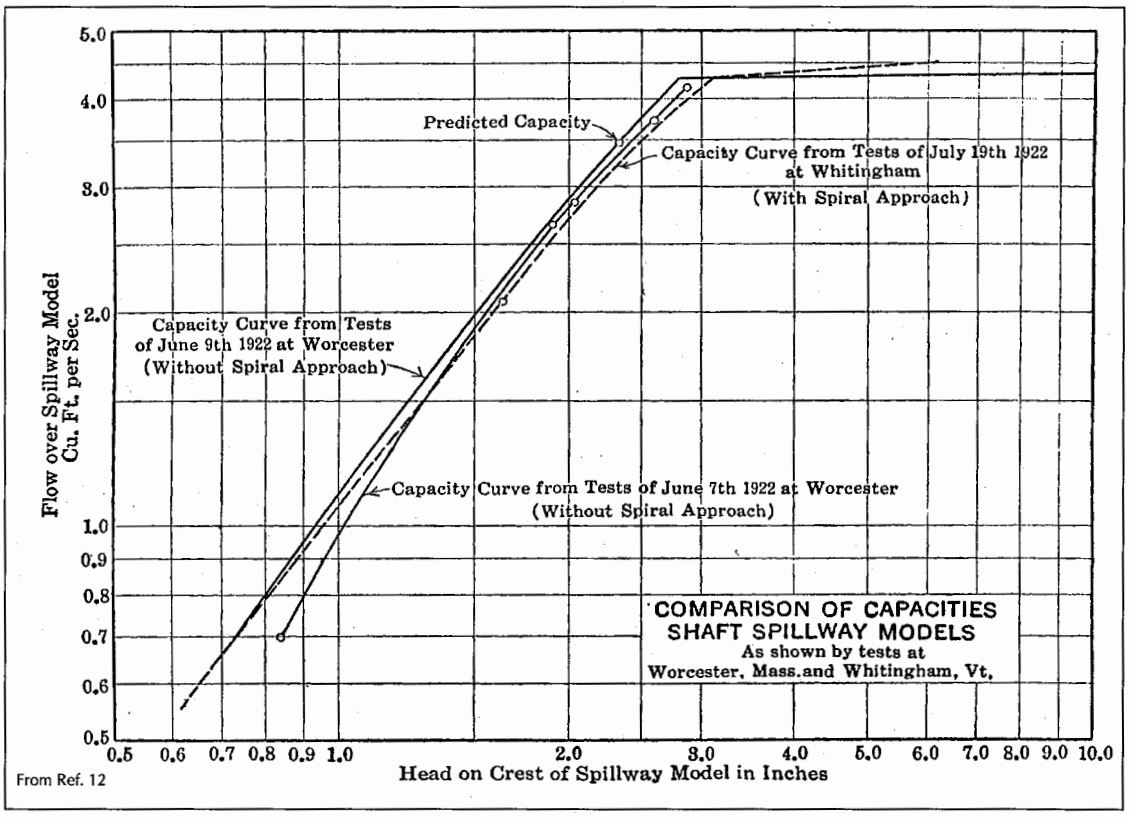


FIGURE 10. An architectural model of the Davis Bridge Project.



From Ref. 12

FIGURE 11. Davis Bridge "morning glory" spillway capacity.

ured on the spillway crest profile. Overhead photographs were taken to show flow patterns. The model was later tested at the Massachusetts Institute of Technology (MIT), and a discussion of the paper by Fred Kurtz touched on similitude limitations of air entrainment.[12] Another early study was done in 1924 at AHL for an outlet stilling basin of a twin-pipe bypass between the Middle and Lower Canals in Rumford, Maine.

Most projects involved power production and the safe control of flood water. The Conowingo, Holtwood and the Safe Harbor studies on the Susquehanna River for an electric power company in Philadelphia were important to the power development and flood control on this river. Various aspects of the Conowingo Project would be evaluated by the laboratory over the years (even up to the present). These studies would include spillway flow capacity, fish passage and turbine flow measurement. For a 1927 model study, the focus was to design an apron so that erosion adjacent to the toe of the dam would be minimal. A 1:30 scale section of the dam was constructed in a flume (see Figure 12) and numerous apron designs were evaluated. It is interesting to note that large crushed rock was used to conservatively represent the large blocks of granite in the actual river. In an *Engineering News Record* article written five years later, a favorable comparison is made between the model and prototype erosion.[13]

Draft Tube Modeling

In the south, a large power company had a major facility completed — Mitchell Dam — which required hydraulic investigations that were conducted at the power company's own hydraulic laboratory. This project would have a major impact on turbine design and Allen. Hydropower was becoming a major study area in civil engineering. In 1921, a hydroelectric option was offered at MIT,[2] and water power engineering courses were being offered at WPI.

The Alden Hydraulic Laboratory in the 1920s was not only doing work in modeling

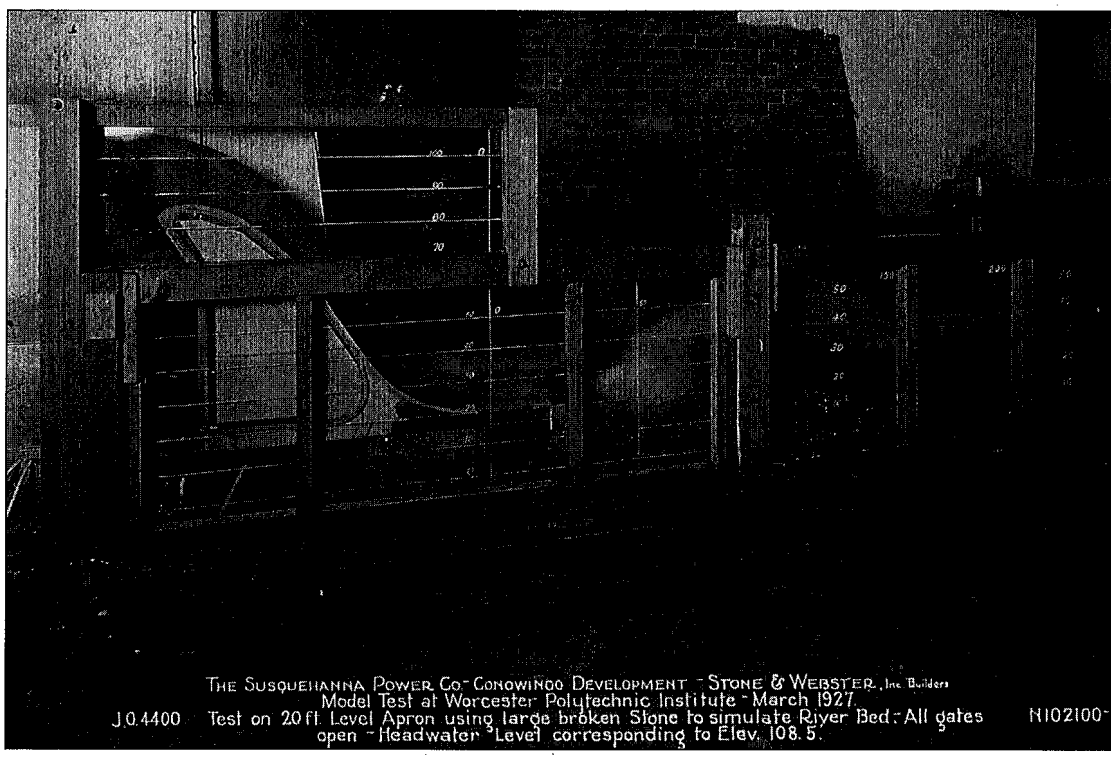


FIGURE 12. Model of the Conowigo spillway apron.

power and flood protection projects, it was also performing extensive modeling to improve draft tubes for two power companies. In 1922, classic experiments were conducted at the laboratory on 12 experimental draft tubes.[14] A Francis-type model runner was used for the tests. It is interesting to note that of the 12 draft tubes, six were designed by the manufacturers, three by one of the power companies and three by Allen and I. A. Winter, a hydraulic engineer for one of the power companies. The test facility layout is shown in Figure 13 and typical data are shown in Figure 14. A technical paper entitled, "Comparative Tests on Experimental Draft Tubes," and written by Allen and Winter, was presented to ASCE in 1923.[14] This paper prompted much discussion and further established Allen and AHL in the field of hydraulics and turbine testing. The paper also was awarded the ASCE James Laurie prize in 1925 for the best paper in hydraulics. Winter was also involved in developing the Winter-Kennedy method for measuring flow using pressure taps on the scroll case of Francis or Kaplan turbines.

Growth of the Power Industry & Cities

In the early 1920s, the efficiency and scope of the US electric power industry led to real national growth — there was an increase in gross national product accompanied by less hours of labor. In many ways, this increasing competition and efficiency should have produced falling prices and increased economic benefits. However, consolidation (some said monopolization) came about in various industries, including companies that generated electric power. Engineering companies also were growing to meet design needs. For example, by 1920, in addition to all of its varied engineering, construction and appraisal activities, one company provided management services under contract to 59 utility companies in 18 states.

Upon the death of President Harding, President Coolidge in 1923 permitted economic concentration, which led to the formation of holding companies, especially in the power industry. These holding companies issued stock that became vastly overinflated in value.

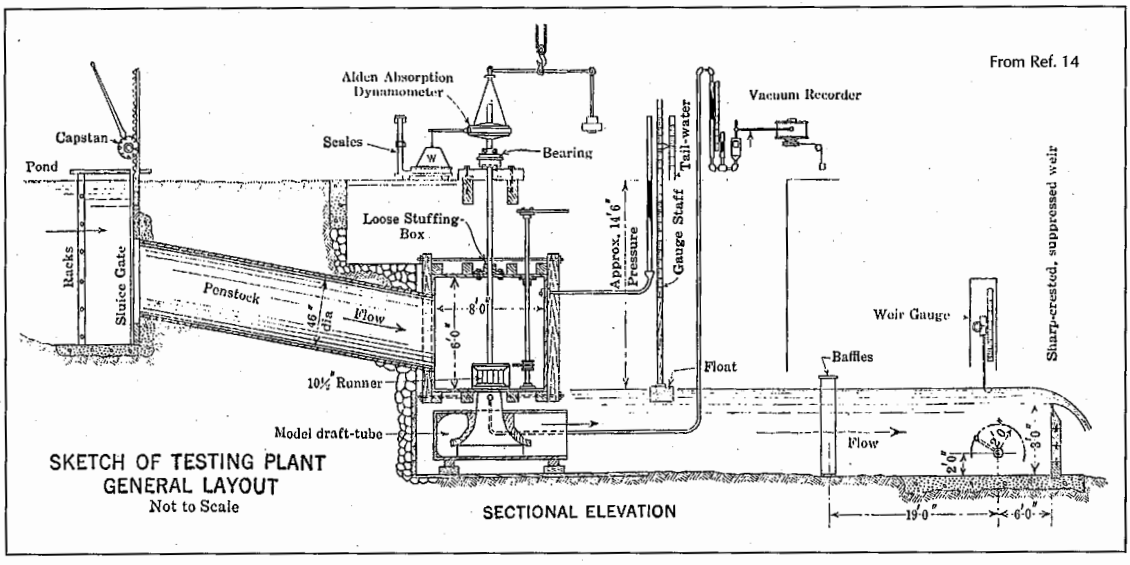


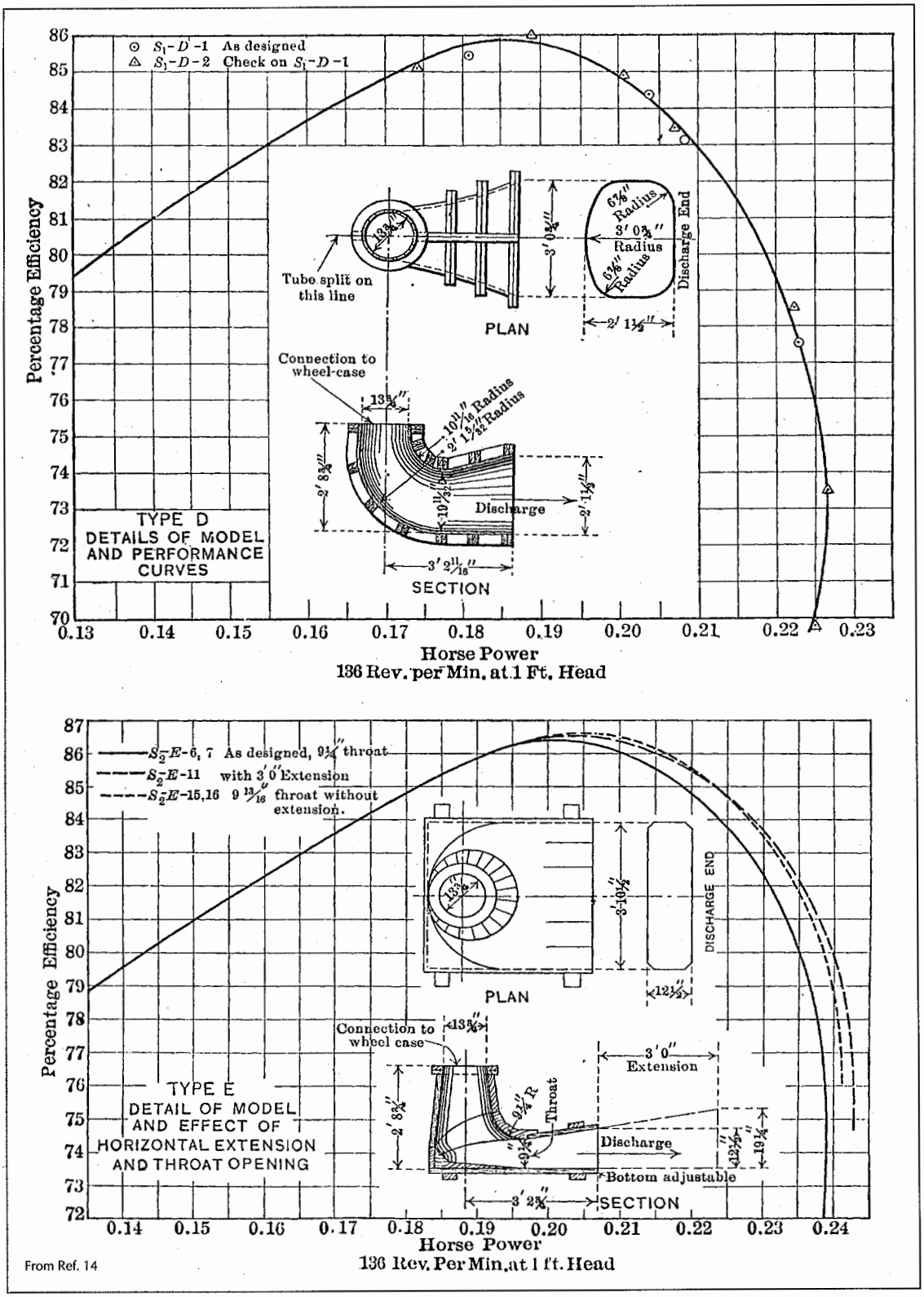
FIGURE 13. Draft tube testing plant layout.

Oversimplifying, there were holding companies for holding companies, and, in the end, few could understand this "pyramiding." By someone owning a few shares in a holding company, financial power became centralized. The shares of these holding companies were bought "on margin" with as little as 25 percent downpayment. The rest of the payment to the stock seller were borrowed from banks based on the "value" of the stocks. But the value of the stocks was inflated by the holding companies far above the worth of the operating companies. In such a climate stock speculation flourished.

By the end of the 1920s, ten large utility companies had absorbed three-fourths of the total electric light and power business. Electric power companies had very substantial economic and political power, and were not hesitant to use it. By June 1929, one engineering firm reported that during the previous six years the utility industry had sold about one-fourth of the total new stock offered to the public.

Besides the need for power and flood control, the expanding population of the United States required more drinking water. Boston, Massachusetts, for example, was outgrowing its 63-billion gallon Wachusett Reservoir which was built in 1906. At that time, that reservoir was described as the largest of its kind in the world. Frederic P. Stearns, in his 1895 report to the city, had envisioned that the Boston system would expand to the Ware, Swift, Westfield and Deerfield river valleys located in the western portion of the state. In 1920, Boston started to look at expanding the system. As part of the study, the Metropolitan District Commission Water Supply Section commissioned AHL in 1928 to study the flow and energy dissipation for Shaft No. 8 on the Wachusett-Colebrook Tunnel. Water from the Ware River was to be diverted some 240 feet vertically into the tunnel and to have a minimum of energy when it reached the tunnel. Two methods to achieve energy dissipation were studied. The first used a double pitch helical "thread" to dissipate energy by rotation, and the second had metal fins attached to the circumference of the shaft. The metal fin technique (see Figure 15 on page 21) was found to be the best method for conducting the flow to the tunnel with the least amount of energy. The shaft was constructed from wood, and narrow vertical windows were used for flow observation. Depending on the flow, 76 to 88 percent of the available energy was dissipated by the "cork screwing" flow. Prototype details of the Ware River intake are shown in Figure 16 (on page 22).

Large coastal cities were developing in the 1920s, and there was concern with the practice of offshore sewage disposal. The reclamation of sewage for fertilizer in Los Angeles was considered, but the proposal was defeated in



From Ref. 14

FIGURE 14. Comparative draft tube test data.

1921.[16] While the wastes were treated with chlorine, dispersion of the effluent continued to pollute the beaches. Eventually, Los Angeles constructed a 5,093-foot long, 7-foot diameter pipe along the sea bottom. The wastes were discharged offshore using two 5-foot diameter branching tunnels. However, problems developed due to fruit and vegetable canneries overtaxing the system. Gasoline and oil discharges were not permitted, and some consideration for the disposal of industrial wastes was being considered. The concept of distributing discharges over a large area eventually led to the diffuser for dilution of wastes (and waste heat) as proposed by Rawn, Bowerman and Brooks.[17]

Ocean travel was also popular in the 1920s. Ship log pitometers were instruments being used to measure the speed of large ocean liners. In those days, AHL calibrated many of these pitometers. In 1926, in nearby Auburn, Massachusetts, Dr. Robert Goddard showed the practicality of liquid fueled rockets. In 1927, Charles Lindberg was the first to fly across the Atlantic.

Special projects at Alden Hydraulic Laboratory were ongoing. One study in 1929 related to the efficiency of a portable fire pump that was driven by the wheel of a car. Allen's report states:

> "The pump was bolted to the running board of a Peerless six cylinder automobile and adjusted so that the pump pulley came tight against the tire on the left rear wheel... The car was jacked up and blocked... The first set of curves show the discharge of the pump at various pressures, each curve being plotted for a different speedometer reading... The effect of greasing the pump is shown... The tire which was driving the pump was smooth with the tread practically worn off... With a new tire the pump would certainly run better."

The tests were made by Clyde Hubbard, and selected data are shown in Figure 17 (on page 23).

The electric power industry was evolving, and production costs were relatively low. There was also the realization that hydropower could be adapted to carry peak load. Conowingo, Safe Harbor and Fifteen Mile Falls were described as peak load plants complementary to large steam plants. At this point in time, transmission voltages reached 330 kilovolts, and the length of transmission extended to 265 miles.

FIGURE 15. A close-up view of a section of the model of Shaft No. 8 showing fin-ribs.

Changes at the Laboratory

Throughout the 1920s, Allen managed to run the Alden Hydraulic Laboratory as a state-of-the-art facility. In 1922, he lined the 50,000-pound weigh tank with copper to reduce leakage and to improve the accuracy of calibrating flow meters. By the late 1920s, Venturi meters ranging in size from 1.5 to 12 inches were being calibrated. In the spring of 1924, a model river gauging station was built on the stream flowing through the property downstream of the main laboratory. The station was equipped with a small concrete dam containing a triangular weir and a building with an automatic water level recorder. In 1925, a mess hall for the staff was constructed using lumber from the old laboratory. An equalizing pond below the laboratory was developed by building a dam

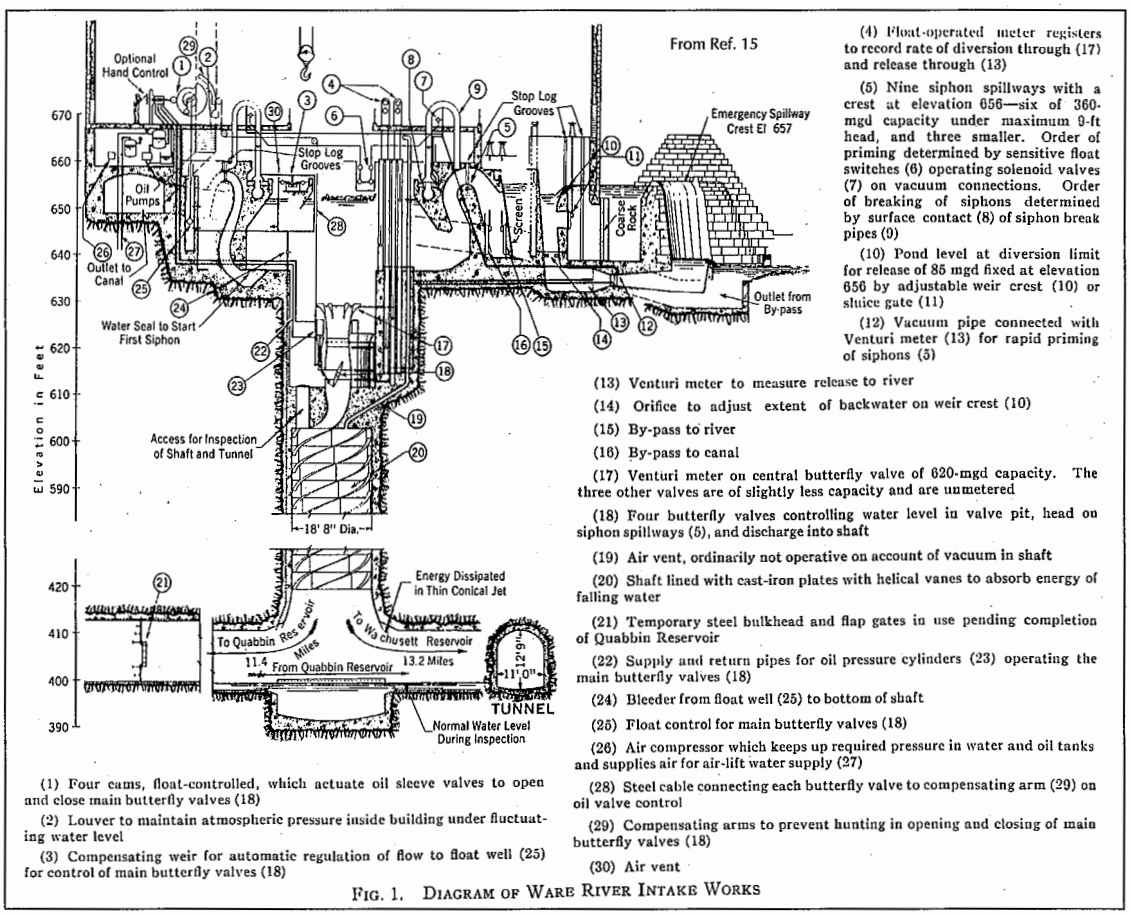


FIGURE 16. The Ware River intake works.

on that site in 1928. An addition to the carpenter shop was built in 1929, as well as a permanent building to store idle apparatus.

Always attuned to new technology, Allen convinced R.D. Johnson to design a differential surge tank (a relatively new concept) to protect the main pipeline to the laboratory. The inner portion of this tank was constructed so it could be hoisted up in the air and, thereby, convert the surge tank to a simple surge tank. Thus, the tank could be utilized by students to study full-size surge tank designs. This tank was one of the prides of Allen, and he frequently demonstrated it to visitors. On one occasion, Allen had sent his visitors to the surge tank tower to observe the oscillating water level. Unknown to Allen, the surge tank had been set up to act as a simple tank. With the waterwheel going at full power, Allen quickly shut down the flow by closing the turbine's wicket gates. Suddenly, he heard the water overtop the tank, and a group of wet visitors came down from the tower none too happy about the demonstration. Needless to say, from that time on, Allen always checked the position of the internal riser in the surge tank before giving demonstrations. In addition, a student used the surge tank for a thesis to determine the effectiveness of measuring the transient level during turbine closure to accurately predict flow. It is interesting to note that in 1929 a model of a surge tank for the Cobble Mountain Development was tested to determine the discharge coefficient through the throat leading to the tank.

Allen's drive to stay up to date with technology is indicated by a 1927 student thesis that compared "an apparatus similar to Gibson's in its essentials, although much cheaper, can be used to give an accurate measurement of water flowing through a turbine." The ingeniousness

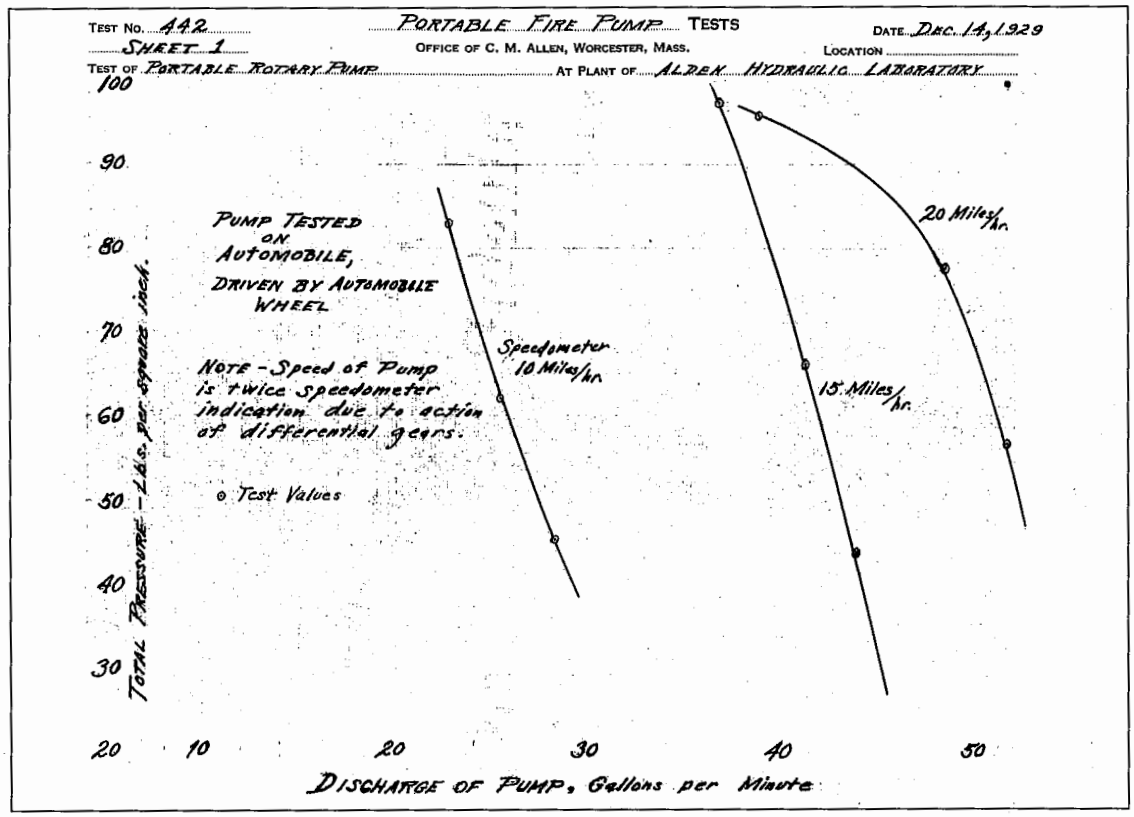


FIGURE 17. Portable fire pump test data.

of the apparatus and instrumentation is particularly noteworthy because they were all constructed at the lab. The 1927 thesis acknowledged R.D. Johnson's advice and criticism. In a similar vein, in Allen's October 1926 report to Hardy S. Ferguson, the salt velocity method was used for turbine efficiency tests in Rumford, Maine, and to calibrate permanent meters (a reducing section at the entrance to the scroll case) for measuring discharge.[18] The details of the piezometer plugs and piezometer ring are consistent with present-day practice.

In the 1920s, Allen was faced with the challenge of building a spillway on the low head pond at AHL. The exit from the pond was very narrow and, thus, precluded building a standard spillway capable of carrying the required capacity. Allen had heard of an ingenious type of spillway — a "side channel spillway" — designed by John Vaughan, a consulting engineer. In talking with Vaughan, Allen concluded that this type of spillway (a double-sided channel with a steep chute) would be exactly what was needed at this location. (Today, some seventy years after its construction, the Vaughan spillway is still active and has successfully passed every flood during its existence.)

A model valve, designed by E.A. Dow, a power company employee, was tested at AHL during the same years. After the study, Dow gave the laboratory a 36-inch Dow valve, which was used to control the flow in the 36-inch line located in the basement of the laboratory. (The valve has since been motorized and is still in use today.)

Activities at AHL in the 1920s were not limited to sponsored projects. Since about 1905, civil engineering students had used the grounds for their Railroad Surveying Camp. Numerous experiments were conducted by WPI students, as well as students from Tufts University and other colleges, to supplement classroom teaching. The main laboratory was both a study and testing area, and was heated by resistance coils powered by the hydraulic turbine. In addition, 52 students worked on 26

senior theses as part of their graduation requirements. Two of the students who did their theses at the lab returned to work there. Clyde W. Hubbard graduated in 1926 and started teaching and working the same year. Leslie J. Hooper graduated in 1924 and began working in Sao Paolo, Brazil, as a test engineer. His job was to measure the efficiency of the many hydraulic turbines owned by his employer. From his undergraduate days at WPI, Hooper had become familiar with the salt velocity method of flow measurement, and he proposed using the technique in Brazil. He corresponded with Allen frequently regarding the method and probably was the first to use the method in the field aside from Allen. In 1927, Allen persuaded Hooper to return to WPI to study for an advanced degree and to work at the lab.

Recognition for the Alden Hydraulic Laboratory & Allen

Recognition for AHL in the 1920s came in the form of awards to Allen for his significant contributions in several areas of hydraulics. In addition to the 1925 ASCE award for best hydraulics paper, Allen was named president of BSCE, received the Clemens Herschel Award from BSCE in 1928 and was presented with an honorary Doctor of Engineering degree from his alma mater, WPI, in June 1929.

Freeman wrote in *Hydraulic Laboratory Practice* describing three notable US laboratories: "namely those at Cornell University, the University of Iowa, and Worcester Polytechnic Institute. . . in addition to their use in the instruction of undergraduates [they] have led all others during the past 10 or 20 years. . . [being] broadly useful to engineers."[11] Freeman's enormous stature as a hydraulic engineer and his belief in scale models, plus growing experience in modeling and similitude, accelerated the use of modeling throughout the United States in all irrigation, power and flood control activities. Freeman's list of Unsolved Problems in River Mechanics was probably like a "call to arms."[11] This growing use of modeling was also prompted by difficulties occurring at various sites such as the costly repair work at Wilson Dam at Muscle Shoals in 1927 where extensive erosion resulted due to the lack of an adequate stilling basin.

When Allen became president of BSCE, it was no surprise that his address at the annual meeting was titlted "Water." For background information, during the 1920s, the cost of coal was soaring and there were coal car shortages and labor problems (in 1919, 3.2 pounds of coal were consumed per kilowatt hour) — factors that promoted the use of water power. Allen commented on the use and conservation of natural resources, water power and Niagara Falls:

"*Use of Water Power*. I would not feel justified in writing this paper if I did not say a few words on the use of water power. I believe that all the water power should be developed that can be shown to be financially economical, and I believe we should go even a step beyond, in view of the probable increase in the cost of coal and also to save all we can. If we don't use water power, we lose it; while if we don't use coal, we save it!

"Furthermore, I believe we should look ahead a bit and imagine what our descendants are likely to call us for using up and wasting so many of our natural resources.

"One of the best ways to conserve coal is *not* to use it — by using water power instead. I believe the time will come, and not so very far distant, when as a nation we shall arrange with Canada to use all of the water at Niagara for power purposes. If I am correctly informed, something must be done before long to prevent the Falls from automatically wearing out. To my mind the simplest way to prevent a thing from wearing out is not to use it. Therefore, why not use the water for power purposes, and incidentally save the Falls for exhibition purposes on Sundays and holidays? I realize the howl that would go up if such a proposition were really put forward in earnest, especially from people who believe that the Falls were put there to be looked at. As comparatively few people ever see the Falls, why not distribute their beauty (although in another form) by putting more horse power behind each individual in the country? This would enable a wider distribution of the good things of life."

During the 1920s, the Alden Hydraulic Laboratory effectively bcame a national resource for developing the hydraulic aspects of the many projects being constructed for power and flood control. Until the early 1920s, the laboratory was mostly used for undergraduate instruction and as a resource for WPI staff in their consulting and applied research. In the mid-1920s, research files began appearing in the name of the laboratory rather than in the name of an individual. Prior to this, most of the work is documented in the files of Allen. With the hiring of both Hubbard and Hooper, the laboratory also acquired its first full-time professional staff.

Many of the lab's clients (engineers) were involved with some day-to-day testing. A 1929 letter from Allen to a client regarding a manifold with a pump for the Rocky River Station in Connecticut, the first pumped storage plant in the United States, indicates the major ongoing activities. Various other manifolds were being tested or bid upon.[19] Unit prices in the bid for a 1:17 manifold with an actual dimension of 1 foot for the penstock are noted in Table 2. Allen's estimate was $2,000 to $3,000, "unless you have a problem different from what I think necessary."

TABLE 2.
Unit Prices for a Manifold Under Test

Expert laboratory assistance doing the setting up and conduct of tests: $25/day
Expert mechanic and model maker: $20/day
Assistant mechanic: $15/day
Common labor: $0.70/hour
Personal services on supervision and consultation: $100/day
Laboratory fee: 25 percent of total of above items

Rock Island Dam & Fishways

An extensive model study was commissioned in 1929 to research the flow conditions on the Columbia River at the Rock Island dam site that included the dam, spillway and powerhouse. The 1:100 scale model was constructed using a sand, cement and sawdust mixture. The prototype contour map had been enlarged on 6-foot square sheets of brown paper using a reflectoscope. Dowel pins were used in the model to get the exact elevation. According to newspaper articles, the model cost $15,000, while the initial dam and powerhouse were estimated to cost about $15 million. The model was constructed so that the dam could be quickly installed and visitors could see the site with and without the dam on the same day. Apparently, a major function of the model was to study the number of fishways to be installed. At that time, full-grown returning salmon on the Columbia River weighed 15 to 20 pounds. Based on the visit to the model, the US Commissioner of Fisheries required a third fish ladder to be installed. The newspaper articles also indicated that hydraulic models were used less in the United States than in Europe, but that financiers of large hydro projects had discovered their value. This study even included an architectural model of the powerhouse to follow the construction of that building.

Depression

In the mid- to late 1920s, some areas of the country were prospering while others were severely depressed. The 1928 election of Herbert Hoover, a very successful mining and consulting engineer, brought to the White House an individual who thought that economic concentration was desirable. Also, he believed that government should generally not be in the power business to compete with private industry. In Hoover's March 1929 inaugural address, he said, "I have no fears for the future of our country." (Subsequently, the shanty towns that developed all over the country were called Hoovervilles.)

In October 1929, the stock market collapsed. Throughout the stock market boom of 1929, utilities led the way. People believed that utilities were the safest of investments, and everyone could see electric power use increasing. As this was happening, Senator Norris of Nebraska was fighting the power industry to prevent a giveaway of the Muscle Shoals facility on the Tennessee River.

The Alden Hydraulic Laboratory entered the Great Depression with an outstanding reputation, excellent client contacts in the elec-

tric power industry for hydraulic modeling, Allen's steady work in flow measurement, some basic research work and a demanding requirement for academic-related activities. There was also work for municipalities (water supply and transmission) and government agencies (flood control). This effort was accomplished with a small staff. Allen continued to split his time evenly between academics and consulting. The staff was multi-disciplined so everyone "pitched in" to get a project rapidly completed. The staff included a carpenter, machinist, instrumentation technician and one or two construction helpers. Based on various correspondence, it is believed that the clients' engineers often came to assist in the testing.

During the Depression, power companies were a source of jobs and, some say, the life blood of politics at the time. During this ten-year period, power demand was doubling every six years (a 12 percent annual compounded rate).

After the Great Depression started, the holding companies symbolized to many the entrenched economic power of the Northeast because electric rates seemed to be unjustly high. Utilities refused to run power lines to rural areas because they claimed they could not get a decent return. Later studies showed that the utilities greatly inflated their cost estimates and only wanted to generate power for densely populated areas where their rate of return was higher. The abuses of the utilities led to various federal stock, financial and utility legislation in the 1930s.

Financial Crisis, Migration & Politics

As late as 1935, many rural areas, including nine out of ten farms, were not electrified. The lack of refrigeration and powered equipment caused great hardship. Manufacturers were scrambling to maintain production and artificially supported prices by offering attractive credit terms. However, the average wage-earner could not afford many of the products. Banks were enthusiastically lending money for stock purchases. Samuel Insull's power holding companies extended into 32 states from Maine to Florida. In 1932 in Texas, Insull (a former private secretary to Thomas Edison) was building the largest dam of the time. When his empire collapsed in 1932, investors lost virtually everything.

As the United States entered the 1930s, there was a national depression unlike anything that had happened in the past. In terms of length, severity and number of people affected, only the Civil War had been worse to this point in time. With no national social security, private charities and local governments tried to meet the needs of the jobless. In 1932, public land was turned over to the unemployed for gardens and almost 300,000 homeowners lost their properties by foreclosure. New construction fell 60 percent between 1931 and 1932. Unemployment would exceed 25 percent (the unemployed would reach 10 million) and millions had their wages substantially reduced. In Worcester, some office workers had their wages cut from $15 to $10 per week. In central Massachusetts, Quabbin Reservoir was built in the mid-1930s and workers were paid 25 cents per hour. It was estimated that total national income dropped 50 percent between 1929 to 1932. Five thousand banks permanently closed and nine million savings accounts were wiped out. A relief meal cost about 6 cents, but states could not afford to feed the hungry. With all of this, floods and drought still occurred.

In the Midwest, reduced rainfall began on the Great Plains in 1930. With the drought in 1933 came the wind and the dust bowls. From the Dakotas to Texas, east to the Alleghenies, wind stripped the farm land and forced the implementation of land conservation practices. Thousands of families were affected on a long-term basis, more than those affected by the floods and hurricanes of the period. From about 1935, a great migration occurred from Oklahoma, Texas and Missouri to California.

President Hoover was a conservative leader who generally believed in conventional economics. Change would have to wait for a new leader. As the Depression deepened in 1930, the Hawley-Smoot Tariff on foreign imports aggravated the crisis. Farmers and industrialists, many of the latter with near monopolies, were crying for protective tariffs on imports. Hawley-Smoot provoked retaliation by European countries on goods made in the United States. To do business in other countries, some industrialists built plants overseas. It was also

in 1930 that initial funding for Boulder (later called Hoover) Dam passed in Congress.

An overall philosophy of the Alden Hydraulic Laboratory during the 1930s can be obtained from words written by Allen in early 1940:

> "The motto of the Alden Hydraulic Laboratory was chosen from an article written some time ago by Bruce Barton: 'When you are through changing you are through.' This motto has fitted conditions at the Alden Hydraulic Laboratory ever since its beginning. It has been especially applicable during the past decade. Although the laws of gravitation and of flowing water remain unchanged, viewpoints and ideas continually change with increasing knowledge. It is, therefore, the object of the Laboratory to provide the best possible facilities for further investigation of the phenomenon of flowing water and for sane instruction in the field of hydraulics."

It was also during the 1930s that staff from the laboratory first became involved with the American Society of Mechanical Engineers' (ASME) Power Test Codes (PTCs). This association would continue for about 50 years. In 1938, Allen, Norman Gibson and others were involved with writing PTC No. 18 on hydraulic prime movers. In 1948, Allen, Leslie Hooper (by this time assistant director of the laboratory) and Clyde Hubbard (by then a principal assistant engineer for a paper manufacturer) were all involved with updating PTC 18.

John R. Freeman's active devotion towards the building of a national laboratory during the 1920s was thwarted by the politics of the day. The US Army Corps of Engineers could see no need for such a facility, and at every opportunity opposed the creation of a national laboratory. The Corps anticipated infringement on their activities and the diversion of funds. It was not until Herbert Hoover was president that this dilemma was resolved. Hoover appointed Major General Lytle Brown to head the Army Corps of Engineers, knowing full well that Brown favored a national hydraulic laboratory and a Corps laboratory. Finally, in May 1930, Hoover signed a bill establishing a national laboratory with a restriction that the laboratory could not do work in the field of any other agency. This provision was probably one reason for the laboratory's failure to thrive. (Eventually the four-story national laboratory became a part of the Bureau of Standards of the Department of Commerce.) Ironically, Freeman's detailed plans for the national laboratory, presented free of charge, were never used and Freeman withdrew completely from any activities dealing with the laboratory. This disregard for a well respected authority who had for a decade advocated a national laboratory seems cruel and unjustified. Later, the federal government established the Bureau of Reclamation laboratory, which began testing in 1930 in Fort Collins, Colorado, at the Agricultural Experiment Station. This facility had been established in 1912 by Ralph Parshall. In addition, shortly after the Tennessee Valley Authority (TVA) was established, their hydraulics laboratory was developed in 1934 in Norris, Tennessee, the construction town for the Norris Dam.

Army Corps of Engineers' Hydraulic Laboratory

The Army Corps of Engineers laboratory was established by Congress in 1928. The site for the laboratory was Vicksburg, Mississippi, the location of the Mississippi River Commission. In 1930, Major General Brown employed Allen as a consulting engineer to meet with him in Washington, DC, and to visit the laboratory site in Vicksburg, to look over the plans and layout, and to make recommendations. This consulting work reflected the respect that people had for the work being done at the Alden Hydraulic Laboratory. In taking the assignment and completing it to the best of his ability, Allen showed his integrity and honesty since both AHL and the Corps laboratory were, in effect, competing for the same work. In the days that the Corps lab was being established, many projects for the Corps were being tested at AHL.

In Allen's consulting report to Brown, he wrote:

- The choice of Lieutenant Herbert Vogel to head the laboratory was excellent in view of his enthusiasm and knowledge of modeling. (It should be noted that Vogel had been assigned to study modeling tech-

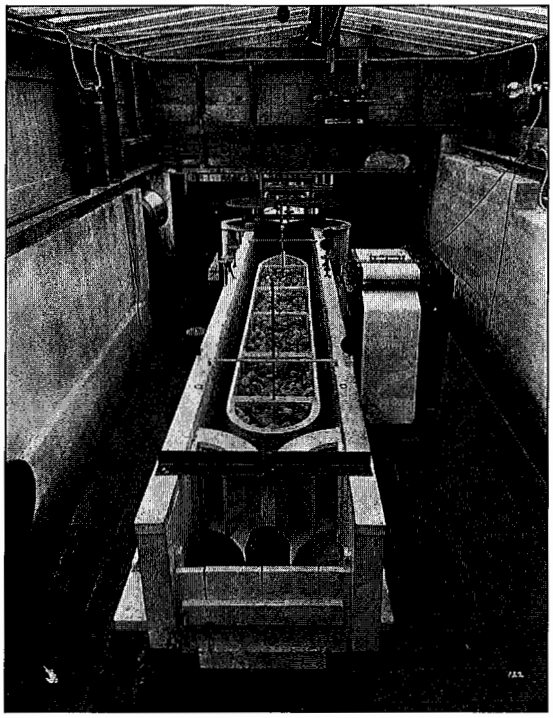


FIGURE 18. The navigation lock model for the Passamaquoddy Tidal Power Project.

niques for one year in Europe, and he joined several of the Freeman Fund Scholars during their second year in Europe. One of their major activities was visiting principal European laboratories.)

- In view of the large models that might be built, it is suggested that undeveloped land adjacent to the laboratory be acquired.
- Initially, work on models should go slowly because it would take time to get organized and to purchase the required equipment.
- There was a need for ample storage area. (This recommendation probably arose from Allen's experience with storing idle equipment.)
- To obtain the most benefit from the models, it was suggested that a good field man should be with the lab staff during the setup and testing of the models.

Corps Testing at the Alden Hydraulic Laboratory

While the Corps laboratory was being established, many of their projects were tested at the Alden Hydraulic Labaoratory. One of the biggest of these projects tested at AHL was the Passamaquoddy Tidal Power Project in Maine, a project that a half-century later is still mentioned as a possibility to produce tidal power. Six different aspects of the project were studied. Discharge flows through various gate designs were investigated using model flow rates of 23 cubic feet per second (10,350 gallons per minute). The application of sector gates for filling and emptying the proposed navigation locks was also studied. As part of the lock filling process, recorded measurements of all restraining forces on a ship in the lock were made (see Figure 18). A study was also conducted of three different methods of constructing a rock-filled dam under conditions of reversing flows. A final investigation involved plotting underwater rock trajectories during the construction of the underwater portion of the rock-filled dam.

Up until the time of World War II, the Alden Hydraulic Laboratory would conduct 11 more studies for the Army Corps of Engineers. The prototype projects were located in all the New England states except Maine. As the result of the 1927 New England floods (the worst since 1869), the spring floods of 1936 and the great hurricane of 1938, the emphasis of the studies was on flood control. Spillway calibrations as well as tunnel exit flows (outlet works) were studied for Union Village, Vermont; Knightsville Dam and Northampton Dike, Massachusetts; and Surrey Mountain, New Hampshire. In addition, flows through conduits and/or tunnels were investigated for Union Village, Vermont; Mill River, Massachusetts; and Park River, Hartford, Connecticut.

Model Testing in the 1930s

Dam and spillway studies were not only precipitated by floods but also by the need for more electric power. On May 18, 1933, the Tennessee Valley Authority (TVA) was established to control and develop the Tennessee and the Cumberland river valleys. These projects provided much work in these hard economic times and may also have been an inspiration to promote other hydropower sites in the United States.

Model spillway studies were also conducted for the Bills Brook Project in Hartford, Connecticut; the Molly Falls Project in Marshfield, Vermont; and the Holtwood and Safe Harbor projects in Pennsylvania. In 1936, a cone-valve outlet model was tested for the Hartford, Connecticut, Metropolitan Water Bureau District Commission. These tests were to prove correct design and ensure the absence of cavitation. For Holtwood, also in 1936, an outside test flume was used to determine pressure distributions on the crest and the spillway discharge coefficient. An outside model was used for Bills Brook.

The limitations of conducting model testing outside would continue for several decades. Alden Hydraulic Laboratory was unique in the large size of projects undertaken. Models were constructed in early spring for testing until the following winter. Wind breaks were used to minimize wind-induced currents.

An interesting study in 1935 involved the use of the Rock Island model that had been dormant for five years. A lawsuit had been initiated relative to damages below the dam and powerhouse. To provide information for the case, river flows were investigated with and without the dam and powerhouse in place. The diversion of flow around Rock Island in the channel below the island was also observed. The re-use of models after the initial study had been concluded occurred a number of times at AHL over the years, and this possibility is given as a reason for retaining a model for a few years after tests have been concluded.

From 1937 to 1938, New York City was investigating the expansion of its water supply system. Intake control works for a roundout on the West Branch Tunnel and a diversion tunnel on the Lackawack Dam were studied using models to ensure proper hydraulic conditions. Clyde Hubbard was very much involved with these and other tests. In correspondence between the New York City Board of Water Supply, Allen suggested — and apparently this was done — that the work be separated into two contracts and staggered in time so the total cost of the first project would be known prior to starting the second project. In Allen's words, "in this way, the overall safety margin in the bid would not be excessive and the lowest possible price would result." The extent of the tests for the control works is notable because of the complexity of the model and the concerns with capacity, efficiency of operation, minimizing air entering the tunnel and the prevention of cavitation in the full size structure. At the end of this period, Leslie Hooper became assistant director at AHL, and Clyde Hubbard left the laboratory to enter the US Navy.

Other than theses studies, the 1930s were rather quiet at Alden Hydraulic Laboratory, perhaps due to the economic times. However, pitometer, ship log and current meter calibrations continued to be performed. Flow calibrations were also performed on 20- and 24-inch Kennison flow nozzles and 12- and 20-inch Venturi meters. Flow meter calibrations were also performed for a company in Philadelphia, Pennsylvania. At least as early as 1931, AHL was doing head loss tests for various types of valves.

Consistent with the Alden Hydraulic Laboratory's philosophy of studying all types of fluid mechanic phenomena, the lab was contracted to study the best designs and features for a rowing tank for Yale University's Payne Whitney Gymnasium using a 1:5 scale model. The actual tank, powered by two 100-horsepower pumps, was first used in the winter of 1931 and 1932 to train the Yale rowing crews. Two more practice tanks were constructed at Yale using some refinements of the original tank. (Allen must have enjoyed this study that came three decades after his thesis on an oarsman's indicator.)

Further Laboratory Expansion

During the 1930s, Allen did not neglect the expansion of laboratory facilities. Using money donated by the Alden Trust Fund, Allen continued to maintain up-to-date facilities. In 1936, an addition was made to the main laboratory. It included a lecture hall on the top floor and a basement laboratory. This laboratory was later called the "student laboratory" because of the large number of student experiments that were conducted in that facility. At the same time, a steam heating system was installed, and the main building was completely equipped with a sprinkler system for fire protection. In 1937, the "river laboratory" was constructed (see Figure 19). This facility was surrounded on three sides by windows that provided adequate light for

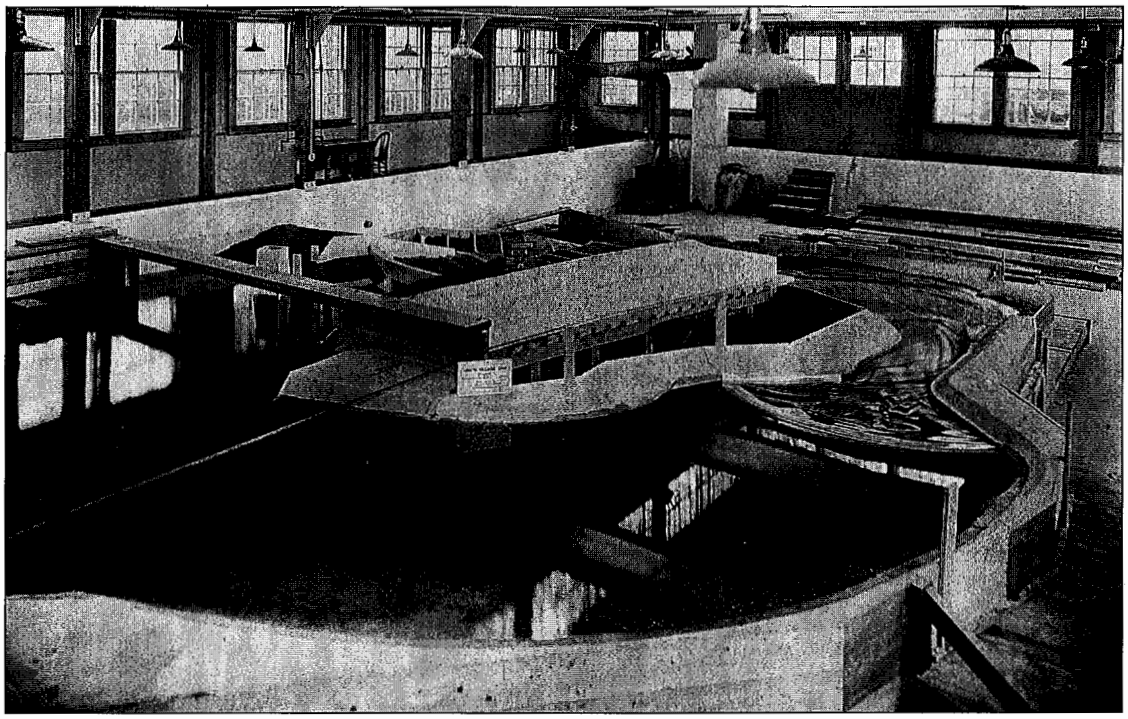


FIGURE 19. The river model laboratory.

photographing models. In addition, this heated facility allowed fairly large model studies to be performed during winter.

In 1930, the WPI trustees voted to expand AHL by acquiring the Fairbanks farm that adjoined the laboratory property. Expansion was required because the existing facilities were so cramped with models and equipment that room for student use was almost non-existent. The farm's barn was used to store "idle equipment," while the farmhouse was converted to drawing rooms and dining facilities for the Civil Engineering students who held surveying classes at the laboratory during their fall practice. The students lived in tents in the fields surrounding the farmhouse.

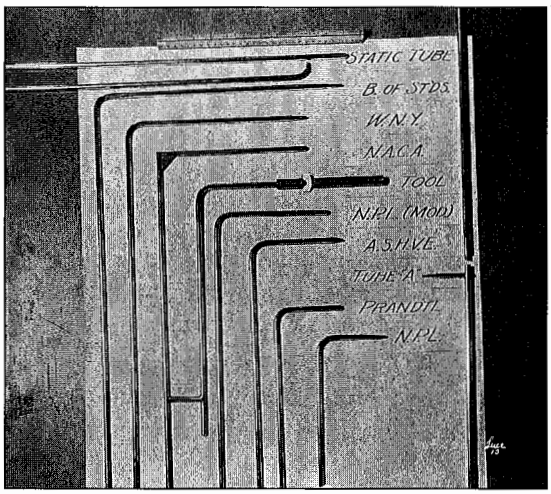


FIGURE 20. Pitot tubes studied by Hubbard.

Alden lab staff also provided instruction for WPI students. Teaching of all hydraulic courses was assigned to the laboratory and was carried out in this period by Allen, Hubbard and Hooper. During one half of the first semester, a senior laboratory was conducted three afternoons a week, and there was also an all-day waterwheel test each week. During the second semester, advanced hydraulic laboratory and thesis work represented the students' activities. During the summer months, WPI sponsored a program called Techniquest, which was designed to show high school students various aspects of engineering studies. Part of this program was staged at the laboratory where Allen delighted in showing the prospective WPI students the models and various projects at the facility. One of the highlights of Techniquest was

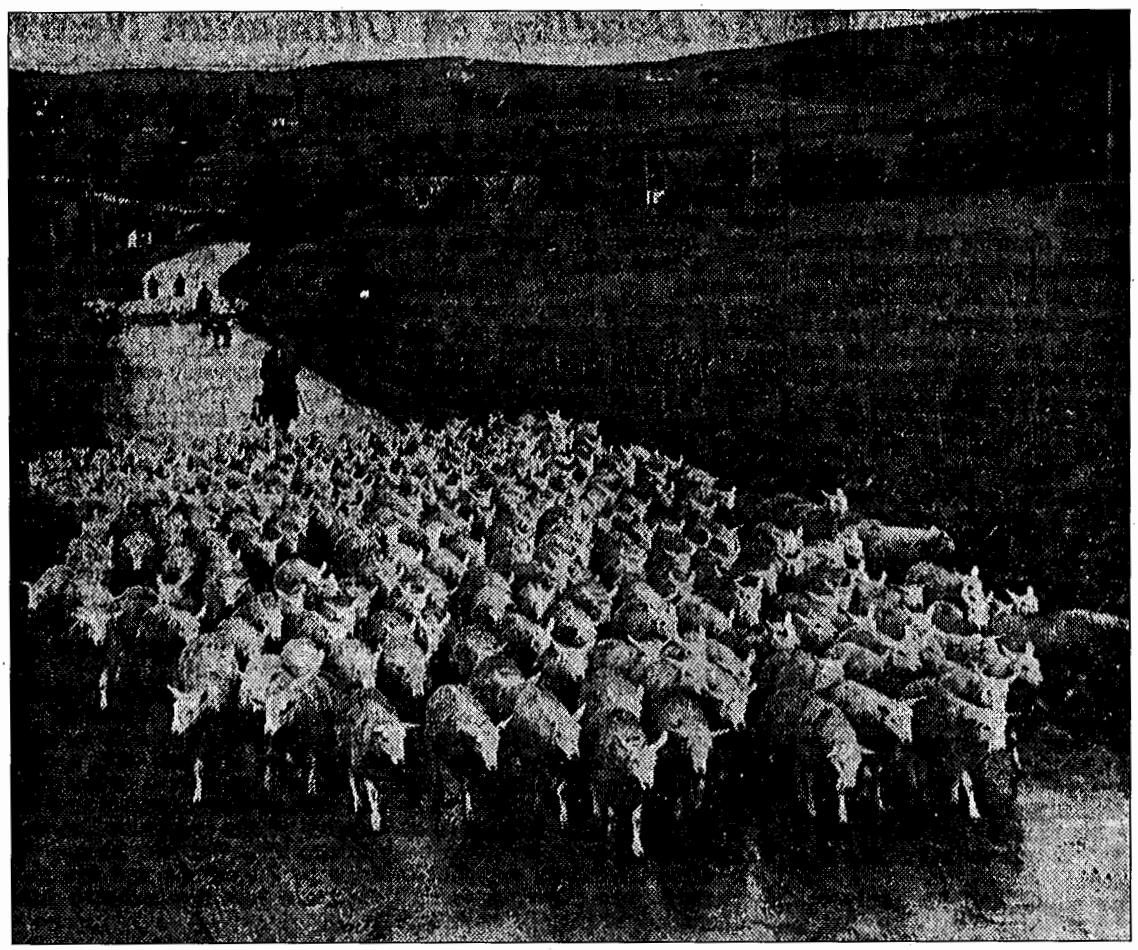


FIGURE 21. Allen's sheep nibbling on grass example of velocity profile.

a picnic at the Fairbanks farm. Allen's teaching style was much appreciated by students because of all the stories he told to illustrate significant points.

Applied Research

When Alden Hydraulic Laboratory staff were not busy with student or project responsibilities, they engaged in research of their choice. Hooper was involved with investigating pressure measuring errors due to variations in piezometer features. He published his findings with Allen in 1932 in an ASME paper entitled "Piezometer Investigation."[20] To this day, it is used as a reference in this area. Hubbard concentrated on examining errors in using pitot tubes. His paper, "Investigation of Errors of Pitot Tubes," was also published by ASME and explained a variety of ways by which pitot tube measurements could be in error.[21] Many pitot tubes were tested (see Figure 20).

Allen continued with his flow measurement studies. In 1934, he published "How Water Flows in a Pipe Line."[22] His discussion of turbulence using the analogy of sheep nibbling on grass is classic (see Figure 21). (In 1947, it became a part of a laboratory Christmas card.)

The 1930s were enjoyable for the laboratory staff, and provided full-time employment in difficult economic times. However, there were some anxious moments. The laboratory had some peculiar financial arrangements with WPI, some of which included getting paid only after a project was completed. This practice occasionally was a hardship for the staff so Allen often paid them from his own pocket and was reimbursed when the funds came through.

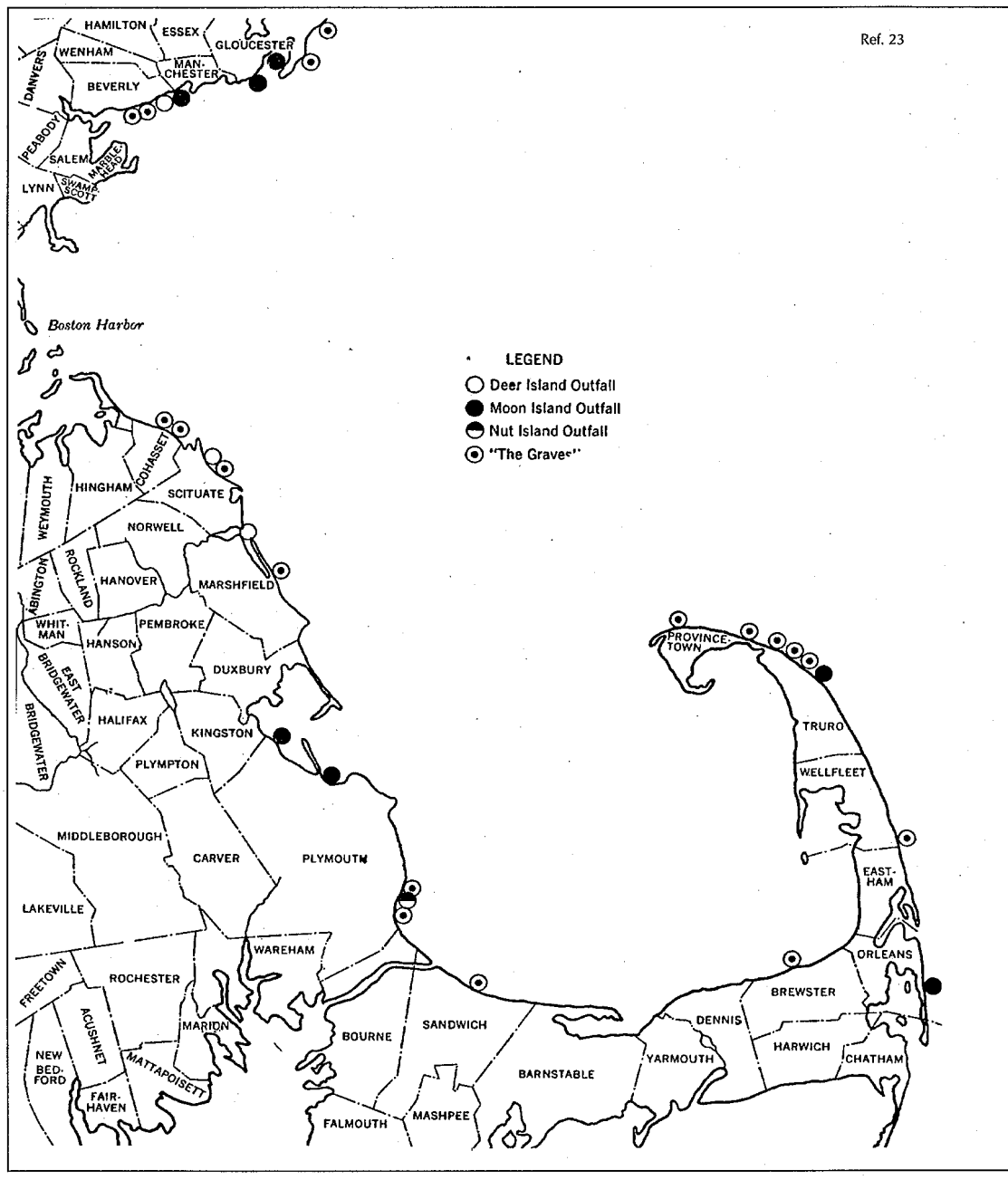


FIGURE 22. Pickup points for float chips dropped at various points in Boston Harbor.

Boston Harbor

At the end of this period, one local concern was the cleanup of Boston Harbor. Prior to 1940, rather comprehensive studies had been performed to determine harbor flow patterns and any changes that resulted with the dredging, filling and construction of Logan Airport. Pollution had not yet depleted the oxygen in tidewaters sufficiently to cause an odor nuisance, but there had been increasing aesthetic demands because of sleek waters and beach pollution. Flow tests in 1935 and 1936 were used to follow discharged material. Surface and deep submergence floats along with chips were used to get current patterns. Fig-

ure 22 indicates the distant locations where some surface chips were found. Various solutions were promised, and experimental treatment plants were established on Deer Island. These plants consisted of fine screens, a settling tank and a pre-aeration tank to aid grease removal. Other experiments were conducted to remove suspended solids. The sanitary concept of the time was to determine the capacity of the waters in which the effluent was to be discharged in order to replenish the oxygen that would be consumed by the bio-chemical purification processes.

An outfall sewer (tunnel) having multiple outlets discharging in the Boston's outer harbor near The Graves was considered but eliminated because of cost. Furthermore, based on float tests, engineers realized that it would still be necessary to remove grease and floating matter. The float chips (patterns) indicated that some flow at The Graves would return to the harbor and other coastal points, which would be objectionable to beach activities.

The New Deal & Electric Power Industry

As the Great Depression deepened to its low point in 1932 and 1933, the people of the United States focused their attention on a new president. Hoover was not able to rally the country or make the changes necessary to get the economy going. The election brought Franklin Delano Roosevelt into office and fundamental changes came to all aspects of US life. Industries including banking, securities, electric power and others were altered. Unions, work place job security and retirement were also affected.

Roosevelt was elected in November 1932, carrying all but six states, four in New England. During the lame-duck period that extended into March (in accordance with the Constitution), the conditions in the country deteriorated since Hoover and Roosevelt were unable to come to any working agreement. (The 20th Amendment to the Constitution was speedily proposed by Congress in March 1932 as a direct result of the experience with the Hoover-Roosevelt transition. That amendment, which changed the date the new President assumed office to January, was ratified by February 1933. Prohibition also ended in 1933.)

The country had to withstand two serious shocks. As the lame-duck period was ending, the nation's banking system was in a state of collapse. Panicky citizens were holding cash to the point that some cities were printing script for local use. Also, about two weeks prior to Roosevelt's assuming the office on March 4, an assassin's bullet killed Chicago's mayor Anton J. Cermak and just missed the President-elect.

In his first 100 days in office, Roosevelt brought about the New Deal. The New Deal legislation brought Social Security, the Works Projects Administration (WPA), the National Industry Recovery Act (NRA), the Tennessee Valley Authority (TVA) Act of 1933, and others. The most far-reaching in terms of the electric power industry was the Security-Exchange Act in 1934 and the Public Utility Holding Company Act in 1935. These two acts set up numerous financial safeguards, and passed a "death sentence" against all far-flung utility holding companies by 1937, except those servicing a "geographically and economically integrated system."

The TVA Act had several purposes, one of which was fostering competition and general rate reductions. The TVA was to serve as a yardstick by which to measure the cost of private power. Some utilities had regularly lowered rates. In fairness, a cost comparison between a public and private enterprise can be very difficult because of allocations for various activities such as flood control, navigation, soil rehabilitation and finance costs. However, whether due to the Depression, efficiency in scale and design, competition or other factors, rates and profits in the utility industry did go down during this period throughout the country. The average residential rate for the whole country declined from 5.5 cents per kilowatt hour in 1933 to 3.67 cents in 1942. The price of TVA power was about 3 cents per kilowatt hour.

Ongoing power supply costs also decreased. In 1934, Barrows noted that "steam plants at public utility plants now average only 1.5 pounds of coal per kilowatt hour and large plants at tidewater can produce power at 0.75 cent per kilowatt hour and less."[24] By comparison, in 1919, 3.2 pounds of coal was consumed per kilowatt hour.

While the price of power charged to the consumer in terms of production cost may or may not have been excessive, the simple fact is that most people (if able and connected to a system) were more than willing to pay the price for the convenience and improvement in life. Industrial usage dropped, but residential use increased substantially. Without increased residential use, many utilities would have failed because towns and cities were not paying their bills. One of the ingredients that allowed some utilities to meet their revenue needs was the greatly increased load due to refrigeration. In the system serving greater Philadelphia approximately half of the residential customers discarded their ice boxes between 1933 and 1938.[25] The average yearly residential power usage increased from 499 kilowatt hours in 1929 to 959 kilowatt hours by 1938.

Roosevelt saw the TVA as a multi-purpose regional planning agency for a seven-state poverty stricken area that had been abandoned by private utilities. Less than 2 percent of the area was electrified. He envisioned that products made in the region would be transported out by the navigation system and, thereby, the entire area would become self-supporting.

The charter for TVA promoted "flood control, navigation, electric power production, power distribution, proper use of land and resources, and the economic and social well being of the people." Even though Roosevelt criticized the private power industry, the TVA was created as a corporation vested with the power of government but operating with the flexibility and initiative of private enterprise.

Private utilities saw public power agencies as a real threat to survival and tried to block parts of the New Deal in the federal courts. These cases lasted for years, and the Supreme Court would generally rule in Roosevelt's favor on these matters.

The private electric power industry was undergoing great turmoil. With the Supreme Court's ruling upholding the Public Utility Holding Company Act of 1935, as of December 1, 1938, all holding companies were required to submit proposals for the integration and simplification of their properties.

Another threat to the private utilities was the Rural Electrification Administration (REA), which was started in 1935 because 90 percent of farms did not have electric power. In some areas, private utilities quickly responded by wiring large areas and bringing new customers on-line. These customers added needed revenues to hard-pressed utilities.

In the first 100 days of the New Deal, numerous reforms and programs in banking, finance and public works were started. Programs in public works were designed to put people back to work. One of the most far reaching was the Works Projects Administration (WPA) under Harold L. Ickes, Secretary of the Interior. The purpose of the WPA was to stimulate heavy industry via the authorization and construction of large public works. One WPA project, the Boulder (Hoover) Dam was completed two and a half years ahead of schedule with work going on around the clock. Bridges, military airports, hospitals, water supply works, dams, canals, sewers, flood control and irrigation projects, wind tunnels for plan design, housing, and some naval warships (later limited by Congress) were constructed. WPA built parts of the TVA, Grand Coulee Dam, Bonneville Dam on the Columbia River, Fort Peck Dam, Boulder Dam and other projects.

The success of the TVA gave rise to authorization of more limited river development projects not having the completely integrated regional planning. In 1937, "projects" were authorized for various regions including the Arkansas River Valley and the Lower Colorado, Ohio, Missouri and Red rivers. In hindsight, knowing the tremendous power needs in World War II and the special role of the TVA region, for many reasons the United States was fortunate that the TVA was created. In addition, the TVA would eventually grow to be the largest generator of electric power in the United States.

Prelude to Another War

At the end of the 1930s, the world was in chaos with Hitler conquering one country after another in Europe and the Japanese doing the same in Asia. When the United States went to war in December 1941, the Alden Hydraulic Laboratory was nearly a half-century old as it embarked on new work related to helping the United States defeat the aggressive forces of the Axis.

In 1939, the US government initiated a program that would use science to aid military efforts should the country enter the war. Aside from the Manhattan Project to develop the atom bomb, a large part of the funding for military developments was spent to finance experimental equipment at existing government facilities, such as the Naval Ordnance Test Station in China Lake, California. Some of the funding was made available to universities and private institutions to conduct basic research for military applications. As part of the program, Columbia University was contracted by the Office of Scientific Research and Development to organize research at various institutions in the fields of medicine, nutrition and weapon development. In 1941, the Alden Hydraulic Laboratory was approached to perform preliminary feasibility tests in the field of hydroballistics. When approached by the US Navy, the laboratory agreed to help and signed an open-ended contract that lasted until 1975.

The electric utility industry, as all other industries in the United States, rallied during the 1930s and 1940s to meet the nation's electric energy needs. National defense spending, lend-lease to England and a general build-up started in about 1939. Finally, the Depression ended virtually everywhere in the United States during the beginning of World War II — when Poland was invaded on September 1, 1939, when France fell in June 1940 and when the Battle of Britain began.

Power Industry & Engineers Respond

The TVA system went through an enormous, rapid expansion. In 1940, four new dams were completed and another three more were under construction. By mid-1942, TVA had 12 dams in operation and a steam plant under construction. TVA employment reached 42,000. Records were set by construction forces working around the clock to build plants and produce energy for several critical needs.

In 1942, Oak Ridge, Tennessee, was adopted as the location for a mysterious undertaking that later would be called the Manhattan Project. The site was selected for its remoteness, generally low population, distance from the coast and potential bombing, and available power. Numerous specialized facilities were needed on a "crash" schedule and a city for 75,000 people had to be constructed. Critical manufacturing plants were located in adjacent valleys so that shielding was provided by the hills against premature explosions.

Three-fifths of the elemental phosphorous for incendiaries, smoke screens and other uses also came from the TVA. In addition, aluminum plants in the valley ran continuously.

Engineering and construction companies built all of the facilities needed to equip the miliary in its global needs — munitions plants, foundries, rubber plants, refineries, chemical plants, shipyards, naval bases, refineries, pipelines and other facilities.

Electric power systems underwent expansions and were faced with war-time shortages and allocations. The manufacture of civilian commodities ceased. Turbines necessary to meet power demand were allocated to the war effort, with naval carriers having first priority. In general, there were no electric power shortages, but instances of restrictions and brown outs (voltage reductions) did occur. In February 1942, Congress enacted wartime daylight savings to reduce the evening peak power demand. In the East, power restrictions were most severe around Christmas 1943 when decorations and lighting were discouraged. The war effort also taxed power plant workers and some strikes and labor unrest occurred. The Los Angeles municipal power system (the largest city owned public utility) was shut down in February 1944 due to workers protesting a small wage increase. Ten days later, the US Army took over the system.

Hydroballistics — Water Entry

By the beginning of World War II, the Alden Hydraulic Laboratory staff was beginning to change. Lawrence C. Neale, a WPI graduate and a future laboratory director, was hired in 1940. Clyde Hubbard left in 1941 to work with a well known consultant, Joel B. Justin, co-author of a popular book on hydropower, first published in 1927.[26] In 1942, Hubbard was commissioned in the US Naval Reserve and, due to his experience at AHL, was immediately assigned as the officer in charge of the design and construction of the circulating water channel

for the David Taylor Model Basin. As the war effort was stepped up, other WPI personnel, especially in the area of electrical engineering, were added to the laboratory staff.

During World War II, student work continued at AHL, and many of the students were part of the US Navy V-12 program. Private test and consulting work essentially came to a standstill during this period since all the staff were involved in the experimental Navy program. The open-ended Navy contract paid the laboratory $40,000 per year to quickly solve problems by testing. The contract enabled various agencies to obtain experimental results without going through a lot of red tape. The work at the Alden Hydraulic Laboratory was generally related to thewater entry of ballistic weapons. At the beginning of the war, many torpedoes and bombs entering water from the air experienced instabilities of their underwater trajectories and were, therefore, ineffective. Studies in this area became an AHL specialty. Testing in this area was developed from scratch since none of the facilities or equipment needed to perform this work existed when the original Navy contract was signed.

Most of the work in water entry involved the use of high-speed photography. At the start of the contract, Leslie Hooper, the Alden Hydraulic Laboratory Director, had contacted Victor Sepavitch, an electrical engineer at a textile factory in Worcester, Massachusetts. Previously, Sepavitch had worked at the laboratory when he was a graduate student at WPI. Hooper knew that Sepavitch was studying the motion of looms using high-speed photography and strobe lights — a method recently developed by Harold Edgerton at MIT. This technique, Hooper thought, would be perfect to study ballistic missile behavior during water entry and during the initial stages of air cavity formation behind the projectile in the water.

The first photographs were stop-action, strobe-lighted pictures taken in the 50,000-pound weigh tank in the main AHL building in late fall of 1941. These photographs were a success, but the tank did not lend itself well to photographing the phenomenon since the tank did not have glass sides that permitted easy viewing of the projectile models. In the winter, Hooper consulted with both Sepavitch and Edgerton (who was later known as "Papa Flash") regarding high-speed photography and methods that were being implemented and considered for the Navy work. At this time Japan bombed Pearl Harbor, and the United States formally entered the war. The military research programs were accelerated, and the laboratory became more active in its studies of the water entry of ballistic projectiles.

In 1942, Hooper, Neale, Professor Hobart Newell (from the WPI Electrical Engineering Department) and some technicians began water entry work at the MIT swimming pool. Edgerton had offered the use of his high-speed strobe lights, 16-mm movie cameras, technicians and facilities for developing and processing the film. It was felt that doing the work at MIT was easier because the photographic equipment was readily available and the pool had a glass port on its side that could be used to photograph water entry. However, the MIT pool was required during the day for the Navy V-12 program. Alden Hydraulic Laboratory staff were only allowed to use the facility from 10:30 p.m. to 6:00 a.m. and had to erect and dismantle their equipment every day. Travel to Boston every day, plus the odd hours and the added work of daily equipment set-up and removal, took its toll on everyone concerned. Edgerton then suggested that he could lend the laboratory a spare 16-mm movie camera and could rig a few lights that could be used in Holden.

Testing began at AHL with the equipment loaned by Edgerton, augmented by Sepavitch's strobe lights. Edgerton had supplied Newell with circuit diagrams that he could use to maintain the system. Newell improved on many of the circuits, increasing the efficiency of taking photographs. The equipment was initially set up in the basement of the main building, using a 3-foot deep glass-sided flume to observe the water entry process. It was soon apparent that this facility needed improvement because of its limited size.

Hooper investigated other nearby facilities and obtained permission to use the US Navy's submarine escape training tower in New London, Connecticut. This facility was a 50-foot high circular tower with glass-sided ports. After some initial work, it was decided that the

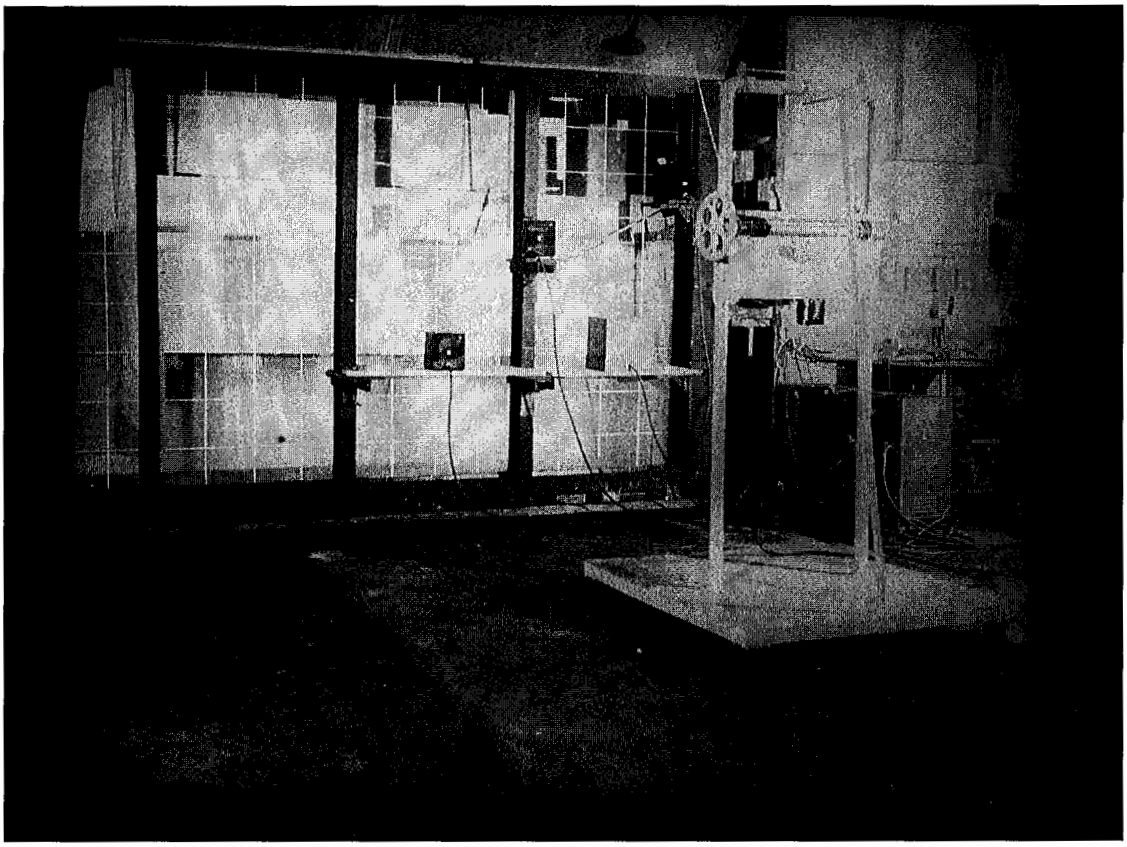


FIGURE 23. Glass-sided tank for entry studies.

same problems encountered at MIT were present in New London; therefore, testing there was discontinued.

In the late spring of 1942, plans were made at Alden Hydraulic Laboratory to erect a glass-sided tank (see Figure 23) for entry studies in the relatively new "river laboratory." Due to security, and for photographic reasons, all the windows that had been installed in this laboratory were covered. The "river laboratory" only had a brief existence. The tank that was installed was 9 feet high by 5 feet wide by 16 feet long. It consisted of a steel frame with four 4-foot by 9-foot by 1-inch thick annealed glass plates in the front and redwood for the rest of the tank. (In the late 1950s, the wooden portion was replaced by steel plate.)

The equipment necessary to conduct the studies consisted of a camera, strobe lights, model launchers and electronic circuitry. The 16-mm camera that had been borrowed from Edgerton had the disadvantage of being slow and its film was small, which reduced sharpness when enlarged prints were made. Hooper designed a new camera and had it built by Carl Anderson, a laboratory machinist who had come to AHL from WPI. The camera consisted of a lens and a drive mechanism of two wheels with slightly raised rims, and used 70-mm unperforated film in a roll. One wheel of the drive mechanism was driven by an electric motor and the other wheel was connected to an electric solenoid. The film was threaded between the rollers and, upon actuation of the solenoid, the rolls pinched the edge of the film and drove the film past the lens into a receiving container. The strobe lights acted as the shutter while the film was in motion. The problem with the camera was that it tore the film from the roll during the initial film acceleration, but this problem was solved by the insight of a student working at the laboratory after returning from a Christmas party. He suggested that the film be removed from the roll and configured like Christmas

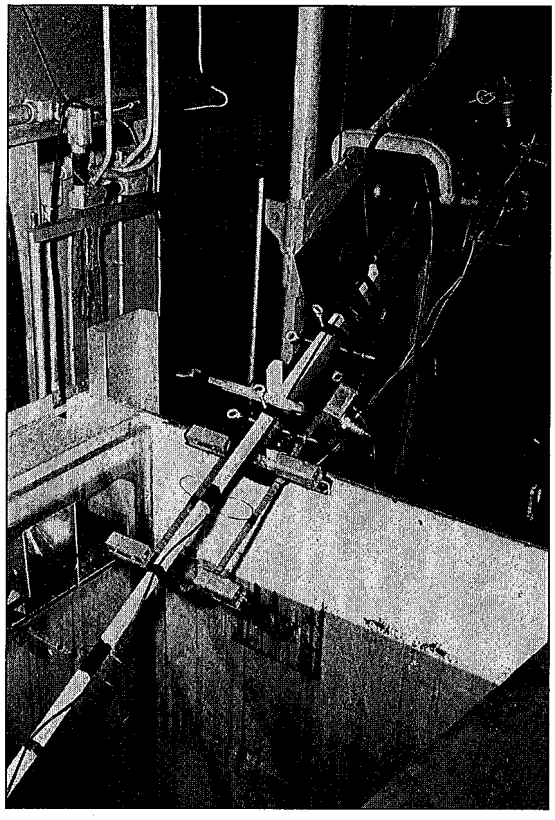


FIGURE 24. Pneumatic blow gun with barrel, velocity indicator apparatus and antenna.

candy ribbon in a separate container. By so doing, only a small section of the film would be accelerated at a time. The scheme was tried and it worked like a charm.

The strobe lights used were originally from Edgerton. Newell rigged new lights that were extremely bright for very short durations and required huge banks of capacitors. To keep moisture from harming the capacitors, they were placed in a separate room adjacent to the river laboratory.

Initial water entry studies involved dropping the models in the tank. This technique sufficed at first, but it lacked the flexibility required for a thorough evaluation of critical phenomena. To study vertical as well as angled water entries, a pneumatic-type blow gun (see Figure 24) was designed by Hooper, assisted by one of his WPI classmates, Ray Tower, a chief engineer and owner of a paper box plant. The gun used carbon dioxide gas as a propellant and operated as a differential piston. The original gun had problems with the escaping gas creating a disturbance on the water surface. This problem was quickly remedied by a spool attached by a rod to the piston that stopped in guides just before the model was released.

The models used in this gun were normally 1.25 inches in diameter and were quite sophisticated. They were usually constructed of more than one kind of material. Besides modeling the shape and total weight, the center of gravity and the moment of inertia needed to be properly scaled. Numerous nose and tail pieces were needed as replacements for those pieces that were damaged during errant trajectories. Most pieces of the models were machined on equipment that was normally used to work on watches. The threaded parts of the models fit so perfectly that it was often impossible to tell where one piece ended and the other began.

For proper operation, the principal requirement was a system that synchronized all components so that they would work exactly when required. Professor George Stannard of the WPI's Electrical Engineering Department designed and constructed a sophisticated timing system, consisting of 76 electronic tubes, fans, relays, variable resistors and motors. This system enabled the operator to automatically initiate a sequence of events, starting with the firing of the guns, followed by the start of the camera, with the strobe lights starting to flash just prior to the model ordnance entering the water (see Figure 25) and reflected, as sometimes occurred. The number of strobe pulses could also be scheduled on the timing system, varying anywhere from one to one thousand pulses per second. The resulting cavities were compared (see Figure 26 on page 40). The total system was so unique that engineers from a major camera and film manufacturer visited to inspect the facility.

To supplement information from the main tank, a 3-foot diameter vertical drop tank was erected in the north corner of the main laboratory. This tank permitted no access with which to photograph water entry; instead it was used to confirm some values of velocity and acceleration observed in the large tank. Ordnance models tested in this smaller facility were

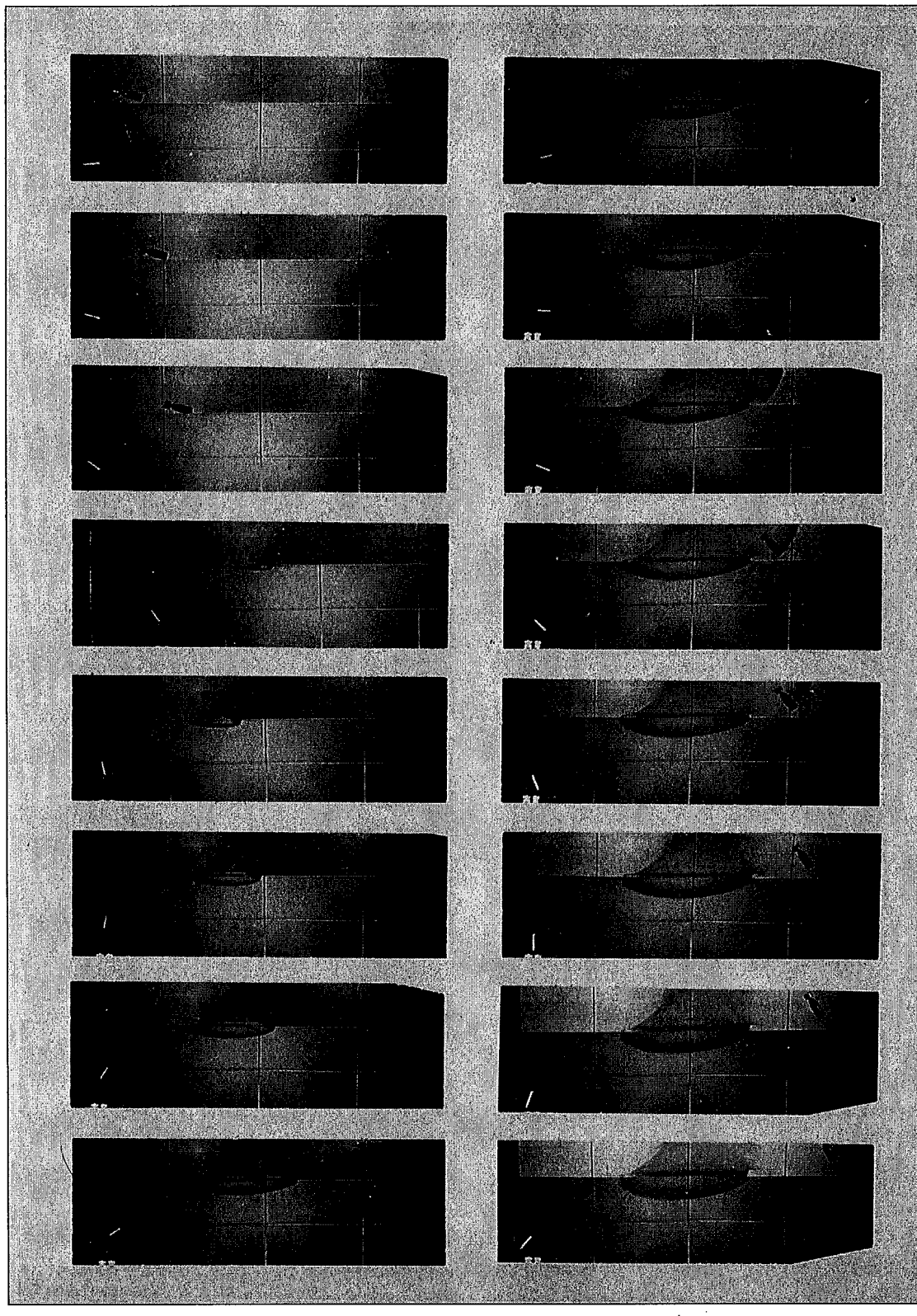


FIGURE 25. A projectile reflecting from the water surface.

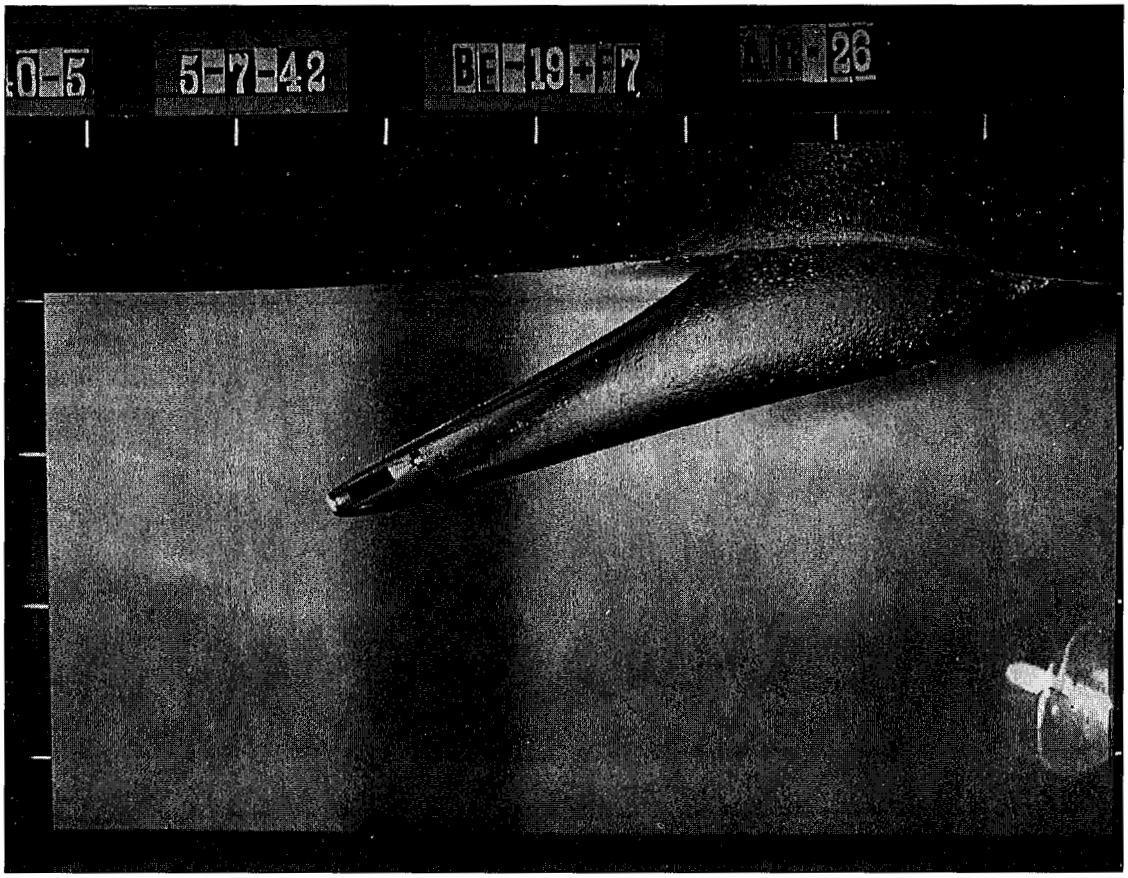


FIGURE 26. The cavity formed at entry.

equipped with a magnet, and a system of coils was mounted vertically every 2 feet in the tank. The coils sensed the model as it went by and sent a signal to a chart recorder. From these tests, accurate time-distance diagrams were obtained and used to plot the velocity and acceleration of the models. In the main tank, this information was obtained by hand-plotting model position frame by frame using the photographic film.

In 1943, another test tank was added to the river laboratory. This tank was 4 feet high by 5 feet wide by 18 feet long and was used to study high-speed water entry. The gun used in this facility was a converted 50 caliber gun. Hand loading of the ammunition allowed obtaining the various entry velocities. Models were mounted in specially designed plastic sabots that would break and release the model prior to entry; a special container caught the sabot parts, preventing them from entering the water.

Among the ordnance types tested at the Alden Hydraulic Laboratory during World War II were depth charges, plunge bombs, jet-propelled devices, bullets and buoyant devices. Thirty-five different models of depth charges were studied. A few of these had better trajectories and faster depth penetration than those used by the Navy, but they were never produced for actual use.

The plunge bomb model was a device made using drawings obtained from the capture of a German experimental facility. Reports captured with the plans described the bomb as having excellent water entry characteristics. Because of this feature and since the bomb seemed to be simple to manufacture, the Navy wanted the shape tested. All of the tests at AHL failed to confirm the claims. In fact, most of the models displayed extremely unstable behavior at water entry with a tendency to tumble after entry. (It was not until after the war that the

truth was learned. At a scientific meeting in Europe, Neale met one of the scientists who had worked at the German facility. When they discussed the project, the German scientist indicated that they were allowed to live at the facility with their families provided they were productive. The facility was considered reasonably safe, and scientists did their utmost to remain there. When it became evident that Germany was losing the war, some scientists falsified data and reports to prevent their removal and that of their families to more dangerous zones.)

The buoyant devices were to be mounted near the bottom of a harbor and released when an enemy ship moved overhead. The models used in this study were made of balsa wood and were released by a person sitting on the bottom of the tank. That person was equipped with a homemade diving helmet fabricated from a pail, a small compressor and a garden hose.

One project worked on at AHL was a type of bomb that the British planes used to sink the German battleship Tirpitz off the Norwegian coast in 1944. The devices were dropped from high altitudes. Upon entering the water, they followed a curved path under the ship and exploded near the unarmored underside, blowing a large hole in the hull.

The rotating boom at the laboratory was also used during World War II for studies on ship logs (a device for measuring speed), hydrophones and paravanes. Two different types of ship logs were studied. The first was a propeller-type log, where the rotation of the propeller was an indication of the ship speed. Besides calibrating ship logs, a 500-hour endurance test was conducted on two propeller-type logs. The officer in charge at the US Navy Bureau of Ships indicated that the tests were done at AHL because it had the only facility in this country capable of making continuous runs. The second type of log was a pressure-sensitive plate where the force on the plate indicated ship speed. The hydrophone studies involved minimizing noise from the turbulent flow around the noise pick-up through use of various shields. Paravanes were devices used to sweep moored mines. They resembled inverted wings and were towed out the side of a ship at the end of a wire. When the ship snagged a mine, it slid down the wire and detonated at the paravane.

Hooper was also active during the war, traveling to numerous Navy facilities in Washington DC, the California Institute of Technology, the China Lake Naval Test Facility in California and the Naval Underwater Test Facility in New London, Connecticut. His expertise was sought after in various aspects of fluid mechanics. In some facilities, he was part of a project team studying ballistic or flow problems. On numerous occasions he left Worcester on one night in a sleeper car, arrived in Washington early the next morning, attended a meeting all day, and returned that evening on another sleeper car to Worcester.

NOTE — *This article is the second in a multi-part history of the Alden Research Laboratory. Part I appeared in the Vol. 14, No. 1, Spring/Summer 1999 issue.*

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REFERENCES

1. Johnson, R.D., "The Surge Tank in Water Power Plants: A Device for Aid in Speed Regulation and Pressure Relief in Water Powers With Long Pressure Pipes and High Velocities," *Transactions of the ASME*, Vol. 30, 1908.

2. Barrows, H.K., *Hydroelectric Engineering*, First Edition, 1927.

3. Rogers, F.H., "Hydro-Generation of Energy," *Transactions of the ASCE*, Vol. 104, 1939.

4. Groat, B.F., "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators," *Transactions of the ASCE*, Vol. LXXX, 1916.

5. Allen, C.M., & Taylor, E.A., "The Salt Velocity Method of Water Measurement," Vol. 45, 1923.

6. Gibson, N.R., "The Gibson Method and Apparatus for Measuring the Flow of Water in Closed Conduits," *Transactions of the ASME*, Vol. 45, 1923.

7. Groat, B.F., "Ice Diversion, Hydraulic Models, and Hydraulic Similarity," *Transactions of the ASCE*, Vol. LXXXII, 1918.

8. Schneider, G.R., "History and Future of Flood Control," *Transactions of the ASCE*, Vol. CT, 1953.

9. Freeman, J.R., "Address at the Annual Convention, Hotel Wentworth, Near Portsmouth, New Hampshire, June 21, 1922," *Transactions of the ASCE*, Vol. LXXXV, 1922.

10. Freeman, J.R., "The Need of a National Hydraulic Laboratory for the Solution of River Problems, etc.," *Transactions of the ASCE*, Vol. LXXXVII, 1924.

11. Freeman, J.R., *Hydraulic Laboratory Practice*, "Appendix 11," Revised Version, American Society of Mechanical Engineers, 1929.

12. Kurtz, F., "The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam, and Hydraulic Tests on Working Models," *Transactions of the ASCE*, Vol. 88, 1925.

13. Reeve, L.L.N., "Erosion Below Conowingo Dam Proves Value of Model Tests," *Engineering News Record*, Vol. 108, No. 4, January 28, 1932.

14. Allen, C.M., & Winter, I.A., "Comparative Tests on Experimental Draft Tubes," *Transactions of the ASCE*, Vol. 87, 1924.

15. Kennison, K.R., "Ware River Intake Shaft and Diversion Works," *Civil Engineering*, August 1934.

16. Knowlton, W.T., "The Sewage Disposal Problem of Los Angeles, California," *Transactions of the ASCE*, Vol. 29, 1928.

17. Rawn, A.M., Bowerman, F.R., & Brooks, N.H., "Diffusers for Disposal of Sewage in Sea Water," *Journal of the Sanitary Engineering Division, ASCE*, Vol. 86, No. SA2, March 1960.

18. Allen, C.M., "Efficiency Tests on Units 1, 2, and 4, Rumford Falls, Rumford Falls Power Company, Rumford, Maine," August 1926.

19. Allen, C.M., correspondence to Chicago Bridge and Iron Works, June 21, 1930.

20. Allen, C.M., & Hooper, L.J., "Piezometer Investigation," Transactions of the ASME, Vol. 54, 1932.

21. Hubbard, C.W., "Investigation of Errors of Pitot Tubes," *Transactions of the ASME*, Vol. 61, 1939.

22. Allen, C.M., "How Water Flows in a Pipe Line," *Mechanical Engineering, Journal of the ASME*, Vol. 56, No. 2, February 1934.

23. Weston, A.D., & Edwards, G.P., "Pollution of Boston Harbor," *Transactions of the ASCE*, Vol. 105, 1940.

24. Barrows, H.K., *Hydroelectric Engineering*, Second Edition, 1934.

25. Wecter, D., *The History of American Life, Vol. XIII, The Age of the Great Depression, 1929-1941*, Macmillan Co., 1948.

26. Creager, W.P., & Justin, J.D., *Hydroelectric Handbook*, John Wiley & Sons, New York, First Edition, 1927.

Innovative Design for Tunnel & Excavation Support for the CA/T I-90/I-93 Interchange

The innovative use of soil-cement stabilization to form berms and deep permanent buttresses and foundations for tunnels provides a way to tackle the difficult ground conditions at this site.

JAMES R. LAMBRECHTS, PAUL A. ROY & STEPHEN TAYLOR

Two major innovations in US construction practice are part of the solution to a complex and difficult portion of multi-billion dollar Central Artery/Tunnel (CA/T) Project. Deep soil mixing and tunnel jacking, both extensions of technologies used overseas, are being used for construction in very weak soil conditions in the highly congested Fort Point Channel area.

The rebuilding of a major urban interchange between two interstate highways is an especially challenging design and construction undertaking. For the CA/T Project in Boston, the task of extending the Massachusetts Turnpike (I-90) and relocating I-93 northbound (NB) becomes quite formidable when existing conditions include an active railroad yard, a navigable body of water, old buried wharves and foundations, and deep soft ground (more than 35 meters [115 feet] of soft Boston blue clay and channel bottom mud). To extend I-90, three multi-lane tunnels must pass beneath five to nine tracks of Boston's South Station railroad terminal. Two other tunnels that will merge with I-90 skirt along the edge of the Fort Point Channel and must support columns of a new overhead viaduct.

It is difficult to imagine a site presenting a more onerous combination of challenges, first for the designer and then for the contractor. The final design focused on applying two techniques (deep soil mixing and tunnel jacking) in significant innovations for North American construction. The extent of these applications is shown on Figure 1. Although both methods have proven successful in overseas uses, these innovations are being used to magnitudes

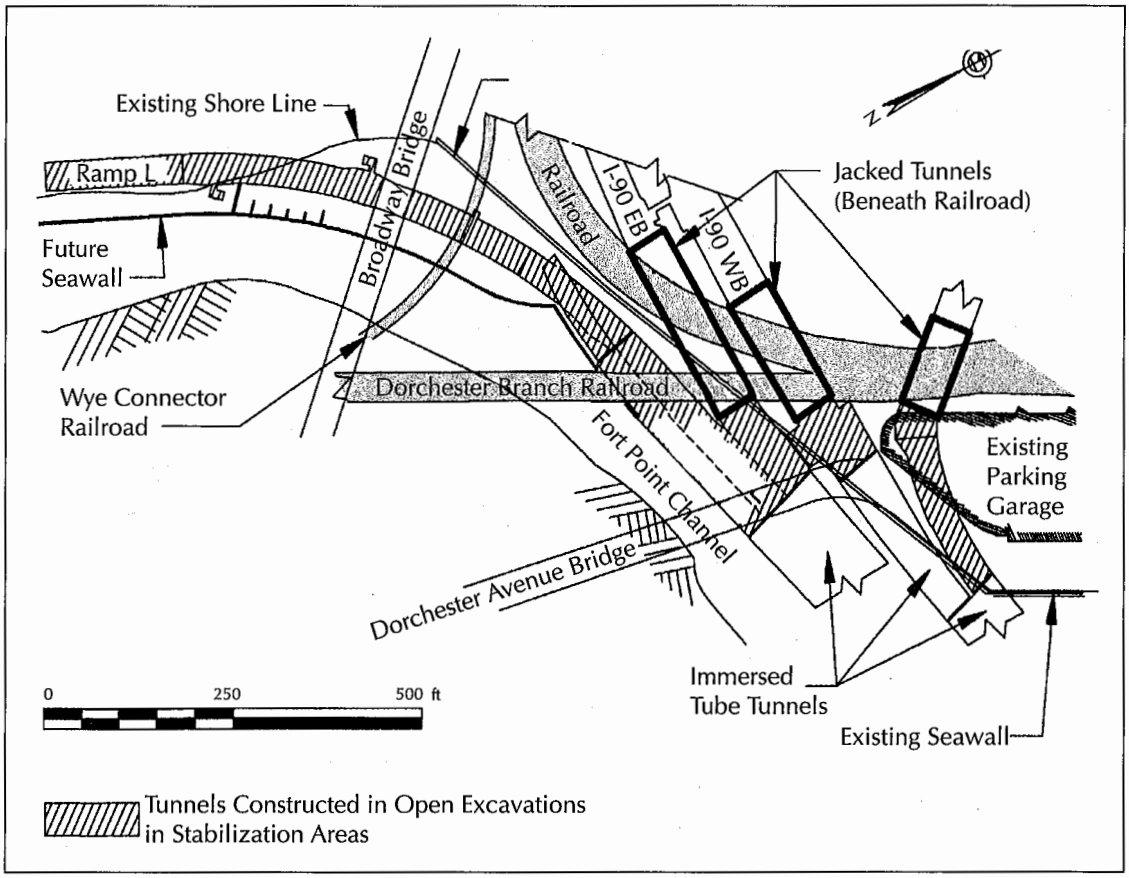

FIGURE 1. Site plan showing tunnels with soil-cement stabilization and tunnels to be jacked.

never before attempted, and in manners quite unlike prior uses. Construction now underway is applying these innovations to effectively overcome site challenges present at the I-90/I-93NB interchange.

Overview of Design Innovations

The first major innovation is the stabilization of the soft thick clay deposit beneath and adjacent to the Fort Point Channel by the processes of deep mixing and jet grouting to create more than 600,000 cubic meters (800,000 cubic yards) of soil-cement. In some areas of Fort Point Channel, a soil-cement berm was designed to buttress the tunnel against lateral loads, but where tunnels are deep, mixing was designed to penetrate more than 35 meters (115 feet) to create large foundation blocks and shear walls for deep foundation support. The construction operations necessary for installing the deep soil-cement in the crowded channel conditions required a preliminary stage of overwater shallow soil stabilization. The soil-cement buttress also was designed to provide temporary excavation support during construction of the tunnels.

The second major innovation developed by the section design engineers is the full-size tunnel jacking under busy railroad tracks of Boston's South Station Transportation Center. Three separate tunnels — one with two lanes and two with three lanes each, at depths of 11 to 18 meters (35 to 60 feet) below ground (or track) level — were designed to advance under active railroad tracks as full-sized, finished tunnel structures. The tunnel structures are first built in deep excavations adjacent to the tracks. These jacking or thrust pit excavations, which are themselves major construction undertakings, use deep concrete slurry walls that incorporate special design features. Using shield-tunneling methods, each tunnel is then ad-

vanced by incremental jacking through the ground under the tracks. The special ground stabilization provided in the design to stabilize ground before tunnel advance has been replaced by ground freezing.

Tunnel Conditions Requiring Innovative Approaches

The construction is extending I-90 eastward about 3.5 kilometers (2 miles) to Logan International Airport from its present terminus at I-93, and provides major connecting tunnels to link with a new I-93NB. At the east end of the rebuilt and expanded I-90/I-93NB interchange, the tunnels will link with the adjoining design section's immersed tube tunnels (ITTs) that will form the Fort Point Channel crossing.[1] The locations of major cut-and-cover and jacked tunnels being constructed in the area are shown on Figure 1. The tunnel outlines have been added to photographs of the preconstruction Fort Point Channel site in Figures 2 and 3 to illustrate both the constituents of the interchange and complex design considerations.

The location of the tunnels along the edge of Fort Point Channel impose large unbalanced lateral earth and water loads as a final design condition. Excavation basal heave and global stability factors of safety were determined to be unacceptably low. Also, the Ramp L tunnel south of the Wye Connector Railroad Bridge is a long continuous structure that must support column loads of the new I-93NB viaduct. Therefore, uniform foundation support and stiff lateral restraint was essential.

During the final design, the delineation of a large area of very soft Boston blue clay in the channel made it impractical to use the original preliminary concept design of a series of filled circular cellular cofferdams founded on existing soils. The particularly deep soft clay conditions between the Wye Connector Railroad Bridge and Dorchester Avenue Bridge (see locations in Figure 1) made it very expensive to attempt the use of conventional deep braced excavation support techniques. Furthermore, the required excavation depths yielded unacceptable low factors of safety against basal heave and global instability.

As an alternative solution, the designer proposed extending the earlier CA/T Project use

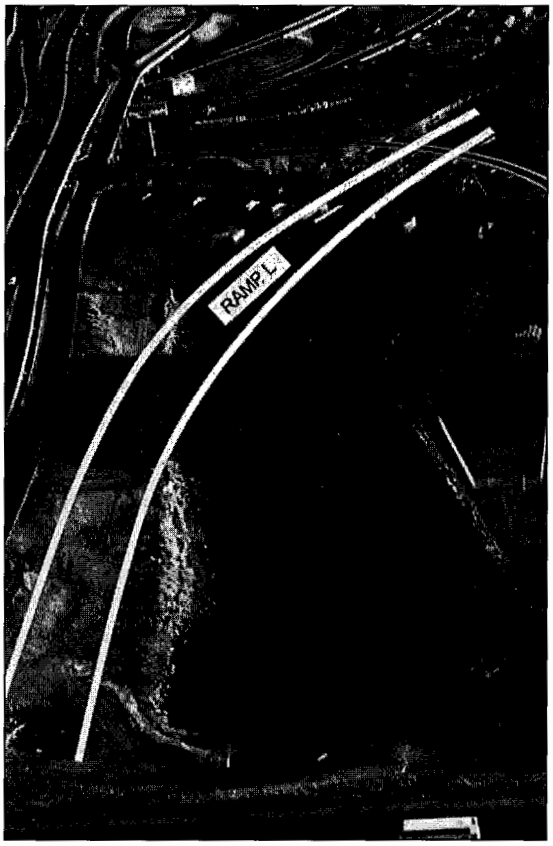

FIGURE 2. Site photo showing the tunnel location in preconstruction Fort Point Channel.

of deep soil mixing to a large area of the Fort Point Channel interchange.[2,3] The use of deep mixing to provide soil stabilization and create soil-cement buttresses as permanent support for the tunnels was an innovative extension of construction methods in Japan. One application used in Japan places precast concrete revetment structures on cement stabilized soft soils in marine areas and creates reclaimed land by filling (see Figure 4).[4,5]

The second innovation of installing jacked tunnels beneath the railroad tracks was proposed by the designer early during project final design. This method avoids the incremental relocating of the railroad tracks and piecemeal tunnel construction in deep excavations adjacent to active tracks that was the basis for the original conceptual design.[7,8]

The ability to jack full cross-section tunnels beneath the tracks (as an extension of previously proven construction technology from

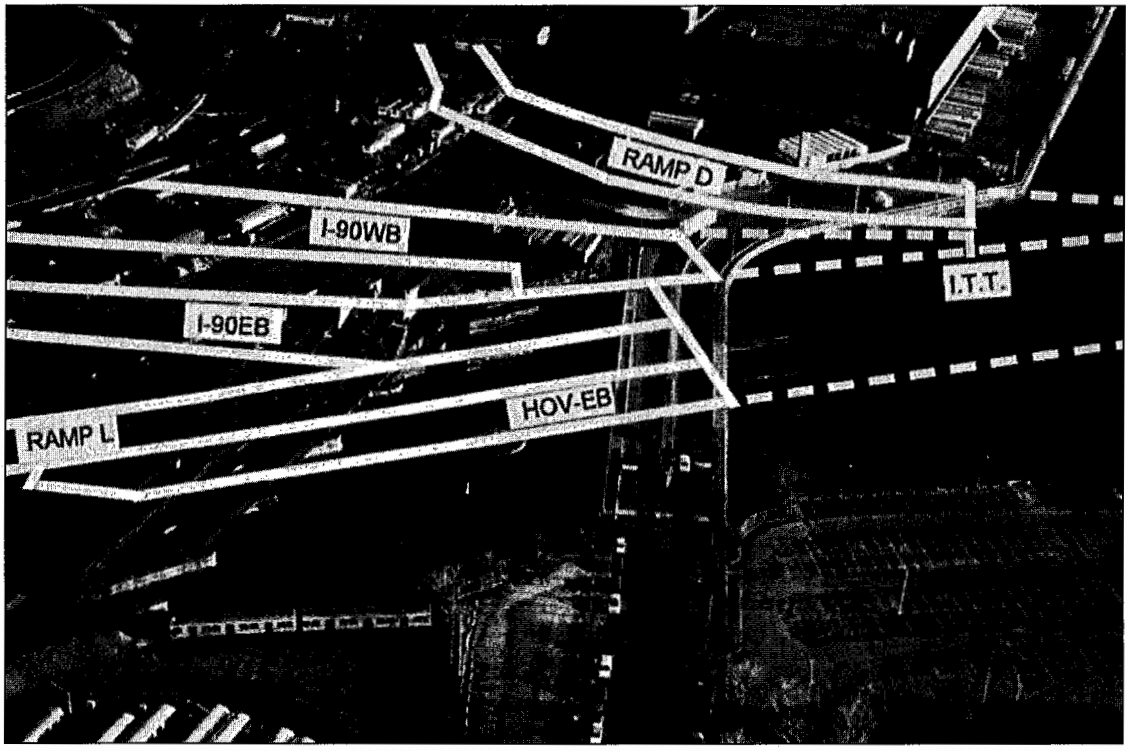

FIGURE 3. Site photo showing the tunnel to be constructed beneath the railroad.

Europe) is an important construction expedient that was favored by the railroad authorities. The tracks handle over 450 passenger train movements daily for both commuter rail and the Amtrak northeast corridor passenger service to New York and Washington, DC. Although tunnels of similar magnitude have been jacked in Germany, England and Japan, the subsurface mixed face conditions of soft clay and organic soils, and the high groundwater table make this effort particularly challenging, and an important innovation for construction in the United States.[9-11]

Site & Subsurface Conditions

The existing conditions in the Fort Point Channel are complicated and crowded as seen from Figure 1 and the photographs in Figures 2 and 3. Site area for the work activities is extremely limited, and this limitation on available space is made more challenging because, at the peak of construction activity, four different contractors must work simultaneously in contiguous and sometimes overlapping sites.

Typical stratigraphy for on-land areas of jacked tunnels is shown in two different tunnel profiles (see Figure 5). Beneath the channel, there is similar stratigraphy, but the surficial fill is not present. Greatly complicating the subsurface are the numerous buried structures, both foundations and old wharfs, which were erected on the site in the 1800s. Rock-filled timber cribs, granite block walls on wood piles, corduroy log roads and wharf platforms, and later relieving platforms were all used for wharfs. The design also expected that zones of dumped ballast stones and, perhaps, the remains of boats or ships would be encountered. The depressed trackway structure (circa 1900, with massive granite block and concrete walls and invert) lies in the line of the two jacked tunnels (see approximate location and configuration in Figure 5). The Fort Point Channel also was cluttered with four bridges, the remains of an earlier bridge and an extensive fender pile system. All of these structures had to be accounted for in the contractors' construction plans for the deep mixing and jacked tunneling.

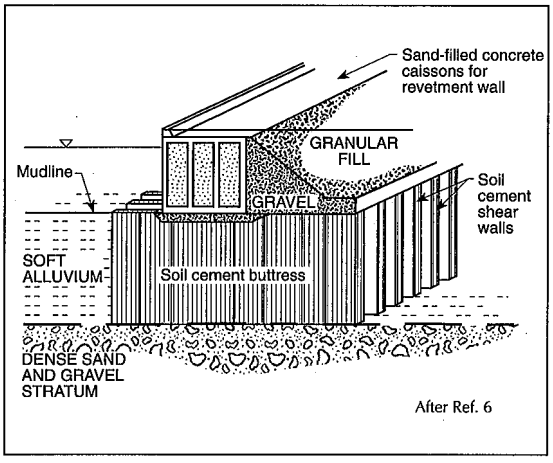

FIGURE 4. An example of the use of deep soil-cement buttress for revetment.

The soil profile throughout the I-90/I-93NB interchange project site is typical of the filled land areas of Boston, except the filling around marginal areas was much more random and variable than in the major land-side filled areas of Boston's Back Bay. The organic soil stratum is primarily the former tidal flat mud. On land, where this stratum was filled over, consolidation has resulted in some increase in shear strength, but within the channel it has very little strength. The primary soil stratum is the Boston blue clay, which is over 30 meters (95 feet) thick in some areas. Its shear strength decreases with depth to a point, and then increases, as shown in Figure 6. The profiles also indicate that the shear strength of the clay in the channel is substantially lower than on-land areas. These low shear strengths pose significant basal instability problems for deep excavations. Basal instability potential is being overcome by the application of deep soil mixing to create soil-cement to replace the soft clay. In some isolated areas, the organic soils and the clay are separated by a layer of fine sand and/or silty sand that is up to 1.5 meters (5 feet) thick. The borings indicate this sand occurs in the path of the Ramp D and I-90 westbound jacked tunnels.

A supplemental subsurface exploration program undertaken during final design included a substantial number of overwater test borings to retrieve samples of clay for consolidation testing and for unconsolidated-undrained tri-

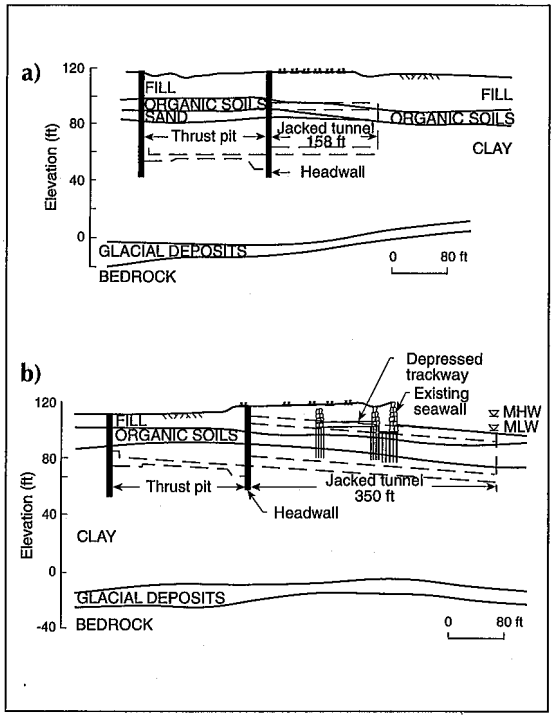

FIGURE 5. The soil profile a) along the Ramp D jacked tunnel, and b) along the I-90 eastbound jacked tunnel.

axial (UU) and direct-simple-shear (DSS) strength tests. In addition, special piezocone in-situ testing was performed beneath the railroad tracks because long train service outages for conventional test borings were not possible. An extensive series of consolidation tests were

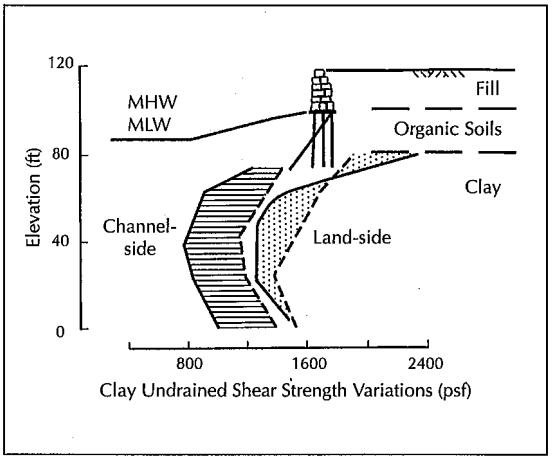

FIGURE 6. Ranges of clay stratum undrained shear strength profiles.

performed to apply the SHANSEP approach to assess profiles of existing clay stratum shear strength.[12]

Special small strain testing of triaxial samples was performed to determine the appropriate parameters for soil-structure interaction modeling of the excavation support walls of the large deep thrust pits, and ground stability at the face of the jacked tunnels. The small strain testing was conducted at the Massachusetts Institute of Technology (MIT) and included measurement of specimen lateral straining. To establish soil model parameters, each triaxial test was modeled using the program FLAC (Fast Lagrangian Analysis of Continua) imposing the stress conditions recorded in the laboratory tests, and program parameters were varied until predicted specimen behavior agreed with test results.[13] Further validation of the soil model was obtained by back analysis of phased subway excavation next to Boston's Don Bosco High School where soil stratigraphy is somewhat similar to the area of the jacked tunnels.[8,14]

Permanent Support for Tunnel Structures on Soil-Cement

In the southern portion of the site (south of the Wye Connector Railroad Bridge), the design incorporated a berm of soil-cement on top of the clay stratum to resist the unbalanced lateral loads from the land-side of Ramp L tunnel. This berm has now been constructed in the Fort Point Channel. The tunnel itself was designed to be founded on deep, drilled shafts socketed in bedrock. Further north, as Ramp L turns east to join I-90 eastbound, the channel was too narrow for installing a berm, so a deep buttress of soil-cement was designed to provide both vertical foundation supports and resistance to unbalanced lateral loads. This condition continues through the area of cut-and-cover tunnels that will link the I-90 eastbound and I-90 westbound ITTs with the jacked tunnels beneath the railroad. Special loading conditions occur at Ramp D where the adjacent ITT cannot accept unbalanced lateral loadings. Along this portion of Ramps L and D, the tunnels were designed to have rigid foundation support to resist both vertical and lateral movement.

Deep Mixing Method

The process called the Deep Mixing Method was used to stabilize the soft soils and create the soil-cement foundation structures that resist both vertical and lateral loads. On this project, triple-shaft mixing equipment has been used with rotary cutting tools on the bottoms that penetrate the ground while injecting either water or cement grout through ports at the bottom of the auger heads. The cutting tools steadily advance into the soil at rates as slow as 25 to 50 millimeters (1 to 2 inches) per revolution, and beater paddles further up the shafts act to disperse and mix the soil and grout, creating a mixture of soil-cement. The three shafts are in one line, with the inner shaft counter-rotating relative to the outer shafts. In granular fill and the organic soils, the deep mixing is fairly easy due to the granular and very soft nature of these soils, respectively. In the clays, the cohesive character requires more effort to churn the clay and grout into a thick paste, which sometimes includes pieces and clumps of clay. (A further discussion of the deep mixing process is in the ASCE Soil Improvement and Geosynthetics Committee Report.[15])

Ramp L Berm

Within the project limits (see Figure 1), the Ramp L tunnel, which will connect I-93NB with eastbound I-90, descends in grade from a shallow boat structure requiring only a 4.5-meter (15-foot) deep excavation, to a 17-meter (55-foot) deep tunnel where it passes beneath the Wye Connector Railroad Bridge. The tunnel alone in the existing ground conditions would experience significant lateral movement and long-term creep due to the unbalanced lateral loading that would occur with the tidal Fort Point Channel on one side and the ground at a 9-meter (30-foot) higher elevation on the other. Since the Ramp L tunnel also supports several columns from the continuous concrete segmental viaduct system that is parallel to the tunnel along the channel west side, horizontal movement of the tunnel must be prevented. The existing organic soils stratum on the channel side of the tunnel could not provide the necessary lateral shear restraint, and excavation

and replacement of organic soils beneath the channel with granular fill to form a berm on the top of the clay stratum was not practical.

A soil-cement berm penetrating into the clay was designed to laterally restrain the Ramp L tunnel (see its cross-section in Figure 7). Ramp L was designed to accommodate 13 millimeters (0.5 inch) of horizontal displacement. The berm width was sized so that about half of the clay shear resistance is mobilized. The rock-socketed drilled shaft foundations that support the vertical loads of the Ramp L tunnel and viaduct also provide some resistance to the unbalanced lateral load.

In the deeper, northern section of Ramp L in the vicinity of the Wye Connector Railroad Bridge, a 6-meter (20-foot) thick base of soil-cement was designed to be installed below the tunnel before excavation to resist basal heave and global instability. However, the contractor elected to install deep concrete slurry walls for excavation support and lateral cross walls to provide internal horizontal restraint and resistance to basal heave. Construction of the soil-cement berm for the southerly portion of Ramp L is complete, as is the southernmost portion of the Ramp L tunnel. The principal installation difficulty for deep mixing was with obstructions, including existing woodpiles, timber platforms and wharf decks, and granite blocks for pile caps and building stones (and concrete in some later cases). Obstruction removal was performed on an as-needed basis following the general pre-mixing clearing.

Deep Buttresses for Ramps D & L

At the east side of the project, deep buttresses of soil-cement bearing on glacial till were designed for permanent foundations of the five contiguous tunnels. The principal design issue required deep buttresses to resist unbalanced lateral load. The final geometries of the tunnels themselves in this area narrow the Fort Point Channel to less than half its former width (see Figure 1), thus preventing use of an external berm to resist unbalanced lateral load. Furthermore, the ITT adjacent to cast-in-place Ramp D cannot hold back large unbalanced lateral loads. Different and more complex design issues developed in the application of soil-cement structures as deep buttresses and foundations.[16]

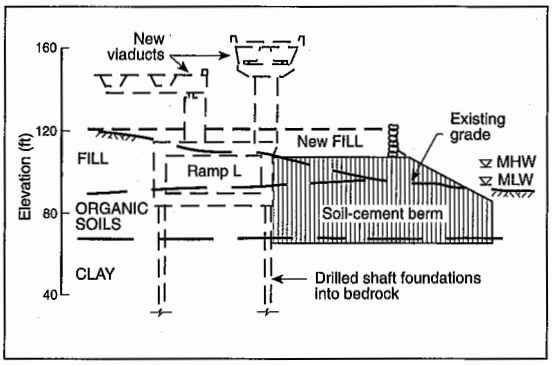

FIGURE 7. Use of a soil-cement berm to buttress the Ramp L tunnel.

The design layout of deep soil-cement buttress structures is shown in Figure 8. For much of Ramp L east of the Wye Connector, shear walls were designed to form triple rows of overlapped, interconnected elements of soil-cement. A typical pattern of auger elements is shown in Figure 9 where the 38 percent coverage makes up the majority of the shear wall width.

For buttress structures to meet the design intent, the soil-cement shear walls require continuity. Therefore, the specification for the installation of the soil-cement by deep mixing required design adherence to close tolerances for alignment, verticality and overlap.

The design and proportioning of soil-cement widths to act as shear panels was determined by structural analysis of internal stresses in the soil-cement.[17] In these analyses, earth pressures and hydrostatic pressures were applied to sides of the buttresses, and base shear resistances developed along the bottom of the buttress in bearing on glacial deposit soils. Particularly high internal stresses developed in portions of the soil-cement buttress along Ramp D due to 15 meters (50 feet) of unbalanced loading. The permanent unbalanced loading was designed to be carried by the adjacent T-shaped slurry wall and Ramp D tunnel down into the underlying buttress. The soil-cement buttresses were designed to develop base sliding resistance on the glacial till (see the cross-section in Figure 10), acting in conjunction with the rock socketed slurry T-wall. As indi-

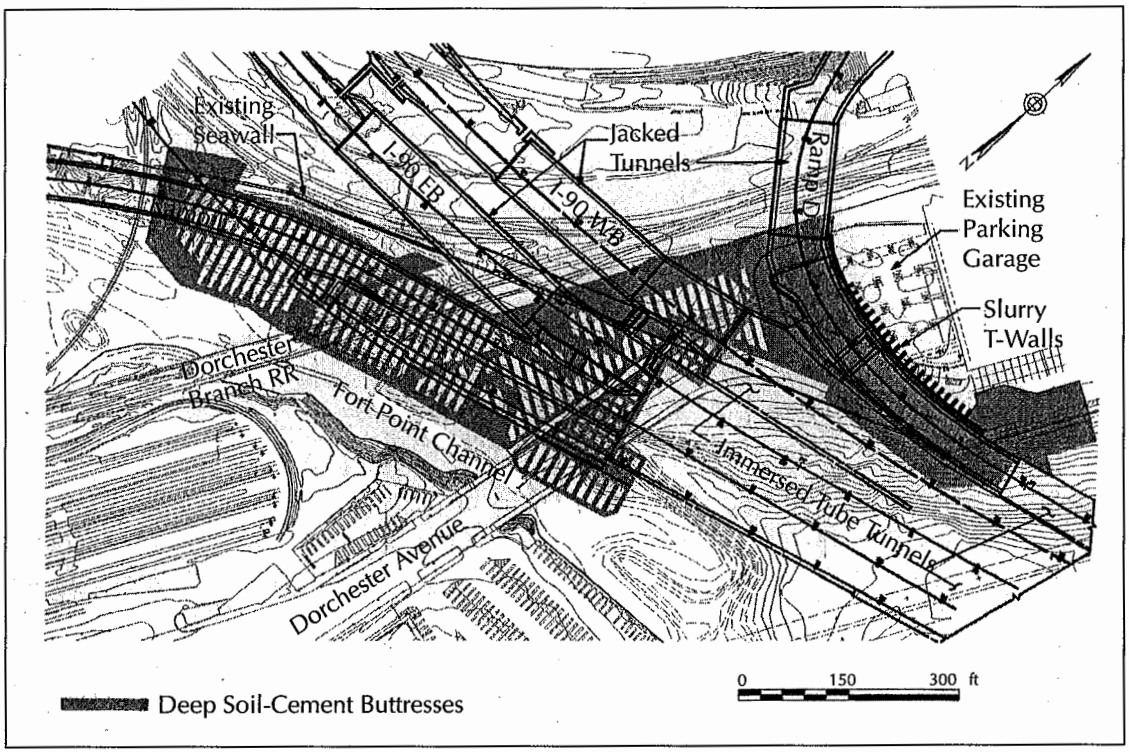

FIGURE 8. Plan of the deep soil-cement buttresses for tunnels.

cated, the ITT adjacent to Ramp D tunnel is founded on drilled shaft foundations into bedrock, which cannot be laterally loaded by Ramp D or the soil-cement buttress.

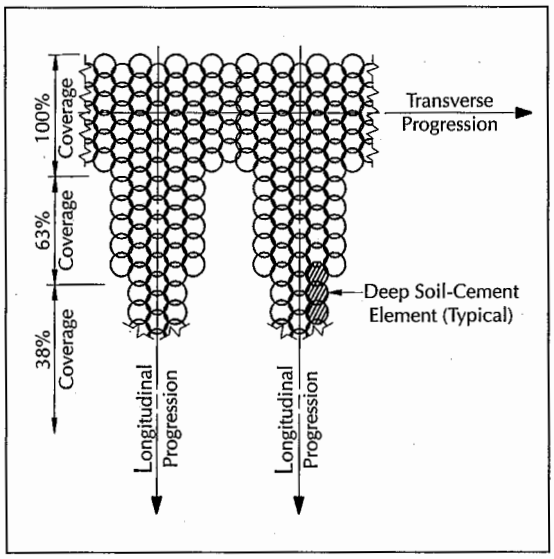

FIGURE 9. Augur patterns to create deep soil-cement shear walls.

Installing Soil Cement in the Crowded Fort Point Channel

Overwater installation of soil-cement is not an unusual construction procedure in Japan. There, huge barges supporting complete grout batching plants and drilling towers for auger mixing are typical. Unfortunately, the Fort Point Channel is generally too confined and water depth is insufficient at low tide to permit the use of such highly specialized water-borne equipment. The two primary bridges in the deep buttress area (Dorchester Branch Railroad and Dorchester Avenue) had to remain open to rail and vehicle traffic until temporary replacements were constructed. These restrictions created the need for a staged approach to overwater installation of soil-cement. Heavy auger equipment was required to reach the 35-meter (130-foot) depths to base the soil-cement on glacial till. The "working platform" for the huge crawler cranes that serve as the base for the deep mixing equipment was formed between the bridges by first mixing soft organic soils and channel bottom "mud" with cement

grout to create a stabilized layer of weak soil-cement on top of the clay stratum. This "shallow mixing" was within areas constrained by the existing bridges, and confined by perimeters of steel sheet piling.

Once the shallow soil-cement was installed, the "working platform" was created with granular fill placed on the shallow soil-cement. The heavy deep mixing crane-rig then operated on this working platform. With deep mixing completed in the two initial areas, the temporary railroad bridge and the relocated Dorchester Avenue roadway/bridge were constructed. With the Dorchester Avenue traffic relocated to the temporary crossing west of the current bridge, the old bridge was removed and deep mixing performed by an overwater operation to complete the soil-cement buttress in the easternmost area (see Figure 8).

The old Dorchester Branch Railroad Bridge was a historic bascule bridge on which the new Old Colony commuter rail train service began operations in September 1997 on three active tracks (originally there were six). Since commuter service must be maintained, a temporary bridge with three tracks has been built to the west of the bascule bridge, which has now been removed. In this area, the final portions of the deep soil mixing can then be performed to link the two initial areas. At the same time, on-land deep mixing progressed for the Ramp D buttress and areas east along I-90 westbound. To achieve efficiency in constructing soil-cement buttress structures, conscientious pre-removal of obstructions was essential.

Soil-Cement Use as Temporary Excavation Support

Along the 150 meters (500 feet) of Ramp L from the Wye Connector bridge to the Dorchester Avenue bridge, a temporary channel-side cofferdam was designed as two parallel rows of sheet piles installed in a zone of 100 percent coverage of soil-cement, as shown in plan and cross-section in Figure 11. A row of sheet piles was also designed to be installed at the other side of the soil-cement adjacent to the existing seawall (along the railroad yard). Although a free-standing excavation support system at

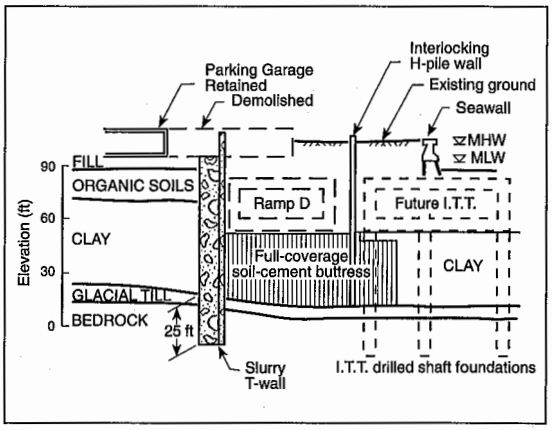

FIGURE 10. A cross-section of the Ramp D full-coverage soil-cement buttress.

the old seawall was desired, the space available was not sufficient to form a cofferdam to independently hold back earth and hydrostatic pressures. A system of struts between the seawall (railroad-side) sheet piling and the channel-side cofferdam was designed to resist the earth and hydrostatic loads from the seawall during excavation for tunnel construction.

The 9- to 12-meter (30- to 40-foot) wide "channel-side cofferdam" also provides a staging platform for construction of the Ramp L and HOV-EB tunnels. After tunnel construction, the cofferdam will be removed, but the soil-cement below will remain. As indicated in Figure 1, the final channel width will be significantly narrowed; however, hydraulic capacity will be adequate for upstream drainage. The completed north side of the channel will be bounded by the seawall on the outside of the HOV-EB tunnel, which will be faced with granite blocks to simulate the appearance of the existing historic seawall.

Further to the east, the soil-cement placed by overwater operations was designed to support three filled circular cellular cofferdams. These cofferdams form freestanding excavation supports at the southern side of the area where the interface cast-in-place tunnel is to be built. The connection between the jacked tunnels and the ITTs was designed to be built in an open excavation, with major perimeter support provided by the future tunnels and the three cellular cofferdams. For

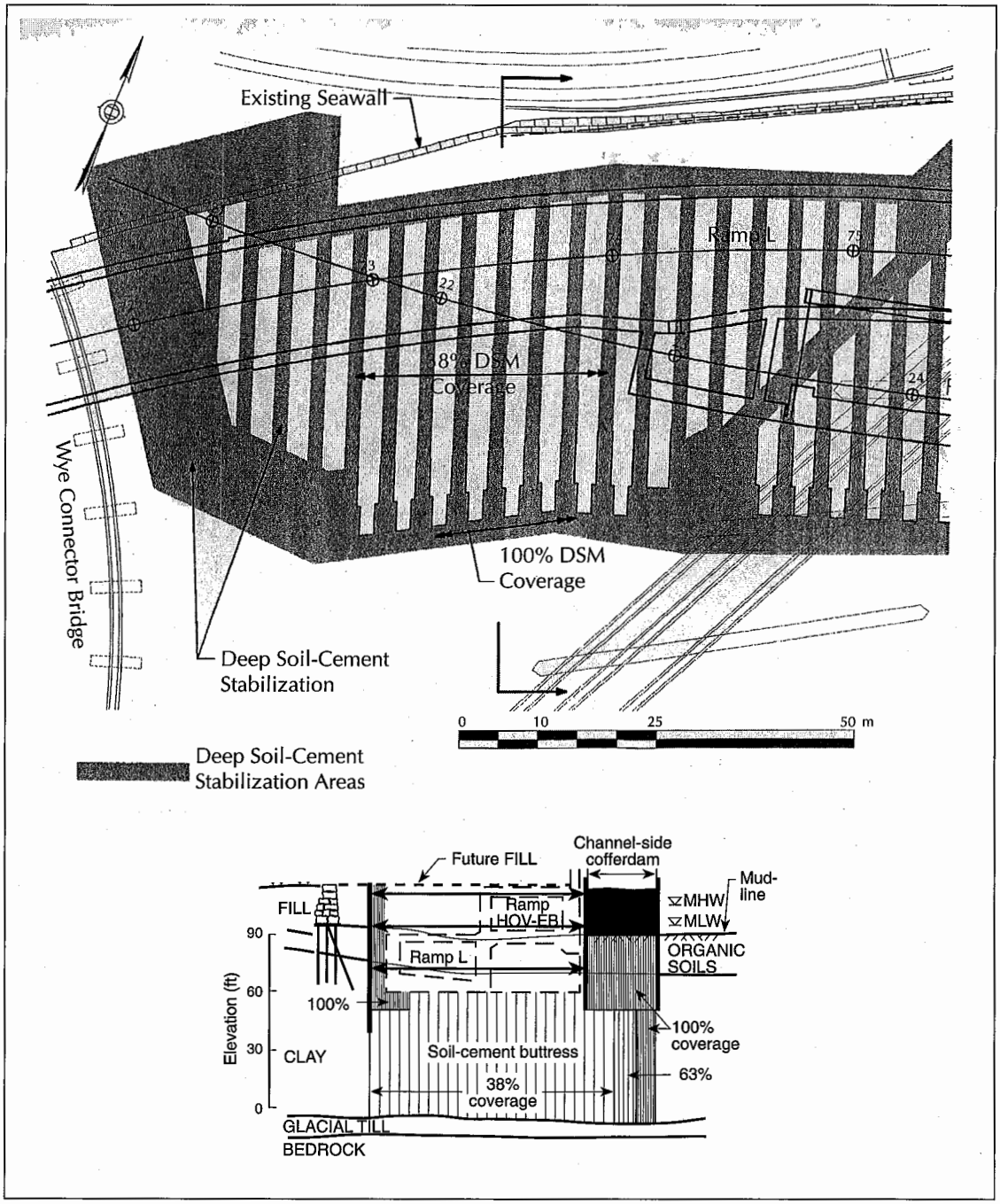

FIGURE 11. Plan and cross-section showing soil-cement use for both excavation support and tunnel buttress foundation.

this last stage of construction, freestanding support systems were designed to be erected on top of the jacked tunnels and the ITTs, with the Ramp D tunnel forming the fourth side. A cross-section through the cellular cofferdam is shown in Figure 12. The foundation design for the circular cells was 100 percent coverage soil-cement carried down to about 6 meters (20 feet) below the bottom of excavation, below which are shear walls of 38 percent coverage.

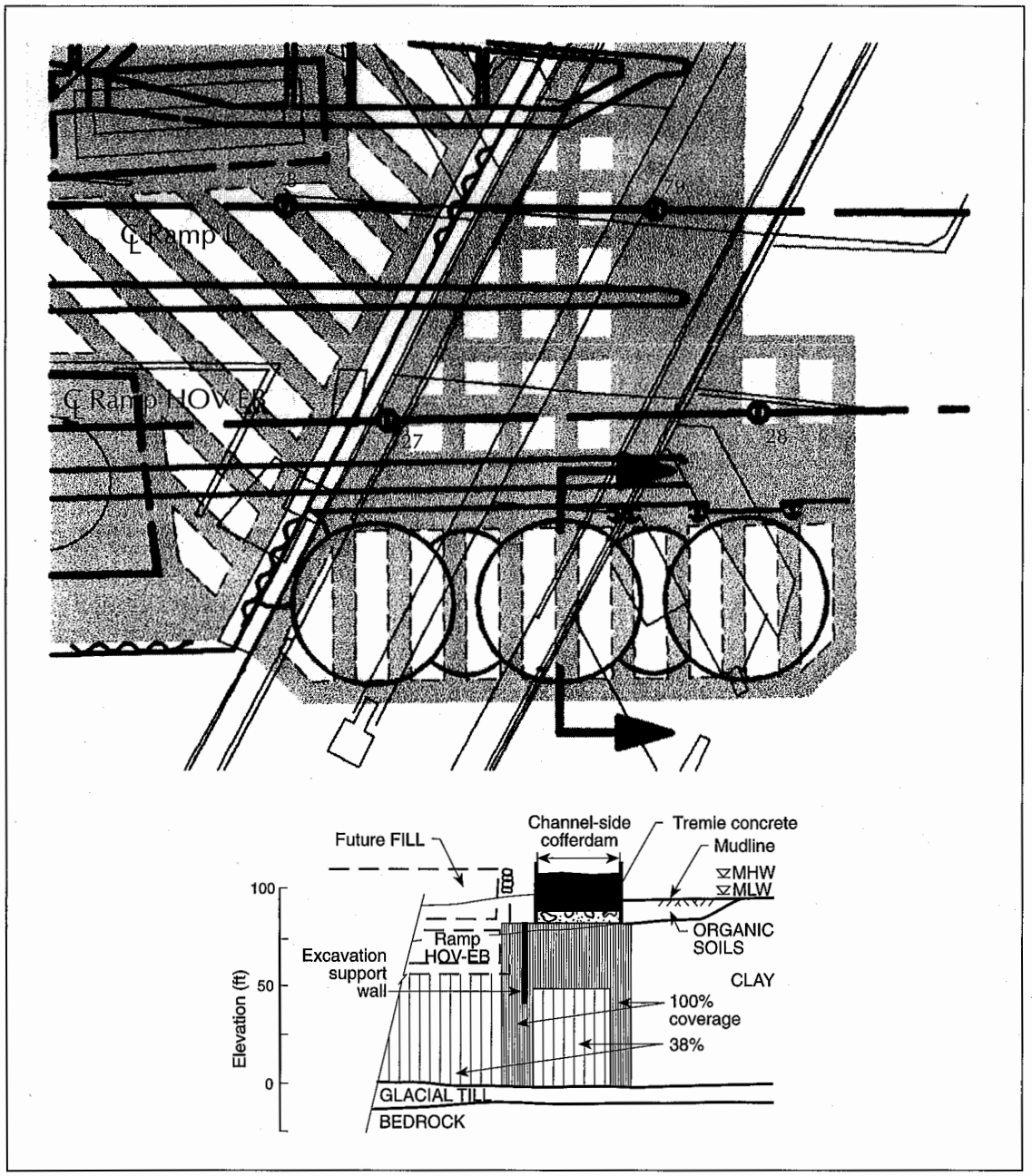

FIGURE 12. Plan and cross-section of cellular cofferdam on soil-cement for large junction excavation.

The Tunnel Jacking Scheme

The three tunnels that are being installed by the jacked tunnel method beneath the mainline railroad tracks leading to Boston's South Station are of the overall sizes listed in Table 1. The locations of the jacked tunnels in final position are shown in Figure 1. The design sizes of the jacked tunnels were set both by the roadway alignments and tunnel installation tolerances (which determine the final tunnel envelope), and by proximity limits set by the railroad authority for thrust pit and reception pit headwall positions adjacent to the tracks. The tunnel

TABLE 1.
Jacked Tunnel Sizes

Tunnel	Length	Width	Height	Depth of Cover
Ramp D	48 m (158 ft)	24 m (79 ft)	11.5 m (38 ft)	7 m (24 ft)
I-90 Westbound	76 m (249 ft)	25 m (82 ft)	11.5 m (38 ft)	5.5-8 m (18-26 ft)
I-90 Eastbound	107 m (350 ft)	24 m (79 ft)	10 m (32 ft)	1-7 m (3-24 ft)

alignments must accommodate highway geometries, installation requirements of the jacking technique, and railroad operational constraints. The profiles along alignments of the Ramp D and I-90 westbound tunnels are shown in Figure 5 with relative depths and soil stratigraphy indicated.

The thrust pits, in which each tunnel is cast and then advanced into the ground, are west of the tracks, as shown in Figure 13. In the tunnel-jacking scheme, each tunnel is divided along its length into a number of units that provide manageable jacking loads and practical jack arrangements. Hydraulic jacks are utilized both between each unit at intermediate jacking stations (IJSs), and behind the rear unit. The unit lengths designed were typically 15 to 20 meters (50 to 65 feet), but have been lengthened somewhat by the contractor. The lead unit is as long as possible to aid alignment — for the first tunnel jacked, this length is now about 27 meters (90 feet), yet it must to be short enough so direction can be controlled by steerage jacks. The trailing units must be long enough to transfer jacking reaction at base slab level without lifting off the thrust base at its front end. The location of IJSs and jack details prepared for the design of the shorter Ramp D tunnel are shown on Figure 14.

The arrangement of thrust pits and the sequencing for tunnel jacking were designed to fit within limited available site areas between the railroad tracks and the adjacent existing interchange roadways. Unusual measures developed in the design to fit the jacked box units for I-90 eastbound into the jacking pit are illustrated in Figure 15. The very limited I-90 eastbound thrust pit length together with the design requirement to have all units cast prior to the commencement of the jacking operation necessitated design planning whereby the three rear units could be cast off-line within the I-90 westbound thrust pit. These units were then to be slid into line behind the front three sections during the I-90 eastbound jacking operation, in the sequence shown on Figure 15. However, a somewhat different sequence is being applied by the contractor wherein the second set of tunnel units will be constructed after the first set has been advanced into the ground.

Thrust Pits

To form the aforementioned thrust pits, special single-strutted excavations up to 70 feet deep have been constructed adjacent to the railroad tracks. The earth support system for the thrust pits consists of rigid soldier pile tremie concrete (SPTC) walls and reinforced concrete slurry walls (T-shaped in plan and post-tensioned for additional stiffness). A base mat of jet grout was provided in the design to be installed from ground surface to form a bracing element in excess of 6 meters below invert level to provide restraint to inward wall movement during excavation. This restraint and the cantilever effect of the walls penetrating below the base mat reduced the potential for ground surface settlement, which is a particularly important issue adjacent to the tracks.

In the design, the excavation sequences required intermediate supports using large pipe struts. Then, after installation of the 1- to 1.5-meter (3- to 4-foot) thick reinforced concrete thrust base slab that acts as a bottom permanent strut, the lower pipe struts would be removed to create approximately 11-meter (35-foot) high working spaces for the construction and installation of the jacked tunnels.

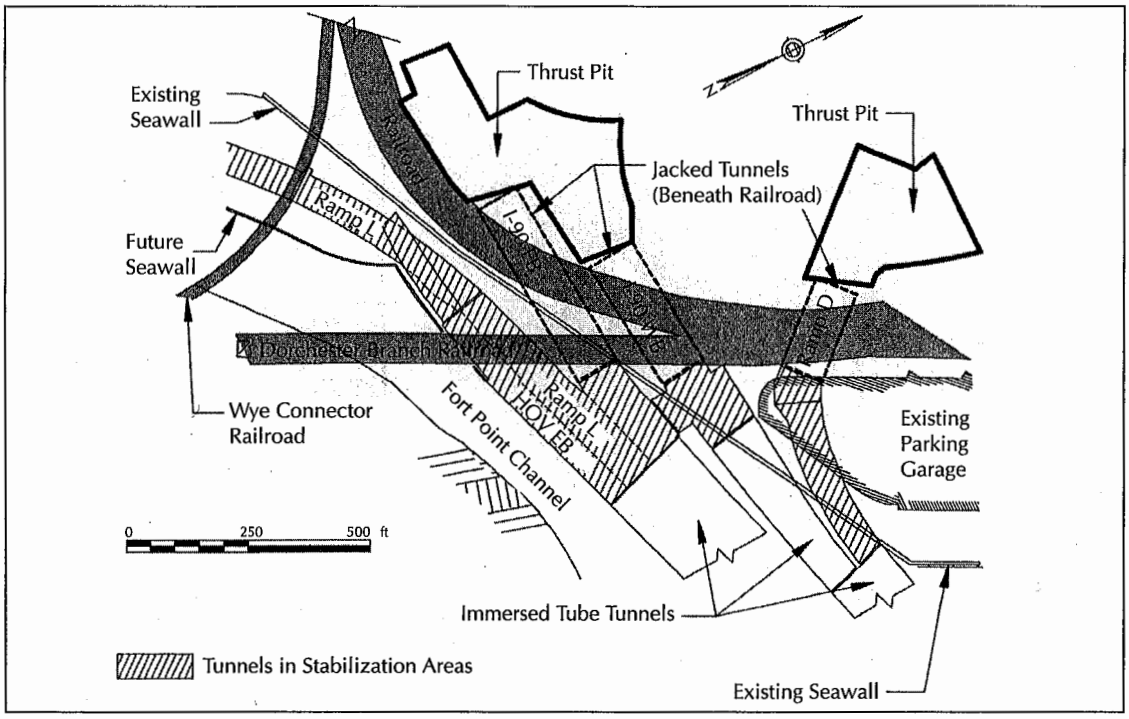

FIGURE 13. Plan locations of the thrust pits and final jacked tunnels.

The jacked tunnel units are cast on the thrust base slab. Jacking loads are applied onto an "upstand" wall at the rear of the thrust base, and resistance to prevent backward movement of the thrust base is from skin friction along the soil interface both below the thrust base/jet grout and behind the side earth support walls. The structural analysis of the complete thrust base system was modeled using three-dimensional

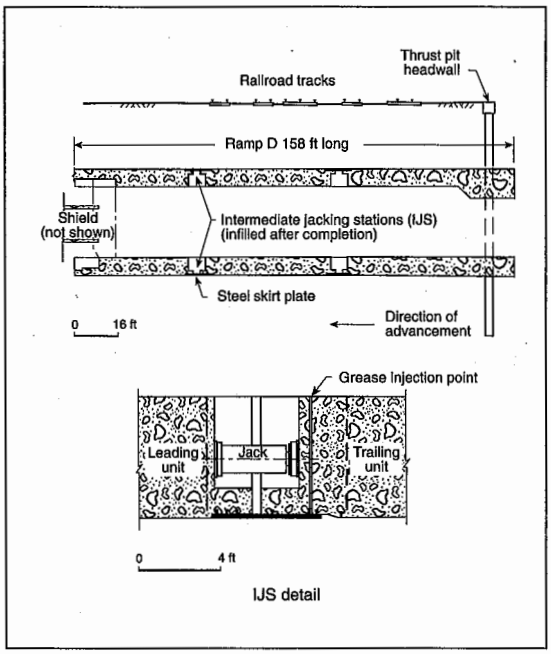

FIGURE 14. Intermediate jacking stations location and detail.

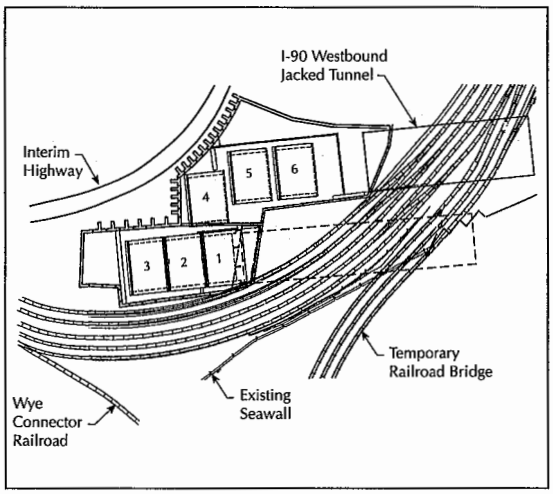

FIGURE 15. Plan showing off-line casting of I-90 eastbound tunnel segments.

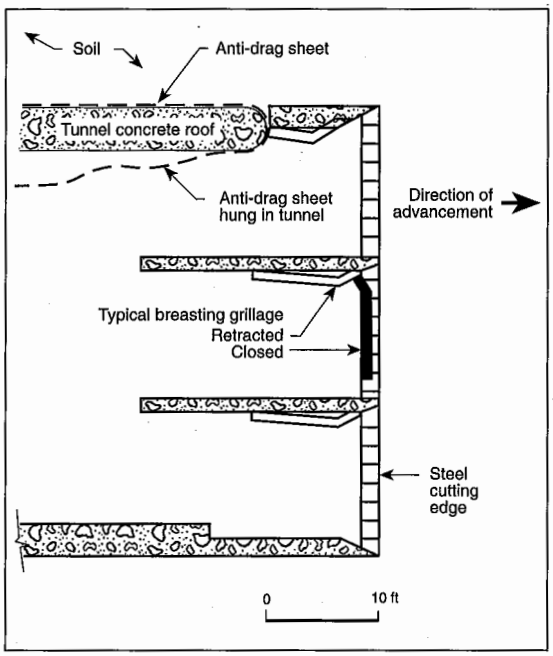

FIGURE 16. Anti-drag sheet and other details of the jacked tunnel shield.

finite element analysis. Lateral movements of the pits were assessed by finite difference analysis using the program FLAC. The calculated thrust pit displacements under working jack loads of approximately 180,000 kN (20,000 tons) are expected to be on the order of 25 millimeters (1 inch).

The thrust pit headwalls are the entry points for the tunnel from the pit and into the ground. Headwall removal must be expeditious at the start of jacking since the shield penetrates the headwall and the ground beyond. A cross-section of the shield designed for these tunnels is shown in Figure 16. It includes breasting grillages that can be closed in case of prolonged work stoppage or unstable ground.

The headwall was designed to be modified SPTC elements, comprised of vertical steel wide flange sections in a cement-bentonite matrix (rather than concrete). Headwall entry is usually a critical phase in the jacking operation, because lateral loads supported by the wall must be transferred temporarily to the tunnel shield until the support can be provided by embedment of the shield in the ground. The design provided for a gradual entry with headwalls skewed at 15 degrees to allow for progressive shield advance and wall removal in sections across the face. A skewed head wall does, however, produce an imbalance in ground pressures on the sides of the lead tunnel unit, which, in turn, can cause the tunnel to shift off line during the jacking cycle. This shift is then controlled by guides at the front of the thrust base and by the application of corrective adjustment to the rear jack forces.

Jacking Forces & Hydraulic Jacks

The design for the hydraulic jacks used to push tunnel units through the ground called for 4,500 kN (500 ton) capacity. Typically, 40 jacks were to be installed at each IJS. Following initial headwall breaching, the rear jacks pushing against the upstand wall are used to drive the full tunnel forward into the ground until the lead unit is fully embedded. Then, the jacks pushing against the first IJS are used to drive just the lead unit forward. Each trailing unit in turn is then pushed forward by its IJS jacks. Thus, the whole tunnel is moved forward and through the ground in a "caterpillar" action. This use of IJSs at regular intervals reduces the force required at any one time to push the tunnel segments ahead. This arrangement also assists in the directional control of the tunnels during installation and can reduce the amount of ground disturbance.

Multiple jacks are used to provide a factor of safety on calculated working loads on the order of 2, thereby allowing for variations in jacking resistanc e. This jack "over-capacity" makes additional force available should it become necessary to exert directional control. The use of multiple jacks also allows for the equipment to be operated at lower hydraulic pressure than their rated capacity in order to reduce wear and maintenance.

Reducing Friction Between the Tunnel & Ground

There are several benefits in reducing friction between the moving tunnel and the stationary ground. The first benefit is to lessen the pushing force required from the hydraulic jack system. The friction reduction acts to produce a more uniform friction around the tunnel perimeter, which aids in installation alignment, and prevents excessive ground settlement and

drag distortion. The design included several measures to minimize drag on the tunnel units as they advance beneath the railroad:

- An anti-drag sheet system between the tunnel roof and the overlying soil, with grease pumped through the roof to the underside of the drag sheets.
- Specified close tolerances and smooth finishes on the external concrete faces of the tunnel units.
- Overcut plates down the sides of the leading edge of the shield.
- Lubrication around the tunnel sides with bentonite slurry.

The anti-drag sheets provide a separating membrane between the tunnel and the overlying soil whereby the sheets remain static relative to the soil above to reduce the tendency for the advancing tunnel to drag soil forward, which can result in track movements. The anti-drag sheets have to withstand large tensile forces; they are anchored at the headwall of the thrust pit and, thus, prevent the overlying soil block from being carried forward with the advancing tunnel units. The anti-drag sheet system designed was approximately as shown in Figure 16, with the sheet being comprised of multiple laminated layers of steel sheets 0.8 millimeters (1/32 inch) thick to maintain flexibility for feeding out through roof slots behind the shield. The system was designed to cover about 70 percent of the roof surface. The grease injection ports were designed to be through the roof of the tunnel (not shown on Figure 16). However, the contractor is using a patented anti-drag wire rope system for both the roof and invert slab. The effectiveness of friction reduction accomplished by an anti-drag system is to some degree related to the depth and nature of the overlying soils.

Control of Tunnel Alignment With Guide Paths

Jacked tunnels can be installed within close tolerances. However, once a tunnel starts to go off-line during the jacking cycle it can be difficult to correct the alignment. Therefore, primary emphasis must be given to ensure that the tunnel starts off in proper alignment. Then, throughout the tunnel advancement, efforts must be taken to accurately maintain the alignment, with corrective steps taken immediately if any deviation becomes apparent.

Highway-sized jacked tunnel units cannot be pushed or pulled back on-line as can small-diameter jacked pipes. When there is concern for the ability to maintain vertical alignment, then "guide paths" can be installed prior to tunnel jacking to aid in resisting the tendency of a tunnel to dive into soft ground. The design of these tunnels included two guide paths per tunnel to provide more positive guidance than that afforded by using only the steerage jacks at the lead IJS. Guide paths were included in the design to mitigate the risk of exceeding the allowed vertical alignment tolerances. The function of the guide paths is to limit the effect of nosing by providing a load resistance ahead of the tunnel face. Essentially, the guide paths act as a pair of skis ahead of the jacked tunnel. They are not intended or designed to support total vertical tunnel loads.

The guide path arrangement, shown in Figure 17, was for reinforced concrete guide paths to be cast into the bottom half of 2.8-meter (9-foot) diameter tunnels driven by conventional means. On top of the guide path, special slide tracks of aluminum/bronze running plates on steel billets were called for, over which the slide base of the jacked tunnels ride.

For long jacking distances, special construction methods must be employed to aid in maintaining vertical alignment. These methods include:

- Careful construction tolerance control to ensure that the tunnels are of regular cross-section without surface distortions or irregularities;
- Control of friction around the tunnel to minimize sudden changes in drag forces;
- Locating IJS jacks in the corners of units to maximize the corrective moment that can be applied;
- Side guides on the jacking base for horizontal alignment for unit entry;
- Continual careful mining techniques; and,
- Application of ground treatment to reduce the chances of local instability that would affect control of shield advancement.

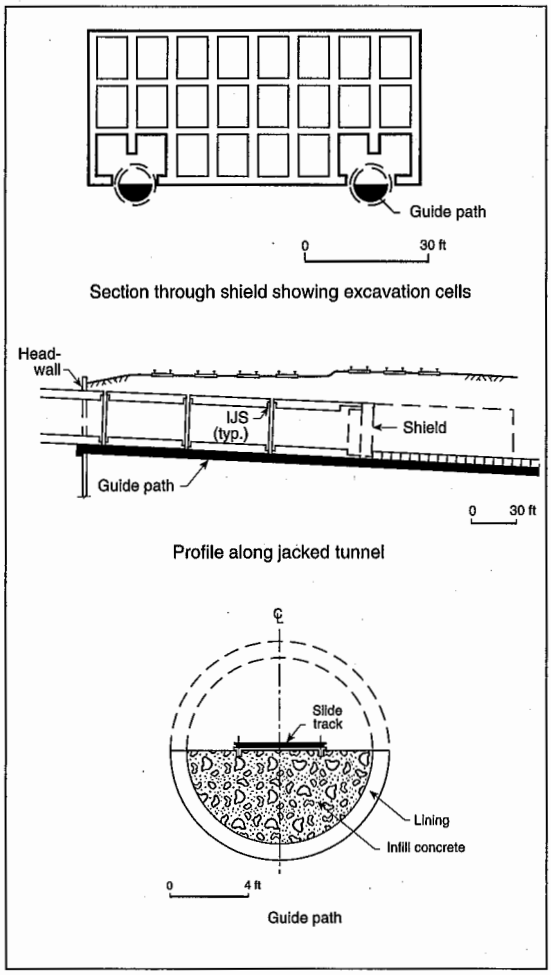

FIGURE 17. Tunnel profile and shield section showing guide paths.

Calculations during design indicate that for the assumed site ground conditions, there could be a tendency for the lead unit to nose down under the peak loads on the shields if guide paths were not provided. The mechanisms in soft ground are such that the propensity for nosing down increases progressively the further the tunnel is advanced. Other related factors also considered in tunnel alignment control for jacked tunnels in soft ground include:

- Remolding of medium sensitive clays in the zone of high contact stress beneath the shield floor.
- The tendency for the clay beneath the shield floor to become softer due to relaxation into the open face.
- The passage of the trailing units over soft ground, which could cause the units to scour themselves deeper into the ground, causing misalignment and more severe settlements at ground level.

To analyze the forces on the guide paths and underlying soil for structural design purposes as the tunnel is jacked over the guide paths, the paths were idealized as beams on elastic foundations. In these analyses, predictions were made of the deformations of the guide tunnels to assess their effectiveness in maintaining the required vertical alignment control. The results indicate that the limited contribution of the guide paths would be sufficient to boost the tunnels and prevent "nosing" into the soft clay. An assessment was also made of the contact pressures between the underside of the jacked units and the slide track at the top of the guide path.

Control of the Tunnel Face

Tunnel jacking is, in general, a soft ground tunneling technique that has an essential requirement of controlling ground movements in order to keep resulting surface settlements within tolerable limits. Therefore, the design must incorporate the means to minimize loss of ground into the tunnel face. The tunnel design provided a shield, which can support the ground at the face while providing the means for efficiently excavating the soil at the face. The shield design also included breasting grillages, which can be closed to provide additional face support if the ground stability conditions warrant. Further, ground treatment measures were included for application prior to and during jacking to aid in maintaining face stability, thereby controlling settlements and providing for the safety of the overlying railroad and the workers at the excavation face.

The shield geometry and the level of ground treatment are interrelated — more ground treatment tends to allow for a more open shield face. The balance between these two factors depends on ground conditions, size and geometry of the tunnel face and the means of excavation and integral face support. The tunnel shields designed for this CA/T Project section provided a number of face compartments to

enhance stability and to provide convenient access to all sections of the face (see Figures 16 and 17). Two horizontal shelves were provided in the design to divide the face into three levels of cells, each with about 3-meter (10-foot) clear height. This height is convenient working headroom, which permits mechanical excavation, and yet is sufficiently low for worker access to the face to remove obstructions. A wide blunt edge was designed for both the shelves and the vertical walls (spaced about 3.3 meters [11 feet] apart) to provide further face support. A conventional steel cutting edge was required at the perimeter of the shield.

The stability of ground in the shield excavation depends on the inherent strength of the soil, its resistance to erosion by water, the degree of control provided by the shield, and the techniques used by the construction workers at the face. In the granular fill materials where strength is provided by friction, the greatest threat to stability is possible erosion resulting from groundwater gradients and flowing water at the face. In the clay, stability concerns center primarily on low undrained shear strength.

Due to the relatively thin cover that is largely granular fill, it was essential to predrain groundwater from the fill stratum prior to advancing the tunnels. In the lower, soft cohesive strata, horizontal reinforce ment was included in the design to enhance face stability.

Design analyses were made of tunnel face stability for two different soil stratification conditions (shown in Figure 5) using FLAC. The first model, for the Ramp D alignment, specifically addressed the strength of the Boston blue clay and its effect on face stability. The second model was for the shallower I-90 eastbound and westbound tunnels that have granular fill in the upper cell, organic soils in the middle shield level and clay at the lower level. The results showed the need for draining groundwater from the permeable layers, and for resisting face pressures in the clay. A soil nailing system was developed in design to add resisting force to the ground at the tunnel shield face.

Ground Treatment to Enhance Face Stability

Considerable ground treatment measures were required in the design to enhance stability at the face, comprising a combination of dewatering, grouting and soil nailing. Also, retractable face support grillages were to be provided in each cell of the shield, ready for contingency use and to support the face during any prolonged jacking stoppages. It was not, however, intended that grillages be used during normal jacking operations.

Dewatering of the area near the tunnel is essential prior to breaking through the headwall, and was designed to be carried out from the ground surface. To reduce dewatering-induced ground settlements, a perimeter grout curtain cut-off down to the clay was set out to surround the plan area of the jacking operation. During the tunneling operations, local dewatering could be achieved as needed using lances from the shield.

Face stability analysis indicated that stability is significantly enhanced where the fill is grouted in front of the shield. The grouting also helps support the ground when removal of obstructions is necessary. Therefore, the design required fill and sand strata immediately behind the headwall to be grouted using sodium silicate to at least 6 meters (20 feet) horizontally from the headwall prior to shield entry. The design also required grouting the fill and sand strata within the tunnel cross-section directly from the tunnel face to maintain a minimum 6-meter (20-foot) zone of grouted ground ahead of the advancing shield (with the grouting operation extending ahead of the face by about 12 meters [40 feet] at repeating intervals). Figure 18 shows the pattern of ground treatment designed for I-90 westbound tunnel. To enhance the stability of softer zones in the clay, soil nailing was designed to be installed, as indicated.

Ground Movements, Track Settlement & Instrumentation Monitoring

Ground movements are expected during all stages of wall installation, pit excavation and grouting. However, during design, the greatest potential for ground movements was considered to be associated with the tunnel jacking operations. Assessments of face loss and shield overcut were used to produce cumulative settlement contours as jacking proceeds, and to evaluate surface gradients and rates of move-

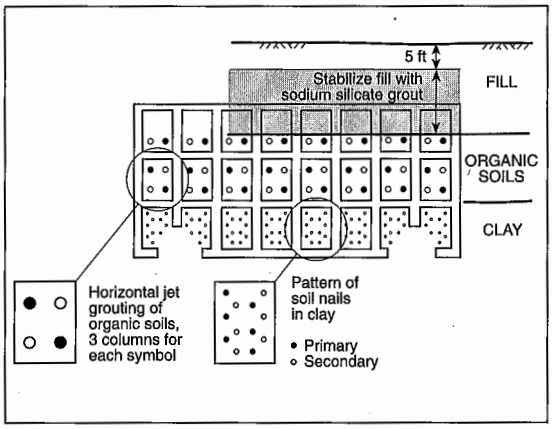

FIGURE 18. The design requirement for ground pre-stabilization at I-90 westbound.

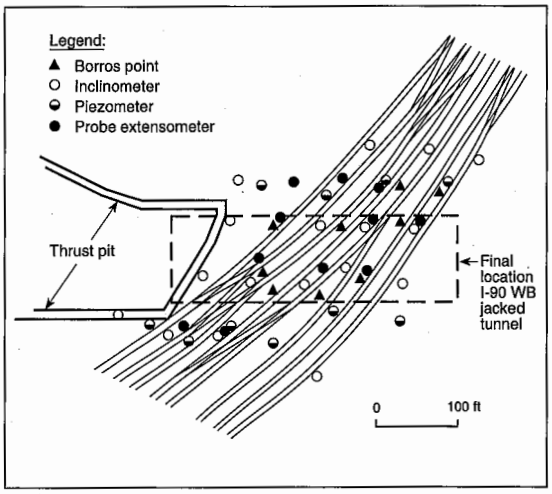

FIGURE 19. The geotechnical instrumentation layout for I-90 westbound.

ment. Lateral ground movement toward the open excavation face at the shield was not expected to be more than 15 centimeters (6 inches). Other factors considered (jacking tolerances, ground lost at major obstructions and in their removal, shield overcut, soil scouring at IJSs and the effects of dewatering) caused predicted cumulative surface settlement to be on the order of 25 centimeters (10 inches). For the rate of settlement that was forecast, it was determined in design that routine railroad maintenance and regular surfacing and reballasting of the tracks could keep net movements within the threshold limit of 25 to 50 millimeters (1 to 2 inches).

The design included extensive geotechnical instrumentation to monitor vertical displacement at the ground surface and vertical and horizontal movements within the ground. The monitoring system will provide early warning of any unexpected ground movements that may cause difficulty to railroad operations. It will also provide data for the track maintenance program as well as permit comparison of actual movements to predictions for back analysis and adjustments to tunneling methods.

The scope of the complex instrumentation layout designed for the path of the I-90 westbound tunnel is shown in Figure 19. Optical surveying will be used to monitor track movement.

Contractor-Proposed Design Changes

The specialist nature of construction activities and the variety of ways in which tunnels can be installed often give rise to benefits in allowing the contractor to develop temporary works to suit its experience, methods of working and equipment. The case on the CA/T Project in Boston allows the contractor to implement several modifications to the original design, including:

- An anti-drag steel rope system (contractor's patented method) instead of the laminated steel sheet method called for in the baseline design for this project section.
- Ground freezing to support the ground ahead of the jacked tunnel instead of extensive grouting and soil nailing.
- Elimination of the guide paths in expectation that the frozen ground will relieve pressures that would otherwise drive the tunnel downward.

Ground freezing is expected to provide a more homogenous ground improvement than the array of grouting and soil nailing originally designed. It does, however, require a great degree of access over the tracks in order to install the brine freeze tubes, which was not an option for the designers. The freezing process (installation and soil solidification) requires substantial installation, operating equipment and maintenance; however, the contractor has accepted these requirements so that jacking can

proceed with fewer disruptions once the freeze zone is established than would be required for incremental application of grouting. Freezing the ground ahead of the tunnels also has allowed modification of the shield details because soil movement into the face can be significantly restrained.

Summary

The use of soil-cement stabilization to form berms and deep permanent buttresses and foundations for tunnels is an important innovation in the design of measures to "tame" the difficult ground conditions in the complex I-90/I-93NB interchange at Fort Point Channel that is currently under construction. The southern 290-meter (950-foot) long portion of Ramp L tunnel is buttressed against 11 meters (35 feet) of unbalanced lateral earth loads by a 12-meter (40-foot) thick by 37-meter (120-foot) wide berm of soil-cement that stabilized miscellaneous granular fill and a very soft stratum of organic silt. In narrower channel areas where the berm is not possible, the soil-cement was designed to extend to depths of 30 to 38 meters (100 to 125 feet) to bear on glacial till deposits, thus forming rigid buttress shear walls that resist the unbalanced soil and water lateral loads. In all, approximately 600,000 cubic meters (800,000 cubic yards) of soil-cement are being formed to stabilize soft organic soils and marine clays to provide excavation base stabilization and resist unbalanced lateral loads.

The tunnel jacking technique was designed for the crossing beneath the busy railroad at Boston's South Station Transportation Center. It was proposed as a solution to avoid extensive relocating of the tracks for incremental cut-and-cover tunnel construction. The three tunnels at depths of 11 to 18 meters (35 to 60 feet) with lengths from 48 to 107 meters (158 to 350 feet) are first built in large braced excavations adjacent to the tracks, and then jacked under the tracks using shield tunneling methods by incremental jacking and excavation.

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REFERENCES

1. Fairweather, V., "Boston Blockbuster," *Civil Engineering*, ASCE, December 1996.

2. O'Rourke, T.D., McGinn, A.J., Dewsnap, J., &

Stewart, H.E., "Case History of an Excavation Stabilized by Deep Mixing Methods," *Design and Construction of Earth Retaining Systems*, Geotechnical Special Publication No. 83, ASCE, 1998.

3. Cheney, R.E., "Deep Soil Mix Wall Technology, *Proceedings STGEL Conference*, Atlanta, Georgia, October 1994.

4. Bruce, D.A., "The Return of Deep Soil Mixing," *Civil Engineering*, ASCE, December 1996.

5. Kawasaki, T., Niina, A., Saitoh, S., Suzuki, Y., & Honjyo, Y., "Deep Mixing Method Using Cement Hardening Agent," *Proc. of 10th ICSMFE*, Vol. 3, 1981.

6. *CDM Manual*, Cement Deep Mixing Method Association, Hama-Rikyu, Bldg. 6F, 21-1, 8-chrome, Ginza, Chuo-ku, Tokyo, 1988.

7. Lambrechts, J.R., Roy, P.A., & Winsor, D.W., "Central Artery Project Innovations for Excavation and Tunnel Support," *Proceedings of the Geotechnical Symposium on Excellence in Civil Engineering, Central Pennsylvania Section ASCE*, October 1997.

8. Winsor, D.S., & Taylor, S., " The Development of a Baseline Design for Jacking Highway Tunnels Beneath Mainline Tracks at Boston's South Station," *Design and Construction of Earth Retaining Systems*, Geotechnical Special Publication No. 83, ASCE, 1998.

9. Finsterwalder , K., "Jacking of a Four-Lane Road Tunnel Consisting of a Reinforced Concrete Box Section Under the Main Line of the German Federal Railways as Part of the 'West Tangent' of the Bochum Motorway," *World Tunneling*, April 1989.

10. *Construction Weekly*, "Germans Keep Welsh Trains Going," March 17, 1993.

11. *Rail Bulletin*, "Minimizing Disruption During Bridge Construction — Bedford Case Study," December 1994.

12. Ladd, C.C., & Foott, R., "New Design Procedure for Stability of Soft Clays," *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 100, No. GT7, 1974.

13. ITASCA, *FLAC — Fast Lagrangian Analysis of Continua, Version 3.3*, Itasca Consulting Group, Minneapolis, Minn., 1995.

14. Lambe, T.W., Wolfskill, A.L., & Jaworski, W.E., "The Performance of a Subway Excavation," *Performance of Earth and Earth-Supported Structures*, ASCE, Vol. I, 1972.

15. *ASCE Soil Improvement and Geosynthetics Committee Report*, "Section 2.5, Deep Mixing, Ground Improvement, Ground Reinforcement, Ground Treatment Developments 1987-1997," Geotechnical Special Publication, No. 69, ASCE, 1997.

16. Lambrechts, J.R., & Roy, P.A., "Deep Soil-Cement Mixing for Tunnel Support at Boston's I-93NB/I-90 Interchange," *Ground Improvement, Ground Reinforcement, Ground Treatment Developments 1987-1997*, Geotechnical Special Publication No. 69, ASCE, 1997.

17. Lambrechts, J.R., Roy, P.A., & Wishart, E.J., "Design Conditions and Analysis Methods for Soil-Cement Buttresses in Fort Point Channel," *Design and Construction of Earth Retaining Systems*, Geotechnical Special Publication No. 83, ASCE, 1998.

The Use of Physical Modeling to Enhance Nut Island Headworks Design

A program of physical modeling, after using mathematical modeling on the initial design, can be an indispensable tool in eliminating functional flaws and operational problems.

WILLIAM C. PISANO, HANSJÖRG BROMBACH & RICHARD ATOULIKIAN

Construction of 360-mgd (15,700 l/s) headworks has been recently completed at the existing Nut Island primary treatment facility to provide screening and grit removal prior to discharge to the new Deer Island treatment facility. Flow enters in a 90-degree turn and then is distributed into six screening channels followed by degritting. The 90-degree turn created a large stable vortex and secondary eddies that resulted in an unacceptable distribution of flow between screening channels, with heavy deposition under all design conditions. A curved entrance deflector wall and other hydraulic enhancements satisfying functional design criteria were recommended from the model experiments and were incorporated into the design package. All enhancements have been constructed.

Introduction

As part of the $4 billion Massachusetts Water Resources Authority's (MWRA) Boston Harbor cleanup program, the new Nut Island Headworks has recently been constructed at the site of the existing Nut Island primary treatment plant, which was demolished. The headworks provides screening and grit removal prior to discharge into the new Inter-Island Tunnel for final treatment at the new Deer Island 1,200 mgd (52,500 l/s) facility (see Figure 1).

An existing 12-foot (3.7-meter) high level sewer (HLS) drains the southerly portion of the greater metropolitan Boston area. Its flow enters a junction chamber constructed around the HLS and turns at a right angle into the new headwork's transition section. This new transition conduit distributes the flow into six sepa-

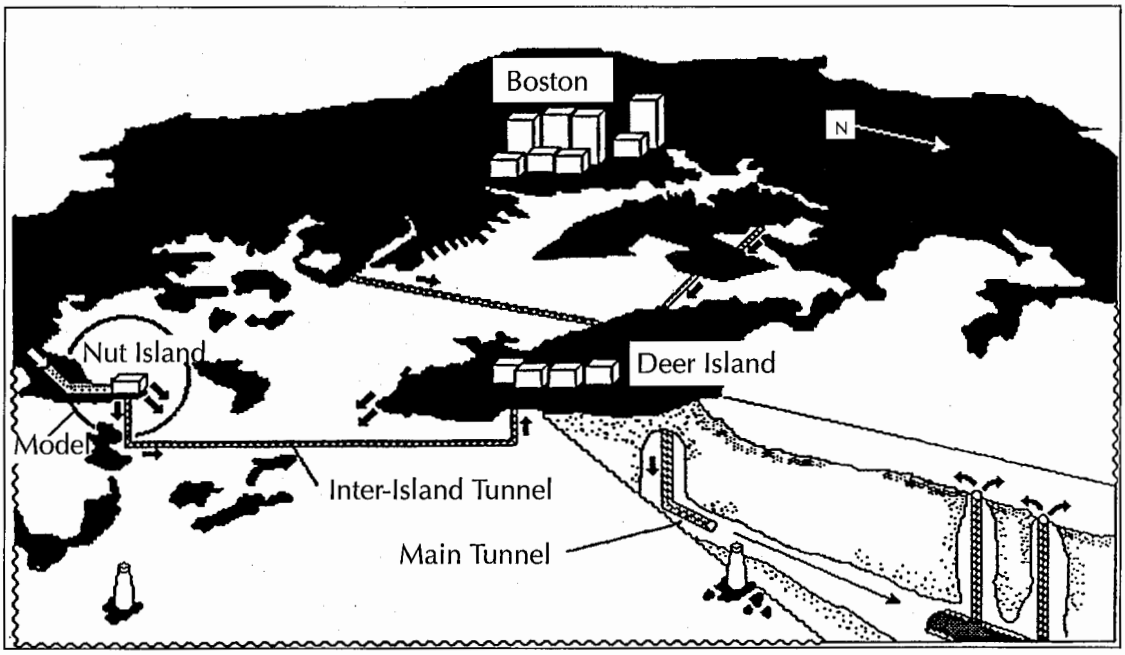


FIGURE 1. General layout of the MWRA Boston Harbor cleanup program.

rate channels for velocity reduction and flow straightening prior to entering the new headworks facility.

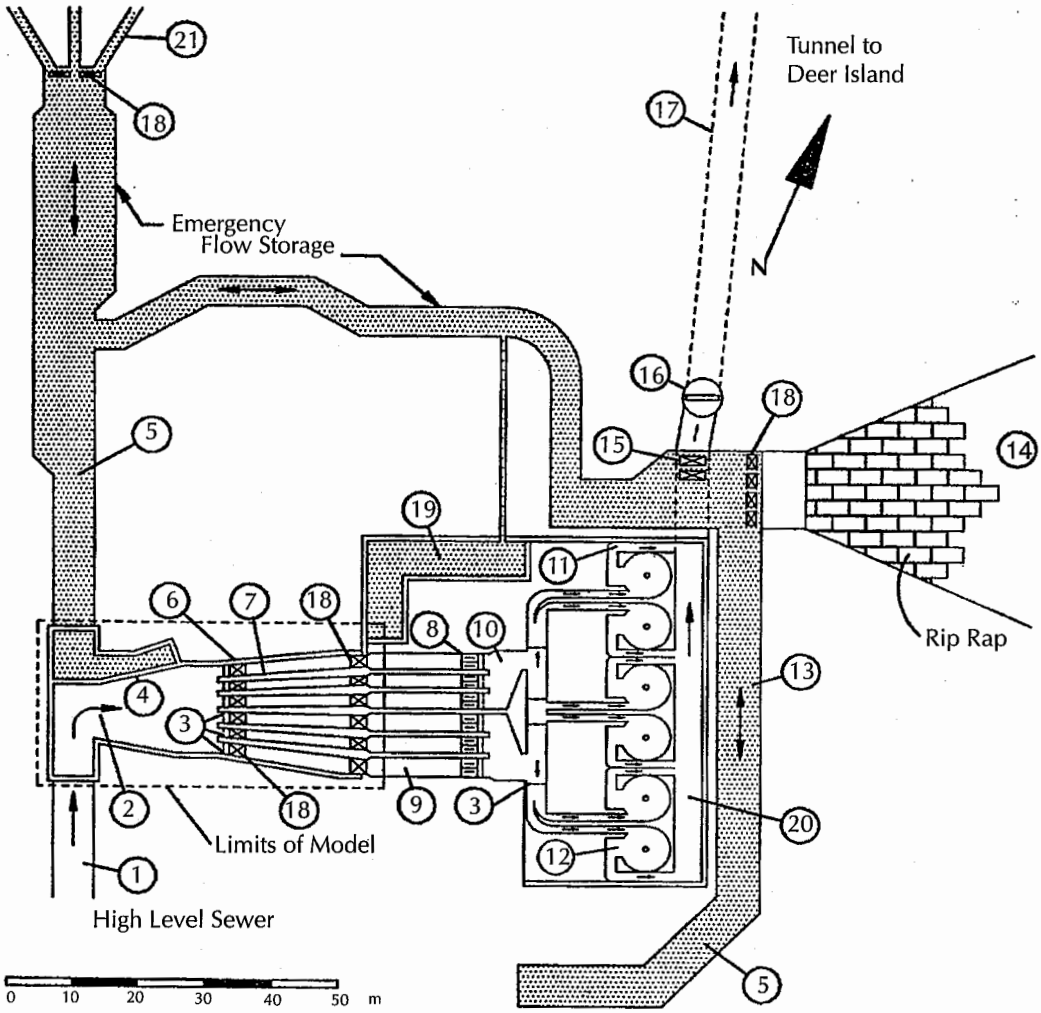
Inside the new headworks facility the six separate channels expand to six screening channels. Six catenary type mechanical screens have been installed, with four units on line at maximum flow and two units as standby. The number of screens in service will vary with discharge (dry weather flow of 80 mgd [3,500 l/s], maximum design flow of 360 mgd [15,700 l/s] and maximum hydraulic capacity of 400 mgd [17,500 l/s] — see Figure 2).

Flow exiting the screen channels enters a common grit chamber influent channel for redistribution to six grit influent channels and six vortex grit removal chambers. Five grit units are online at maximum design flow (72 mgd [3,150 l/s] each), with one standby unit. Grit removed (airlifted) from the vortex units is classified and washed. Both the removed grit and screenings are mechanically conveyed to container storage for off-site disposal.

Flow exiting the grit chambers enters a common effluent channel that conveys the flow to the Inter-Island Tunnel shaft. The new facility is equipped with an emergency bleedback storage system for capturing any spills over an emergency weir located within the junction chamber. Such flows are directed into storage created using existing influent/effluent conduits.

The preliminary concept design of the headworks (see Figure 3 on page 66) was optimized during the final construction design phase.[1,2] The project was constructed for $70 million and was put into service in October 1998. Hydraulic physical modeling was performed on the design.[3-5]

A concurrent mathematical hydraulic modeling effort was conducted to note hydraulic gradients of the upstream HLS (the immediate 12,000-foot [3,660-meter] section) and through each element of the headworks facility with linkage to a hydraulic surge model of the Inter-Island Tunnel.[2] However, the mathematical model could not describe the complications of vorticity caused by the 90-degree turn in the junction chamber. Therefore, the physical model was used to overcome this deficiency, but it was limited to the inner portions of the headworks. The physical model included the HLS entrance, the junction chamber (and emergency overflow), influent channels to the screens and the channels through the screens. The downstream vortex degritters were not ex-



1. Influent
2. Junction Chamber
3. Distribution Channel
4. Overflow Weir
5. Emergency Flow Storage
6. Transition Section
7. Channel Divider Wall
8. Screens
9. Screening Channel
10. Collection Section
11. Degritter Effluent
12. Vortex Degritter
13. Interim Conduits Converted to Emergency Flow Storage
14. Emergency Spillway
15. Surge Relief Orifices
16. Inter-Island Tunnel Shaft
17. Inter-Island Tunnel
18. Sluice Gates
19. Emergency Flow Storage Wet Well
20. Headworks Effluent Channel to Tunnel Shaft
21 Emergency Outfall

Note: Screen Effluent & Degritter Isolation Gates Not Shown.

FIGURE 2. An overview of the Nut Island Headworks.

plicitly modeled, but the degritter effluent tailwater conditions estimated by the mathematical model were simulated by adjustable weirs. The vortex degritter tailwater conditions became the terminal boundary conditions for the physical model investigations.

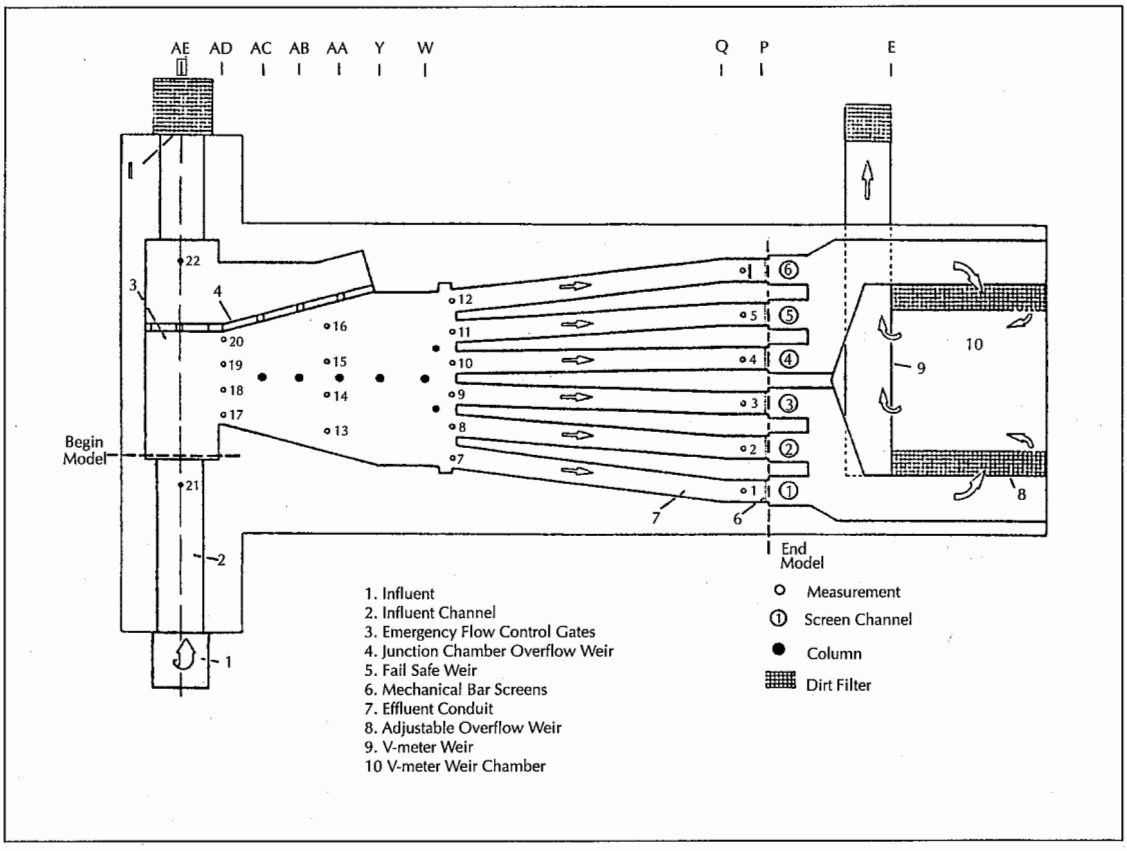


FIGURE 3. A model of the original design concept.

Objectives of Physical Modeling

This task included the construction, operation and optimization of a physical model of the headworks inlet area from immediately upstream of the connection with the HLS to immediately downstream of the six new mechanical screens.

The principal objectives of the modeling effort included confirmation and/or implementation of simple modifications (enhancements) to:

- Ensure the adequacy of flow distribution through the junction chamber to the six new screens;
- Ensure the adequacy of grit distribution through the junction chamber to the two halves of the system — left hand side (LHS) and right hand side (RHS) looking at the headworks from the LHS;
- Ensure that undue grit deposition does not occur for the range of design flows;
- Identify unforeseen hydraulic conditions and develop corrective changes; and,
- Determine capacity of the side-overflow weir under peak flow emergency conditions.

Functional Objectives

These physical modeling objectives were translated into functional objectives.

For dry weather flow conditions (80 mgd [3,500 l/s]), the functional objectives were:

- Two screens in service — one screen on each half of the system (LHS and RHS);
- Adequacy of flow and grit distribution as defined by achieving approximately equal distribution to each of the two channels in service; and,
- No undue grit deposition.

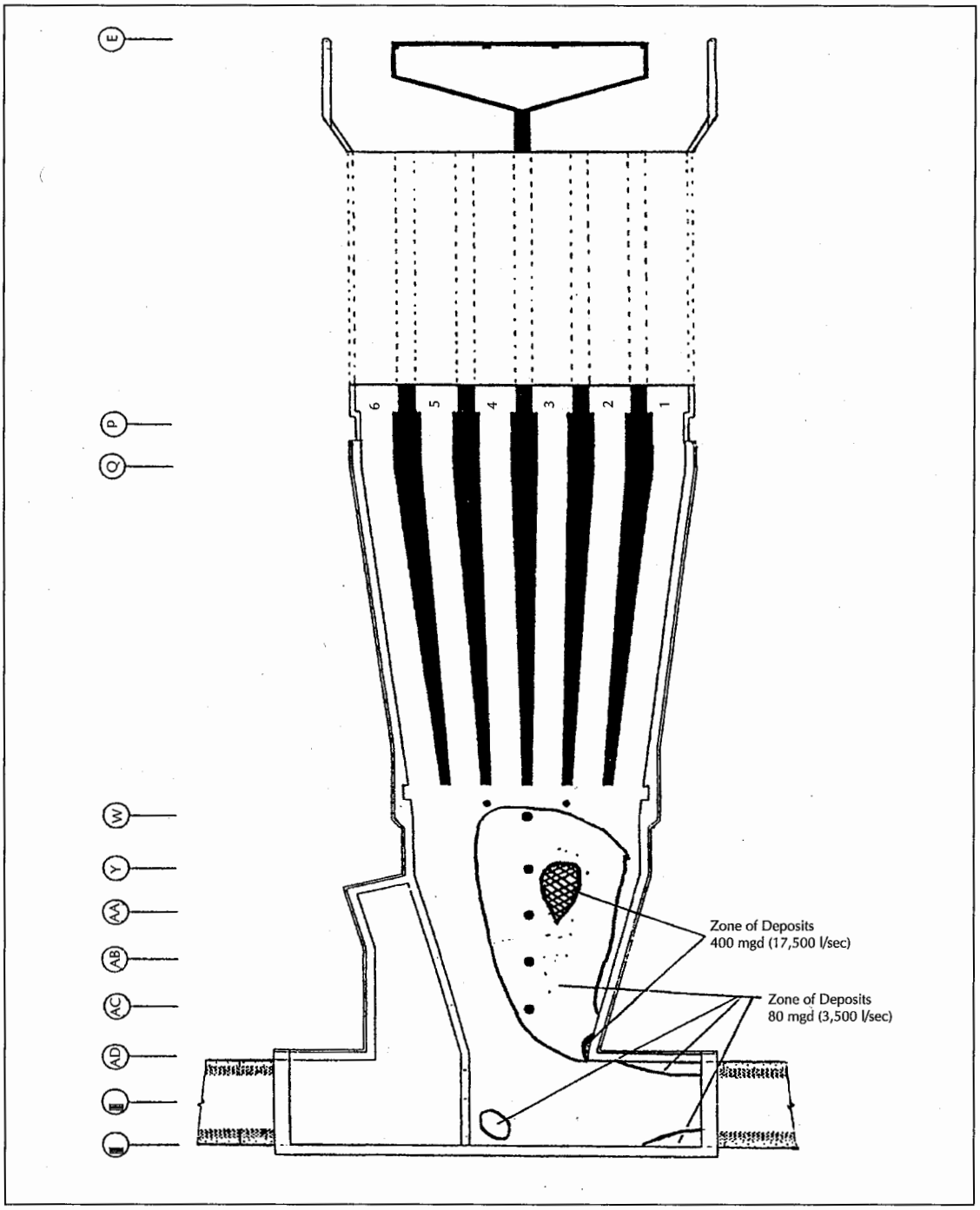


FIGURE 4. Zones of grit deposition (no control, preliminary design concept).

For maximum treatment capacity (360 mgd [15,700 l/s]) and peak hydraulic capacity (400 mgd [17,500 l/s]), the functional objectives were:

- Four screens in service — two screens on each half of the system (LHS and RHS);
- Adequacy of flow distribution as defined by achieving maximum flow less than 110 mgd (4,800 l/s), the maximum preferable hydraulic capacity of screens under nor-

TABLE 1.
Peak Flow Simulations (No Control)

Flow (mgd)	Channels in Service	Dirty or Clean Screen	Min/Max Flow Range (mgd)	Min/Max Flow Range (ratio)
360	1,2,5,6	Clean	125/65	1.9:1
360	1,2,5,6	Dirty	105/75	1.4:1
360	1,3,4,6	Clean	130/75	1.7:1
360	1,3,4,6	Dirty	115/80	1.4:1
360	2,3,4,5	Clean	110/80	1.4:1
360	2,3,4,5	Dirty	105/85	1.2:1
400	1,2,5,6	Clean	135/75	1.6:1
400	1,2,5,6	Dirty	125/85	1.5:1
400	1,3,4,6	Clean	130/65	2.0:1
400	1,3,4,6	Dirty	135/90	1.4:1
400	2,3,4,5	Clean	125/85	1.5:1
400	2,3,4,5	Dirty	125/95	1.3:1

Note: 1 mgd = 43.9 l/s

mal conditions, to any of the four screens in service;

- Adequacy of grit distribution as defined by achieving approximately equal distribution of grit of the two halves of the system (LHS and RHS);
- No undue grit deposition;
- No spillage over the emergency overflow weir at maximum hydraulic capacity; and,
- The HLS does not surcharge.

Construction of the Physical Model

The physical model was constructed, using a combination of plexiglass and laminated plywood, to a geometric scale of 1:10 (Froude similitude scaling). The model simulated the influent junction chamber emergency side overflow weir, transition section to screen influent channels, the screen channels through the six screens and the section leading to the common grit influent channel. The inflow to the model was measured by an inductive flow meter with an accuracy of ±0.5 percent. The water depths were measured at 22 points using piezometer pick-ups. Velocity was measured using a propeller flow meter at the height of the chamber and along the screen influent channels. Simulated gritty material (polystyrene particles) was used for color tracing to visually (qualitatively) define the solids distribution between the LHS and RHS and to identify any particular deposition problems (enhancement to velocity profiles). The gritty material used in the experiments simulated a settling velocity of 1.08 ft/s (33 cm/s), corresponding to very fine gravel with grain diameters from 2 to 4 millimeters. Floating lights with an underwater depth of 4 inches (10 cm) were used to show the middle velocity streamlines for photographs and videos. Styrofoam pieces were used to simulate floatables.

Results From Phase I Modeling

The initial phase of the modeling activities was conducted at three flow regimes: design low flow, maximum treatment capacity and peak hydraulic capacity. Runs were performed at each of the flow regimes for the logical variety of screen combinations (to determine the worst case for each flow regime) and for clean and dirty screen conditions (all screens clean or all screens dirty).

Phase I Findings & Conclusions

The principal findings and conclusions derived from the results of the initial phase of physical modeling activities include:

- Flow patterns in the transition section are non-symmetric for all three flow conditions (80, 360 and 400 mgd). A pronounced main vortex is developed plus secondary eddies.
- Severe sedimentation is identified for all flow conditions in the transition section (see Figure 4 on page 67).
- The system exhibits unequal flow distribution through channels in service, which is more pronounced at maximum and peak design flows (see Table 1 and Figure 5). The difference is lessened as in-service channels change from outside to inside pairs, and as the condition of the screens changes from clean towards dirty. At most maximum and peak flow conditions, the distribution of flow exceeds the peak hydraulic capacity of the screens (110 mgd [4,800 l/s]) for at least one channel.
- Non-uniform flow distribution occurs through the influent channels to the bar screens at peak hydraulic design flow. Velocities through the screen influent channels are adequate at all flow regimes to prevent significant grit deposition.
- Unacceptable spillage occurs over the emergency overflow weir at peak hydraulic flow conditions.
- Enhancements will be required to achieve the stated functional objectives.

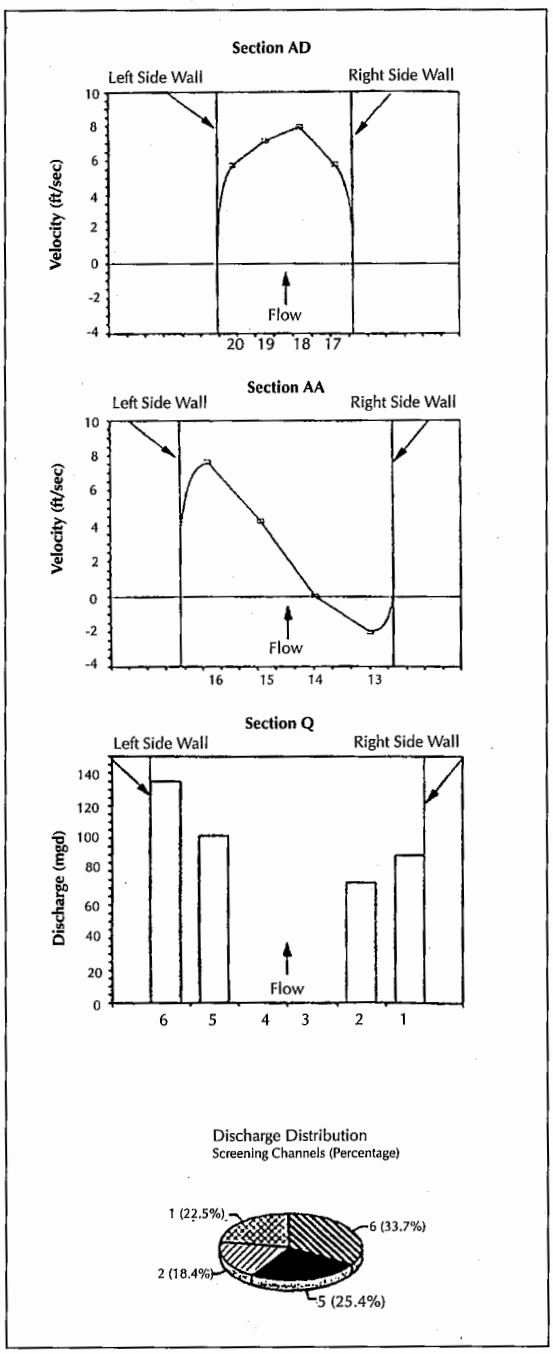


FIGURE 5. Model results for original design concept conditions (400 mgd flow, clean screens and open channels 1,2,5,6).

Recommended Enhancements for Optimization (Phase II)

Subsequent to the review of Phase I results, the following elements were recommended for in-

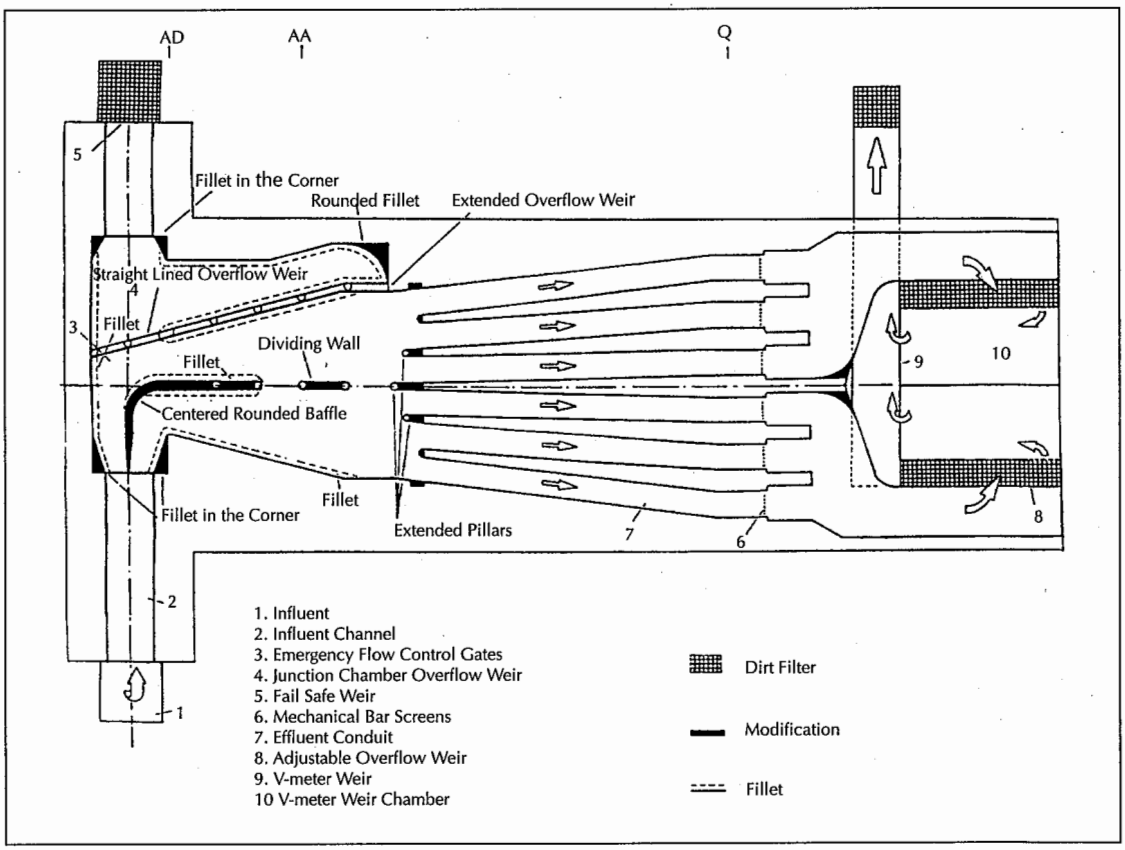


FIGURE 6. Model with optimized improvements.

vestigation in the optimization phase (see Figure 6):

- A single curved flow vane to improve distribution of flow to the transition section;
- A straight line junction to the chamber overflow offering the following advantages: no sedimentation corner under dry weather flow, and spillage over the weir reduced at peak flow;
- Two concrete fillets in the left and right corner at the HLS inlet to mitigate sedimentation;
- Concrete fillets in the corners of the overflow weir chamber to mitigate sedimentation;
- Rounded pillar profiles at the inlet to the influent channels;
- Rounded edges for the contraction section;
- Raise the emergency weir elevation and use rounded weirs and rounded columns between the weir windows to increase discharge capacity;
- Extend the wall dividing channels 3 and 4 to reduce length of stop log assemblies, thereby permitting easy isolation of each half of the system; and,
- Install an asymmetric flow vane near upstream end of eccentric flow vane to improve the distribution of grit between the two halves of the system (ultimately not recommended).

Phase II Optimization Results

The curved entrance deflector wall (see Figure 7) and the other hydraulic enhancements satisfied the functional criteria. Distribution of the original design concept flows and optimized design flows (after physical modeling) under maximum flow (400 mgd [17,500 l/s]) are presented in Figures 5 and 8, respectively, for the worst-case scenario of "open" outer and inner channels under clean screen con-

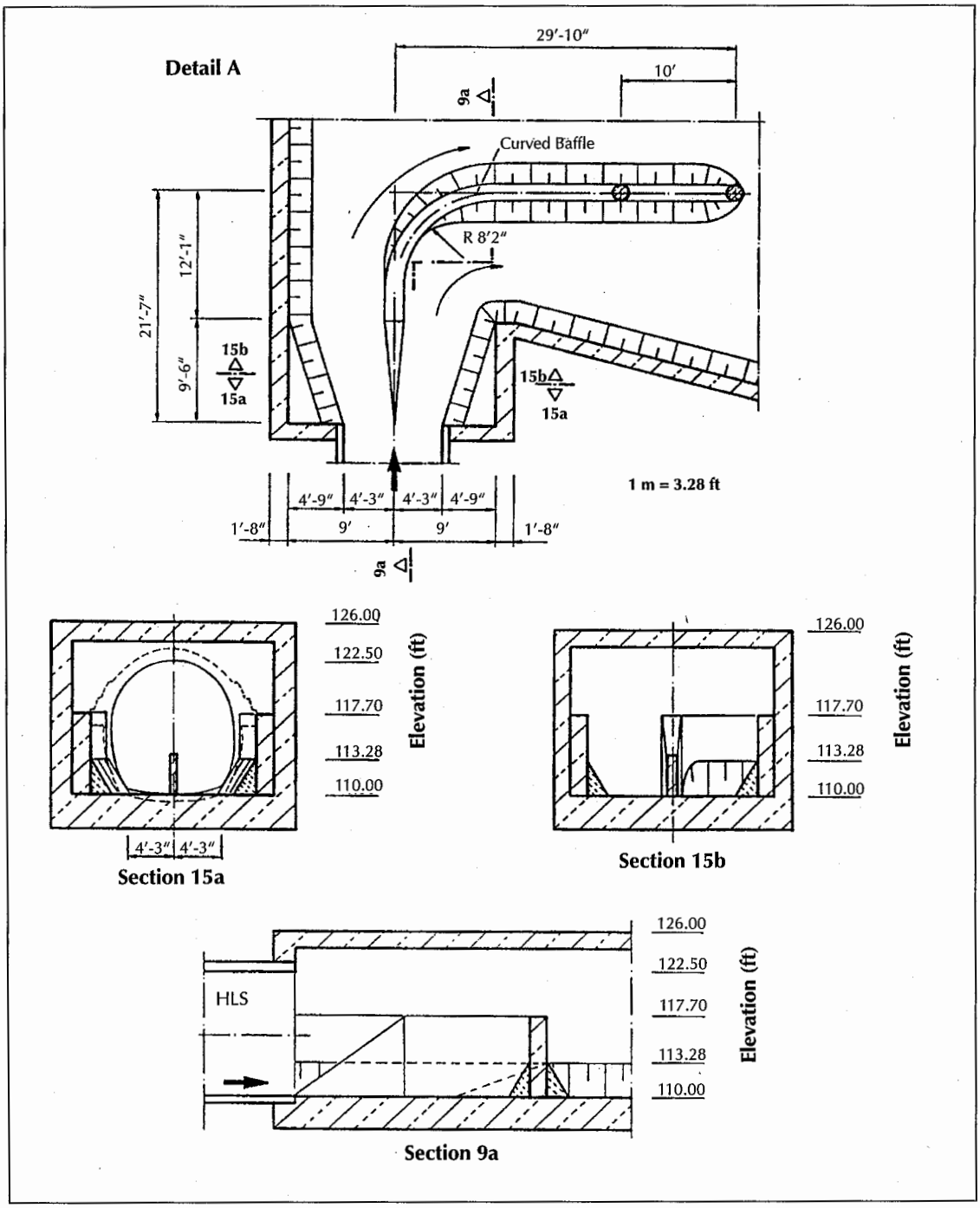


FIGURE 7. Detail of junction chamber (new curved baffle) for the Nut Island Headworks.

ditions. Spillage at the maximum hydraulic flow condition is eliminated. The distribution of floatables and "swimmers" showed acceptable uniformity and sedimentation was minimized for all flow conditions. These recommendations were included in the final design documents.

NOTES — *The preliminary design was optimized during final design by Montgomery Watson, Bos-*

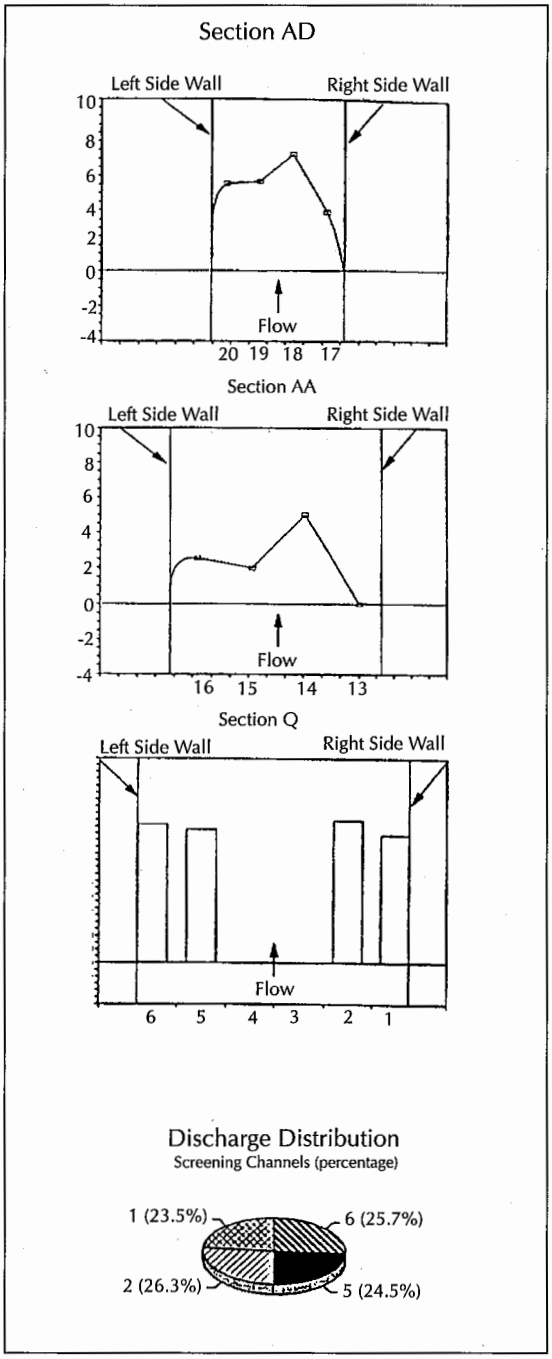


FIGURE 8. Model results for design-modified, optimized conditions (400 mgd flow, clean screens and open channels 1,2,5,6).

ton. Umwelt-und Fluid-Technik, Germany, under the direction of Dr. Hansjörg Brombach, used a hydraulic physical model to check and optimize the design concept. This article represents the opinions and conclusions of the authors and are not necessarily those of the MWRA.

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REFERENCES

1. "Design Package 4, Nut Island Headworks, Conceptual Design Report," Metcalf & Eddy to the MWRA, May 1989.

2. "Updated Final Hydraulic Modeling Report, Nut Island Facility," Havens & Emerson, unpublished, October 1990.

3. "Hydraulic Model Tests for the Nut Island Headworks," Umwelt-und Fluid-Technik, Bad Mergenthiem, Germany, October 1990.

4. Atoulikian, R., Brombach, H., Pisano, W., & Urban, F., "Hydraulische Modellversuche fur ein Klarwerk in Boston/USA," *Wasserwirtschaft*, February 1993.

5. Pisano, W., Brombach H., Gallinaro, G., Atoulikian, R., Basista, J., & Papes, T., "Physical Modeling to Enhance Nut Island Headworks Design," 66th Annual WEF Conference, Anaheim, Calif., October 1993.

What Has the Finite Element Method Done for (or to) Geotechnical Engineering?

The three articles that follow are based on presentations at the ASCE National Convention in Boston in October 1998 by experts on the development and use of finite element methods in geotechnical engineering.

JOHN T. CHRISTIAN

The finite element method grew out of the aircraft industry, which needed precise computational methods to avoid excessive weight in its products. Before the 1960s most geotechnical engineers — researchers as well as practitioners — had been reluctant to employ numerical methods and computers because the techniques available were not powerful enough to deal with the complicated geometries and nonlinear material properties that occur in practice. Most of the first users of finite element methods were also programmers. Effective use of finite element methods often required that the analyses be done by the programmer. At that time, computers were awkward. They had to be fed with decks of punched cards at inconvenient times and places. Output usually consisted of many sheets of printed paper, which presented another data-processing problem.

As computers became more powerful and more widely available, this view changed. If there is a single point at which the finite element method became a respectable part of geotechnical engineering, it was Clough and Woodward's description of simulating the construction of an earth dam by finite elements and recovering the observed deformations.[1] There followed an explosion of applications. Soon, the technique gained a strong position among the tools available to the geotechnical profession so that today there is a large body of experience in the use of numerical methods for geotechnical problems. Engineers have developed effective ways to deal with many situations and have also learned that some problems remain intractable.

Current Finite Element Capabilities

As computers have become more accessible and easier to use, so too has finite element software. The engineer can obtain a great variety of modern, user-friendly, finite element software suitable for geotechnical applications. First, there are several systems designed specifically for geotechnical engineering. Such packages

include PLAXYS, CRISP and SIGMA, as well as FLAC (which is actually a finite difference code). These programs include features unique to geotechnical needs, such as Modified Cam-Clay constitutive models, effective stress analyses, and so on. A second group of widely used programs consists of general-purpose, nonlinear finite element systems such as ABAQUS, ANSYS, ADINA and SAP. While these programs were usually written for structural or mechanical applications, they permit the user to incorporate additional routines to deal with peculiarly geotechnical issues. Because of their large capacities and versatility, they are the systems of choice for very large problems with many degrees of freedom or steps of loading. Third, there are the special-purpose programs, such as those that handle dynamic problems or simulation of tied back excavations. Since these systems are intended for specific and limited purposes, their user interfaces are usually less sophisticated than those in the first or second groups. Also, they tend to be more idiosyncratic and buggy.

Considerations

The geotechnical organization contemplating acquiring finite element capabilities can expect that most of the available packages will run well on a modern personal computer. The cost of licenses varies among the different packages, and each system is licensed with a different set of options among the software units. Nevertheless, including the cost of hardware, a satisfactory finite element system can be set up today for approximately $10,000.

However, far larger investment in learning to use the system effectively is required. These systems have learning curves — some are quite steep. Furthermore, if one does not use the system for a period of time, one forgets what has been learned and has to climb back up the learning curve when the next job comes along. Climbing a learning curve the second time can be more tiresome and frustrating than it was the first time. Organizations must be prepared to invest not only in the hardware and software but also in personnel resources to keep skills current.

Finally, as the speakers at the session emphasized, intelligent interpretation of the results is absolutely critical. The most important person in the finite element team is the person who can explain what the results mean. There is a statement, often attributed to John von Neumann, but actually included as the motto of Hamming's classic book on numerical methods: "The purpose of computing is insight, not numbers."[2]

ACKNOWLEDGMENTS & NOTES — *While the session "What Has the Finite Element Method Done for (or to) Geotechnical Engineering?" (held at the ASCE National Convention in Boston in October 1998) was planned as an informal exchange of views, several persons in the audience suggested that, since it provoked a lively discussion, the principal speakers' comments should be collected in* Civil Engineering Practice. *Even though discussion was limited to the finite element method, other numerical techniques — such as finite difference, boundary element and discrete element methods — have found useful application in geotechnical engineering. Many of the insights found in the following three articles also apply to those methods. Professor Stephen G. Wright of the University of Texas, Austin, former chairman of the Geo-Institute Committee on Computer Applications, first suggested the topic for the session, and Brian Brenner encouraged the authors to prepare the written versions of their remarks.*

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REFERENCES

1. Clough, R.W., & Woodward, R.J., III, "Analysis of Embankment Stresses and Deformations," *Journal of the Soil Mechanics and Foundations Division, ASCE*, Vol. 93, No. SM4, 1967.

2. Hamming, R.W., *Numerical Methods for Scientists and Engineers*, New York, McGraw-Hill, Dover Books, 1973.

Applying the Finite Element Method to Practical Use in Geotechnical Engineering

Proper use of the finite element method does not mean employing it as an end in itself; its application still requires data collection and sound engineering judgment.

J. MICHAEL DUNCAN

The finite element method has freed geotechnical engineers from having to base their analyses on oversimplified soil properties and boundary conditions. The method has removed many limitations from everyday analyses, making it possible to analyze:

- Stress and deformation (with nonlinear soil properties);
- Soil-structure interaction (modeling construction sequence);
- Seepage (modeling hydraulic properties as well as they can be known); and,
- Consolidation (with two-dimensional drainage and nonlinear behavior).

However, the finite element method has not freed geotechnical engineers from the necessity to understand geology and site conditions well, to measure soil and rock properties with care, to understand the phenomena that control behavior and to take construction processes into account in a realistic way. No method of analysis can eliminate the need to do those things. Thus, the finite element method has not affected the basic requirements for good geotechnical engineering. Where it has been used without attention to these fundamentals, it has produced nothing of value and has served only to submerge real problems beneath a sea of largely irrelevant computer output. A brief examination of three examples where the finite element method has been particularly useful can help in establishing its proper application.

Port Allen Lock

Port Allen Lock was one of the first applications of the finite element method to soil-structure interaction (see Figure 1).[1] The lock

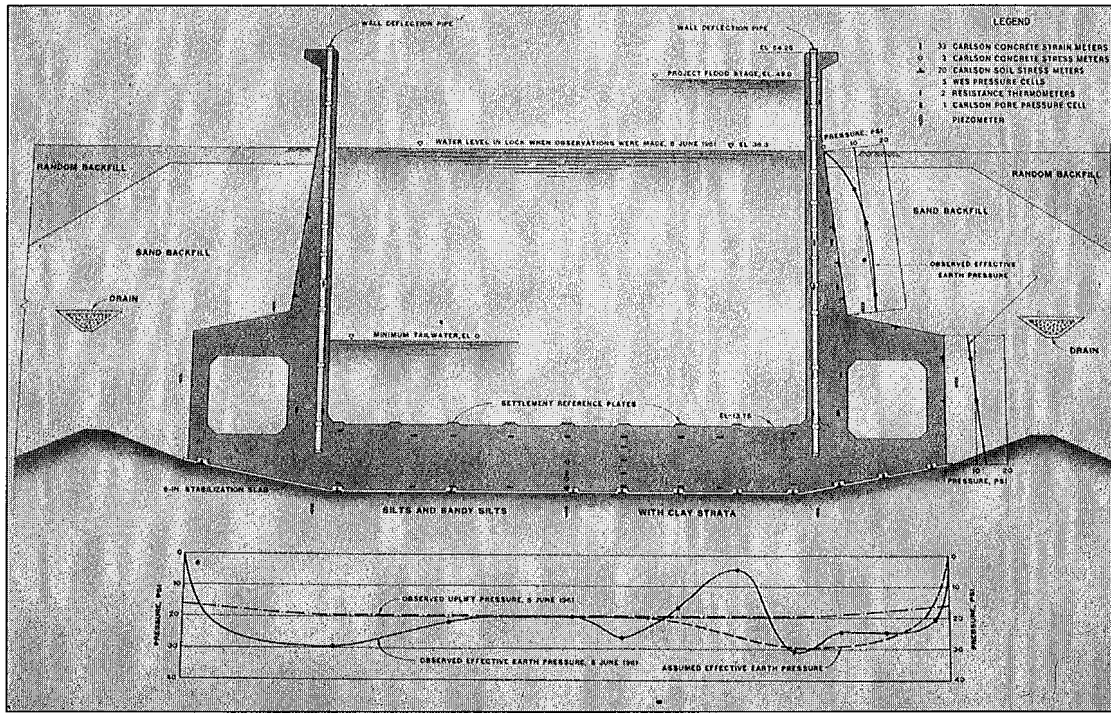

FIGURE 1. A cross-section view of the Port Allen Lock, with a base pressure profile below.

chamber is about 75 feet wide by 75 feet high by 1,100 feet long. The soils at the site were so deep that conventional pile-supported lock walls were not suitable; instead, an integral U-frame lock chamber was used. The structure was designed and built in the late 1950s, before the finite element method was developed. Because it was so well instrumented, it provided an excellent opportunity to compare results of finite element analyses to actual field behavior in a complex soil-structure interaction problem.

The base slab is about 10-foot thick, heavily reinforced concrete, poured in three lifts. Construction of the slab was modeled three ways (see Figure 2):

- *One lift-stiff slab:* The calculated deflections were smaller than those measured.
- *One lift-dense liquid:* The calculated deflections were larger than those measured.
- *Two lifts:* Dense liquid when placed, stiff after placement. The calculated and measured deflections were in good agreement.

The example of the Port Allen Lock illustrates the importance of simulating closely what actually happens in the field to achieve useful results.

New Melones Dam

The New Melones Dam, shown under construction in Figure 3 (on page 78), was designed by the Sacramento District of the Corps of Engineers. The objective of the analysis was to predict how much movement would occur during construction of this 670-foot high central core rockfill dam. By comparing measured and computed movements as the dam was built, it was possible to determine whether the dam was performing within a "normal" range of behavior.[2]

Two analyses were performed, using stiff and not-so-stiff properties for the core and the shell. The movements measured in the field compared well with the results of the analysis performed using stiff properties. The second analysis indicated movements on the order of 50 percent larger. Figure 4 (on page 78) shows a comparison between measured and computed cross-valley movements. Comparisons made throughout construction indicated very clearly that the field behavior was at the favorable end of the expected range.

Dam 2 Powerhouse on the Arkansas River

Another step forward provided by the finite element method is the ability to analyze conditions where there is some, but not complete drainage (some dissipation of excess pore pressures, but not complete dissipation of excess pore pressures). When coupled finite element analyses are used, there is no need to define the problem as completely drained or completely undrained, a simplification that is necessary in many other types of analyses.

Figure 5 (on page 79) shows the excavation for the Dam 2 Powerhouse on the Arkansas River. The headrace channel walls range from 0 to about 90 feet high, the powerhouse walls are about 90 feet high and the tailrace channel walls range from 0 to about 90 feet high. All of the walls visible in Figure 5 are concrete diaphragm walls cast in slurry-filled trenches. The headrace and tailrace walls were supported by passive pressure at the bottom and by a single line of anchors tied to massive deadman structures at the top. The powerhouse walls were supported by six rows of prestressed grouted anchors during excavation and by the powerhouse structure after it was completed. The site was surrounded by a plastic concrete cutoff wall to control seepage and make dewatering possible.

Movements of the powerhouse walls during construction were analyzed using the finite element method.[3] The analyses were performed using a new finite element program called SAGE.

Figure 6 (on page 79) displays the effective stresses and pore pressures at the end of powerhouse excavation in December 1995. There is a zone beneath the powerhouse where negative pore pressures developed during excavation have not yet dissipated. A discontinuity in pore pressure contours can be seen at the right side of Figure 6 at the location of the seepage cutoff.

Figure 7 (on page 80) shows conditions about three years later, in September 1998. Groundwater had reached essentially a steady flow condition and the negative pore pressures that developed during excavation had dissipated.

What Has Changed?

The finite element method, and computers in general, provide analysis capabilities that were completely unavailable before 1960. Without those capabilities, geotechnical engineering analyses were often very time-consuming undertakings.

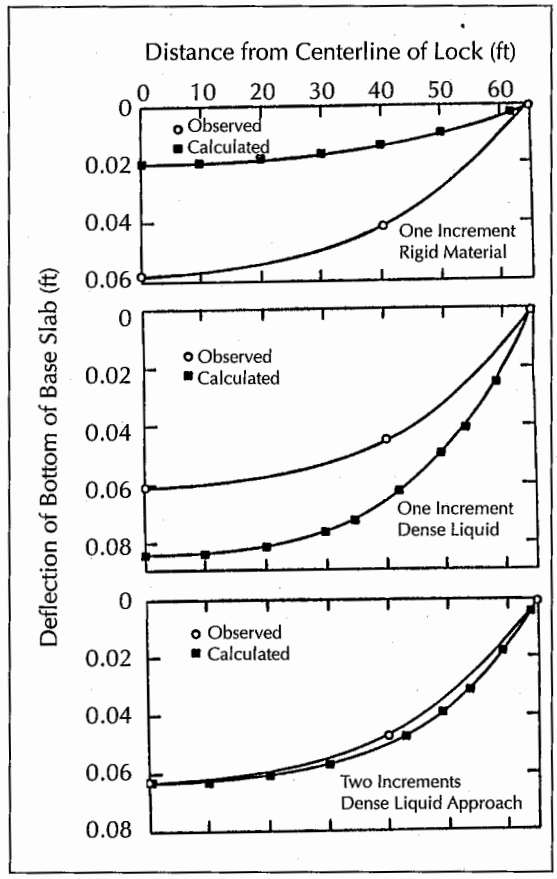

FIGURE 2. Three models for slab construction (Port Allen Lock).

A review of the studies Karl Terzaghi made of the settlements of the Long Beach Steam Power Station on Terminal Island presents a good example of how sound geotechnical engineering analyses used to be attained. That project was very important (a major power plant was in the process of settling 25 feet) and it was difficult (neither values of compressibility nor boundary conditions were easily determined), and there were major difficulties in estimating the magnitudes of the settlements that would ultimately occur.

It is interesting to see, in Terzaghi's correspondence with his client, how the difficulties of performing analyses limited what could be accomplished in the way of predicting the

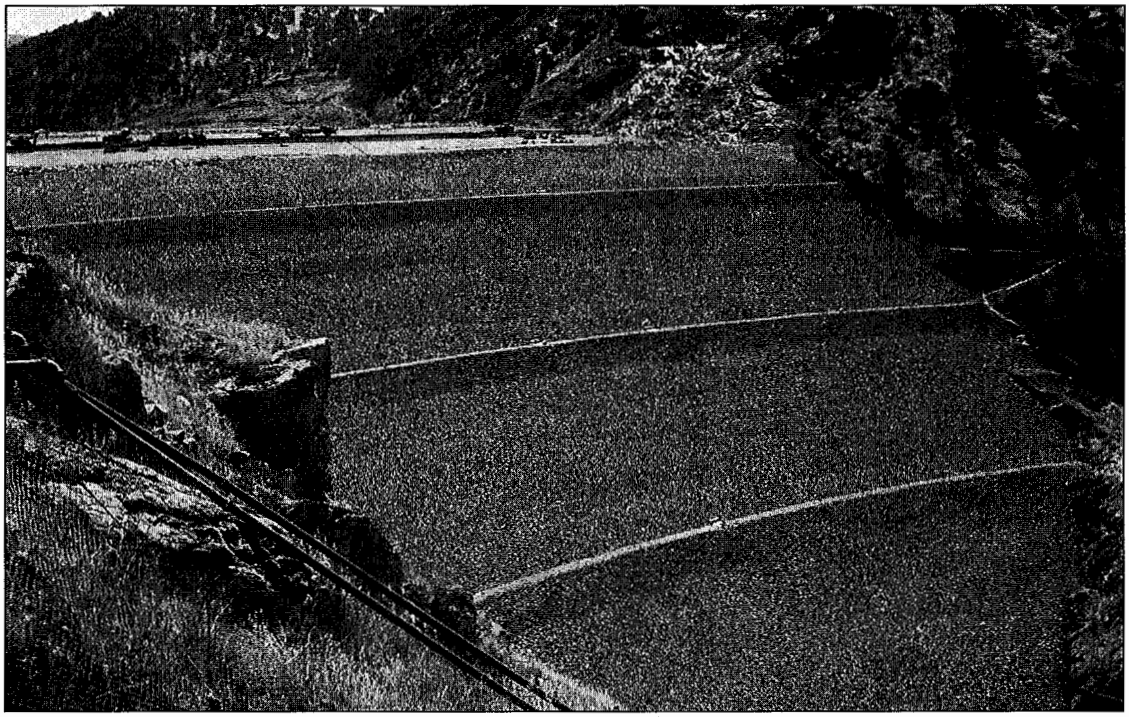

FIGURE 3. The New Melones Dam under construction.

course of future settlements. In one letter, he discussed the computations he wanted done in order to compare measured settlements with the settlements computed using various assumptions regarding compressibility and time lag. Terzaghi indicated that those calculations would take about two months. Needless to say, those computations could be done much more quickly today. Today, many engineers would approach this type of problem by performing parametric finite element analyses to match field observations and gain better insight on mechanisms. In 1958, however, such an approach was not possible.

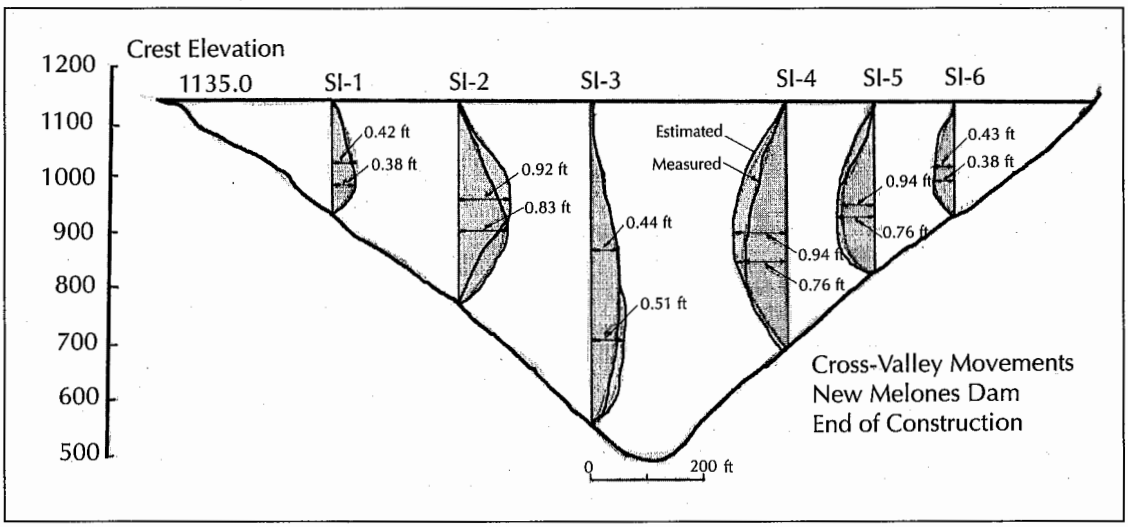

FIGURE 4. A comparison between measured and computed cross-valley movements.

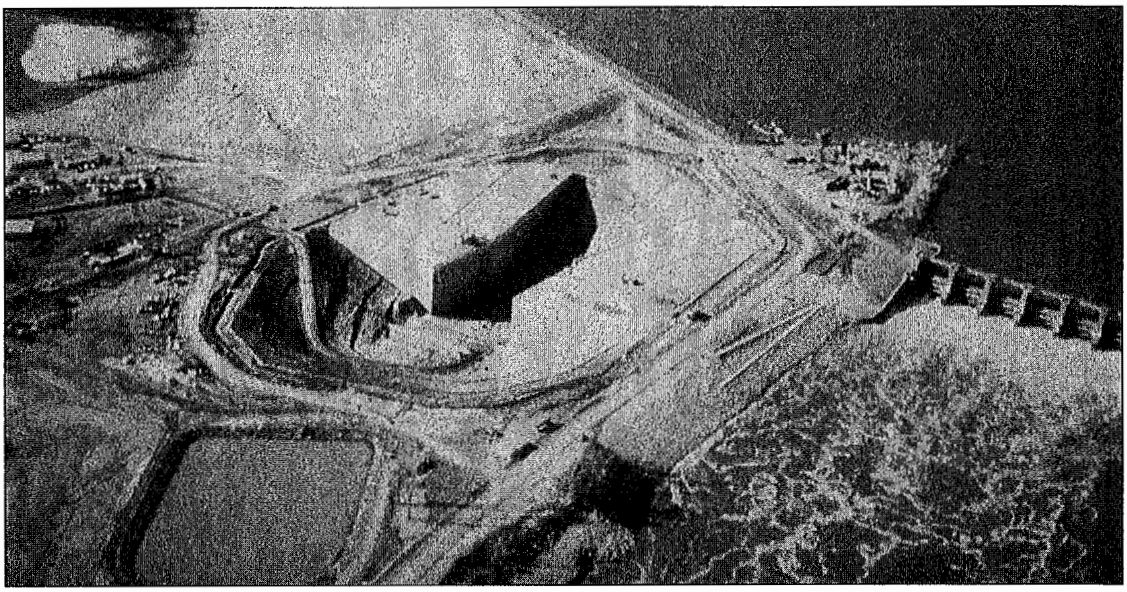

FIGURE 5. Dam 2 Powerhouse excavation.

The finite element method has done a great deal for geotechnical engineering. Before the finite element method was available, engineering understanding of site conditions and phenomena often exceeded the ability to perform analyses, as in the case of the Terminal Island Power Station. Now, the ability to perform analyses usually exceeds the ability to define site conditions.

The amount of judgment and understanding required for good geotechnical engineering is exactly the same as it was before the finite element method was developed. The finite element method does not replace judgment or un-

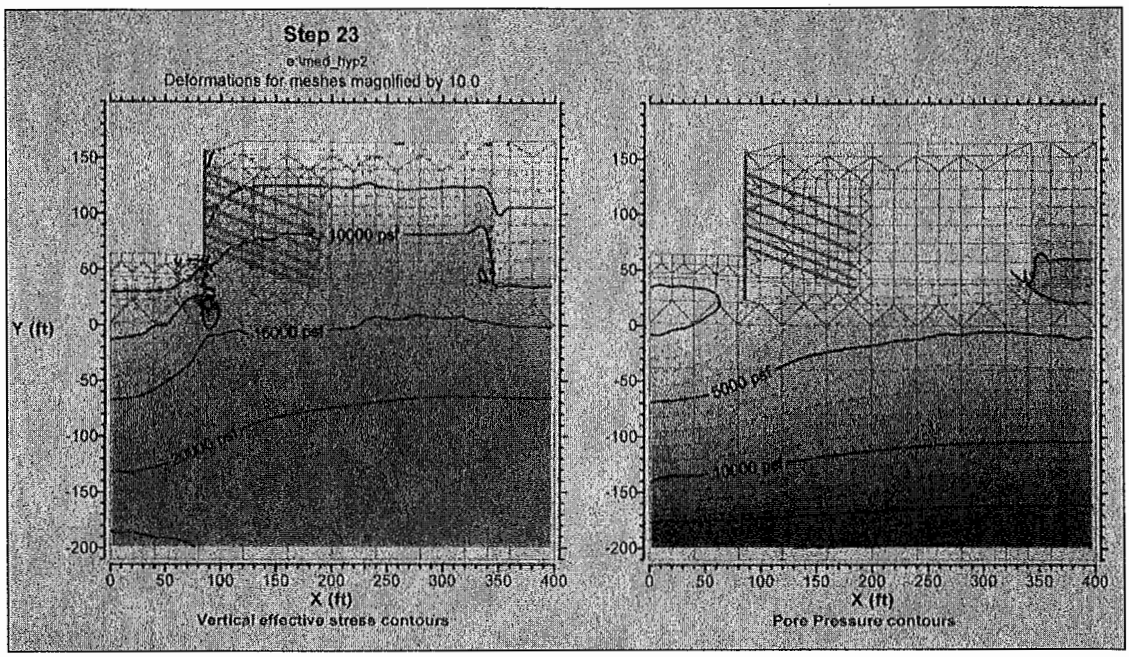

FIGURE 6. Effective stresses and pore pressures at the end of the powerhouse excavation.

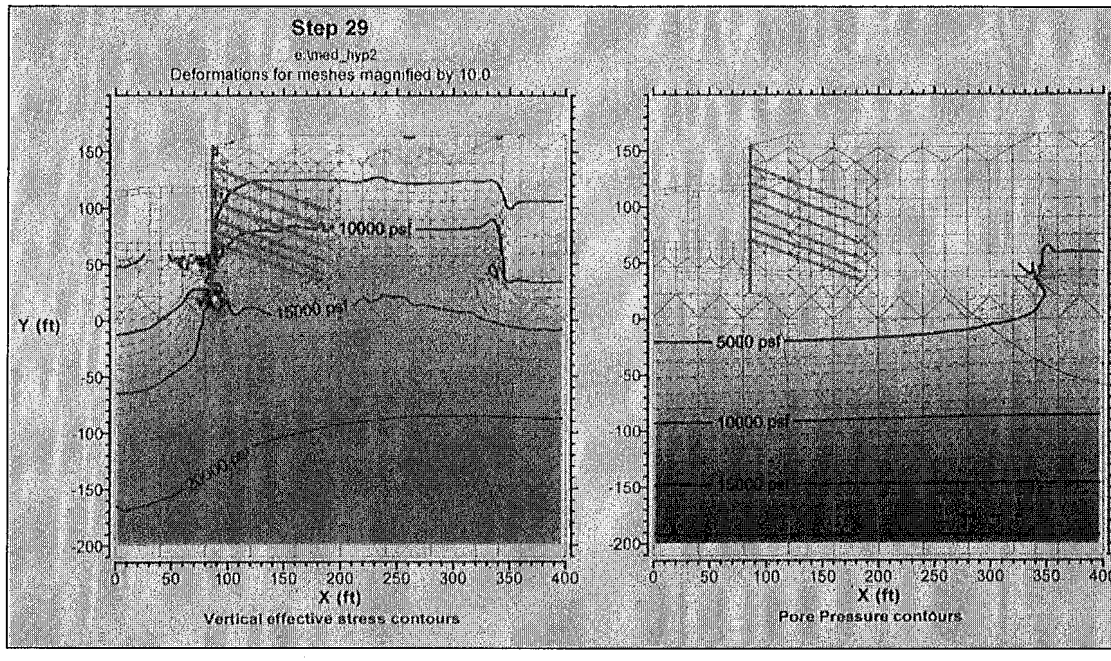

FIGURE 7. Effective stresses and pore pressures about three years after powerhouse excavation.

derstanding of geology, phenomena or properties. However, there is no doubt that when essential judgment and understanding is supplemented by appropriate finite element analyses that are appropriately interpreted geotechnical engineering capability and practice are enhanced.

Of course, the finite element method also can be misused. If finite element analyses become a substitute for data, understanding or judgment; if performing the analyses becomes an end in itself; or, if blind trust is placed in black-box computer programs, the result is unlikely to be useful, and more likely to be misleading and dangerous. However, where the method is used appropriately by capable geotechnical engineers, it has the potential for adding considerable value in many types of geotechnical engineering studies.

NOTE — *This article is based on a presentation at a technical session entitled, "What Has the Finite Element Method Done for (or to) Geotechnical Engineering?" held at the ASCE National Convention in Boston in October 1998.*

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REFERENCES

1. Duncan, J.M., & Clough, G.W., "Finite Element Analyses of Port Allen Lock," *Journal of the Soil Mechanics and Foundations Division, ASCE,* Vol. 97, No. SM8, August 1971.

2. Quigley, D.W., Duncan, J.M., Caronna, S., Moroux, P.J., & Chang, C.S., "Three-Dimensional Finite Element Analyses of New Melones Dam," *Geotechnical Engineering Report No. TE 76-1,* University of California, Berkeley, December 1976.

3. Bentler, D.J., & Duncan, J.M., "Finite Element Analysis of Deep Excavations," *Geotechnical Engineering Research Report,* Virginia Tech, 1998.

The Role of Finite Element Methods in Geotechnical Engineering

Greater computational power and higher accuracy software have led to wider acceptance of finite element analyses within the profession of geotechnical engineering over the past two decades.

ANDREW J. WHITTLE

While the use of finite element analyses is so ubiquitous in many branches of engineering, and even though geotechnical engineers were prominent among the early developers of finite element methods,[1,2] there is considerable skepticism about their usefulness and reliability in solving practical geotechnical problems. One can contrast this situation with their almost universal acceptance in structural engineering design. Cautious use of sophisticated numerical analyses is certainly warranted in geotechnical practice given the spatial variability and complex properties of geomaterials (soils and rocks) and the limited site characterization data available in most projects. Furthermore, there is often extensive experience with simplified empirical methods that appear to provide an adequate basis for safe design.

Apart from these general constraints, the technology of finite element analyses has matured during the last 20 to 25 years, and it has become readily available through advances in user-friendly interfaces and powerful personal computers (PCs). During the 1970s, university research groups carried out much of the development and application of prototype finite element codes (local examples at the Massachusetts Institute of Technology [MIT] included FEAST for stress analyses, FEDAR for steady flow and CONSOL for uncoupled consolidation). Programs originally written for card reader input devices have achieved remarkable durability and have formed the kernels of numerous programs that are still used in practice today. However, the user community today is very different and can be broadly sub-divided into two groups:

- The first are experienced engineers who are familiar with their design problems, who recognize limitations in finite element codes and adjust the use and interpretation of these numerical analyses accordingly.
- The second are junior engineers, who are primary users of the much more powerful second-generation finite element software. They are highly literate in current computer technology, but tend to have very limited knowledge of the underlying formulation and numerical approximations used in finite element methods (these are not topics that are routinely covered in graduate courses in geotechnical engineering) or conversely of soil/rock properties and behavior (topics that are often unfamiliar to students with strong mechanics backgrounds).

Successful application of finite element analyses in geotechnical engineering requires a clear understanding of the design problem/parameters to be solved (based on sound knowledge of the geological context, soil/rock mechanics, groundwater hydrology, etc.) *and* an understanding of the capabilities and limitations of finite element modeling techniques.

Current Program Capabilities

There is some excellent software available for performing finite element analyses of geotechnical engineering problems. One can distinguish three main classes of software:

- Very big, general-purpose finite element codes that include subroutines specifically for geotechnical applications (constitutive models for soils, etc.). Examples include ABAQUS, ADINA and ANSYS. Among these programs, ABAQUS was one of the first to include capabilities for modeling coupled flow and deformation in soils (*i.e.*, capability for modeling effective stress behavior with partial drainage of pore water), and it was the first general-purpose program to include critical state models of soil behavior such as Modified Cam Clay (MCC).[3] Despite these features, the complexity of the program represents a formidable challenge to the user, requiring a long learning curve (several months of training) and an extensive background in mechanics. The difficulties associated with input and output controls can be made more manageable through separate pre- and post-processor programs.
- General-purpose, geotechnical finite element programs are a more recent and welcome development and have been specifically designed for the geotechnical engineering community. Examples include CRISP, FLAC and PLAXIS. (FLAC is actually a finite difference program. Although this formulation entails different numerical and discretization issues, the scope and objectives of the program are consistent with other finite element programs in this class.) These programs were developed for the PC user, and have benefited from the enormous advances in PC software and hardware capabilities, including very convenient features (such as vastly improved operating system graphical interfaces, etc.). The programs are general purpose in the sense that they can tackle a full range of geotechnical applications ranging from foundations (shallow and deep), to earth structures, to excavations and tunnels. These programs also are able to solve problems involving coupled deformation and flow. They typically incorporate nonlinear constitutive laws such as simple cap models (such as MCC, etc.) and variants of the hyperbolic Duncan-Chang shear stress-strain relations.[4]
- Finite element software for specialized or more advanced geotechnical applications (such as ground amplification of earthquake base motions, liquefaction analyses or advanced constitutive modeling) is still being developed primarily within the domain of university research groups.

From a user's perspective, some of the most important advances in finite element programs (especially those such as CRISP, FLAC and PLAXIS) are linked to the automation of previously time-consuming (and technically difficult) tasks such as mesh generation and load

stepping schemes for nonlinear analyses. These programs have greatly improved efficiency and have brought complex, nonlinear analyses within the reach of most practicing geotechnical engineers.

Automation of load-stepping is a very important topic since it shifts the control of solution accuracy from the user to the program itself. For example, programs such as ABAQUS and PLAXIS have default parameters governing the accuracy (error tolerance) within their nonlinear equation solvers (both of these codes use iterative Newton-Raphson solvers that are unconditionally stable). Although users can override some of these parameters, the clear intention is to guarantee solution accuracy to the user and, hence, provide a reliable black-box solving capability for use in design practice. Built-in components of the finite element codes are usually well written and robust.

However, several of the current programs have adopted an open architecture that enables plug-in type modules (subroutines or functions) to be added by the user (examples include FISH functions within FLAC, and UMAT subroutines in ABAQUS). These features are particularly attractive to researchers or advanced users of the codes. However, they represent a major liability in geotechnical practice, where poorly documented or minimally tested subroutines can be used within an otherwise sound code. Recent experience suggests that this type of code customization is quite commonplace but is often marketed under the original product label. Geotechnical practitioners should either require much higher standards for documenting custom features and benchmarking the analyses, or adopt programs with a closed architecture and accept the limitations imposed within these codes.

One should recognize the intrinsic complexity of finite element programs used in geotechnical engineering. The analysis of elastic materials or steady flow in porous media represents a much simpler class of problem than those related to nonlinear soil behavior, transient and coupled flow-deformation, etc. This difference may explain the long time lag between the establishment of finite element capabilities for solving very complex dynamic problems of interactions between three-dimensional structures and elastic soils (*e.g.*, SASSI[5]) or massive groundwater simulations (*e.g.*, MODFLOW[6]), compared to nonlinear effective stress in geotechnical engineering (or contaminant transport models in geoenvironmental engineering).

Although many of the available finite element programs are capable of performing calculations for three-dimensional geometries, the vast majority of applications in geotechnical practice to date have focused on two-dimensional (usually plane strain) conditions. This emphasis is especially true when considering nonlinear soil behavior and/or coupled flow-deformation problems. This situation is about to change. The current generation of PCs have achieved computational speeds that make three-dimensional analyses practical, while sophisticated pre- and post-processing programs (such as PATRAN, IDEAS and FEMAP) have greatly simplified finite element model construction and interpretation. However, one should not underestimate the difficulty of performing numerically accurate nonlinear analyses of three-dimensional problems (*cf.* automated time-stepping algorithms in two dimensions), given the fact that the complexities of constitutive behavior and the representation of spatial distributions of soils are important topics that require further research.

Some Limitations

Finite element analyses provide a powerful tool that can be used in a variety of different modes ranging from simulation (understanding mechanisms of behavior, effects of individual parameters, etc.) to prediction (usually in calculations of deformations, flow or stability) and design (especially structural design, comparison of construction schemes, etc.). However, in all cases, the user needs to pay careful attention to potential limitations in the geotechnical engineering context. Many of these limitations can be traced to three sources:

- Complexity of ground conditions or inadequate site characterization;
- Complexity of geomaterial behavior or inadequate data for appropriate constitutive models; and,

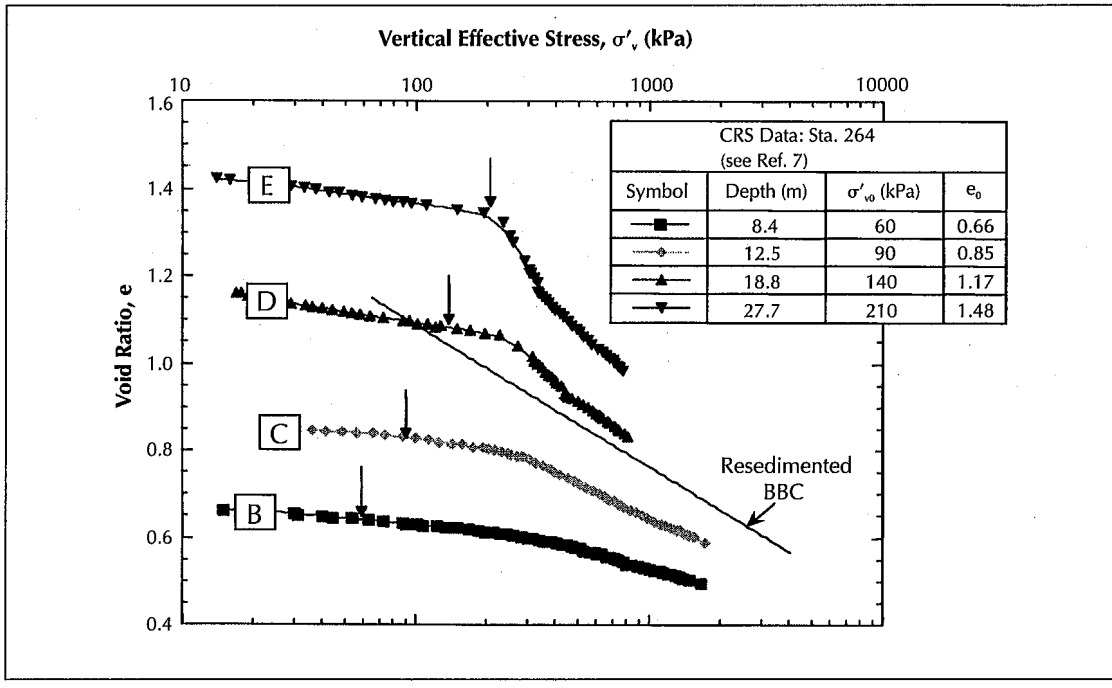

FIGURE 1. One-dimensional compression behavior of a clay.

- Complexity of construction processes that are difficult to represent in finite element models.

Complexity of Ground Conditions or Inadequate Site Characterization. All finite element models are built on simplified models of stratigraphy and/or geological units. Good site characterization data are essential for locating critical/weak soil layers, continuity of drainage horizons, boundary characteristics (usually rock-soil interfaces and rock mass characteristics), and so on. However, spatial variability (*e.g.*, complex layering of alluvial sediments) and macroscopic fabric features (*e.g.*, fissuring of overconsolidated clays masses, voids in karst topography) can represent a formidable challenge. These issues lie outside the spectrum of conventional problems and may either invalidate use of finite element analyses or require stochastic or other specialized modeling techniques (joint elements, etc.) that are best handled by specialists. Jointed rock masses represent perhaps the most extreme case, where the continuum behavior of the intact rock material plays a secondary role to the mechanical and hydraulic properties of the joint system(s).

Complexity of Geomaterial Behavior or Inadequate Data for Appropriate Constitutive Models. There is no doubt that soils exhibit complex material behavior. Even a well studied material such as Boston Blue Clay exhibits features such as undrained strength anisotropy, nonlinear stiffness even at shear strain levels below 10^{-3} percent, stress history and compressibility parameters that vary widely.

For example, Figure 1 shows four one-dimensional compression tests on specimens from different depths within a deep clay layer (from a site north of Boston) — note that specimen E is from the deepest part of the profile. This specimen has the highest in-situ void ratio, and is more sensitive than overlying material with very high compressibility for loading beyond the pre-consolidation pressure (σ'_p).

The diversity of natural soils and rocks as well as other constructed geomaterials (compacted fills, soil-cement mixes) are the basis for the continued existence of geotechnical engineering as a specialty within civil or environmental engineering. While the engineering properties of some materials can be measured through procedures of sampling and laboratory testing, other materials are sufficiently

non-homogeneous or difficult to sample that mass parameters must be estimated by other (usually much cruder) field tests. The net effect is that finite element analyses are often based on very crude constitutive models. The majority of analyses assume linear, elastic deformation properties, while shear strength is characterized by a linear Mohr-Coulomb failure envelope. Even relatively modest advances such as the nonlinear Hoek-Brown failure criterion for rock masses are used comparatively rarely.[8] Most of the current finite element programs include a version of the hyperbolic shear stress-strain law originally introduced by Duncan and Chang.[4] This law has the abiding virtue that it can represent shear behavior for a wide range of materials. MCC remains the most widely used effective stress model and is useful primarily for soft clays.[3]

There remains an important role for the application of more advanced constitutive models,[9,10] especially if more reliable predictions of performance are required (*e.g.*, effects of excavation-induced ground deformations on adjacent structures). However, the additional complexity of these models must be balanced by an equal attention to improved site characterization and measurement of soil properties.

Complexity of Construction Processes That Are Difficult to Represent in Finite Element Models. It is well recognized that there are many construction activities that can cause "disturbance" (*i.e.*, deformations, changes in stresses and soil properties) of the surrounding ground. Examples include the installation of piles, support walls (ranging from driven sheet piles to excavated diaphragm wall panels), tieback anchors (often with multiple grout sequences), as well as grouting or other soil-mixing techniques of ground modification. In some cases (such as the driving of piles in soft clay), these disturbance effects dominate the subsequent pile response to applied loads (due to the generation of excess pore pressures during installation, subsequent dissipation and setup of effective stresses and modification of soil properties close to the shaft) and cannot be ignored in a finite element model. In other cases (such as diaphragm wall installation), the effects on soil stresses and properties are more subtle, and may legitimately be ignored in a finite element analysis (several research groups are currently investigating this topic), although the resulting ground movements may represent a significant fraction of the movements caused by excavation.[11]

Despite these limitations, there is no doubt that finite element analyses can be applied successfully to a wide range of practical problems. Indeed, it is perhaps surprising that they have had relatively modest impact on design methods. This situation seems to reflect other, more general aspects of geotechnical engineering practice.

For example, one should ask why certain empirical methods of design are so widely accepted and/or slow to evolve? One good example is the use of apparent pressure diagrams to design struts for braced excavations. Apparent pressure diagrams were originally derived from measurements of strut loads obtained during construction of subways in Berlin and New York in the 1930s. The current design envelopes (see Figure 2) have evolved using data from projects in several other cities (Chicago, Oslo, London, Washington, etc.). Although the underlying database is substantial, there are many factors that are not explicitly considered in the design charts, such as bending stiffness of the wall (very few of the case studies used high stiffness diaphragm and secant pile walls), levels of strut pre-stress, effects of dewatering, construction sequence, and so forth, while soil profile appears only through the designated ground classification (clay, sand, etc.). For design situations where the bracing system is a temporary structure, there are considerable incentives to reduce the number of struts. Finite element analyses offer an attractive alternative method of design that can handle site-specific ground conditions, proposed construction sequences, and so on, and were first applied for this purpose by Goldberg *et al.*[13] (In this application, the predictions can even be calibrated through observational approaches.) Many subsequent studies have shown the advantages of using finite element analyses for designing strut loads. However, geotechnical consultants most frequently use apparent earth pressure envelopes to design strut loads.

A second general observation is that the geotechnical profession undervalues the impor-

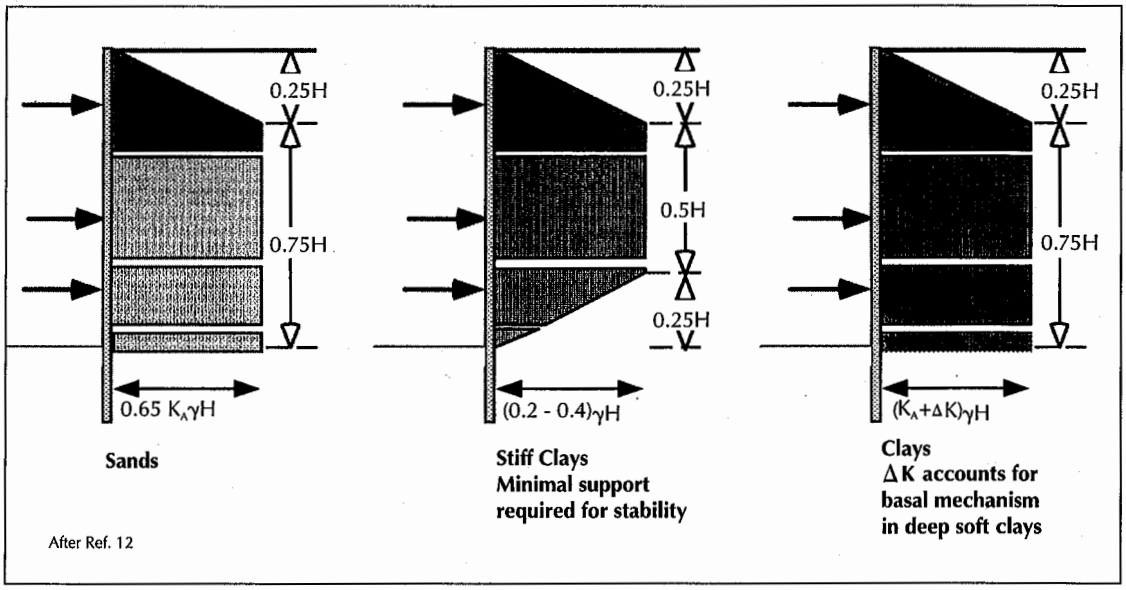

FIGURE 2. Apparent earth pressure design charts.

tance of careful measurements of soil properties (especially laboratory tests — even on high-quality samples). For example, the one-dimensional compression data shown in Figure 1 indicate important variations in clay compressibility with depth (and stress level) within a single clay profile. This type of behavior is only rarely taken into account in simple one-dimensional settlement calculations (where constant values of the compression ratios CR, RR are widely assumed), and even more infrequently in finite element analyses. Similarly, there is a tendency in finite element analyses to rely on published correlations for estimating stiffness parameters, even in projects where laboratory triaxial tests have been performed as part of the site investigation program.

Future Needs

The availability of high-quality software and the massive increase in available computational power guarantee the increased usage of finite element analyses in geotechnical engineering. An assessment of current program capabilities and limitations can lead to the formulation of a number of actions that should be taken in order to achieve wider acceptance of these methods.

In the short term, the profession needs to adopt procedures for benchmarking the programs currently used in practice. This step is particularly important since it provides a way to check that users are achieving numerical accuracy in their nonlinear finite element calculations. It is also essential for validating any non-standard components (constitutive laws, interface elements, etc.) for programs with an open architecture. The benchmarking could include:

- Calculations of standard example problems where complete input parameters for the finite element model are specified (foundations, excavations, etc.) — checking numerical accuracy only;
- Problems where input data are provided but parameter selection for the finite element model is left to the user — checking the modeling approach; or,
- Evaluation of predictive capabilities using case-study data from real projects or controlled experiments.

Wider acceptance of finite element analyses for design will depend, in large part, on the validation of predictive capabilities through the publication of well documented case studies.

The long-term development of finite element analyses will require progress in three main directions:

- Improvements in the measurement, interpretation and representation of material properties in finite element analyses. Many advanced constitutive models have already been presented in the research literature.[14] Some of these models will undoubtedly appear in commercially available finite element software, while others will be used as plug-ins (customized features). However, further research is definitely required to address some specific aspects of behavior that remain poorly understood (*e.g.*, the time-dependent behavior of clays, the behavior of partially saturated natural soils and compacted clays, frozen soils, etc.), and to define properties for certain classes of material (*e.g.*, residual soils, soilcrete, etc.).
- Integration of finite element analyses with statistical and probabilistic techniques is inevitable given the large uncertainties that exist in geotechnical projects (geological anomalies, spatial distributions of soils and their engineering properties, etc.) and should be encouraged. There has already been much work on the development of statistical and probabilistic methods for use in site characterization (*e.g.*, exploration strategies[15]), reliability-based design (*e.g.*, first-order second moment methods combined with limit equilibrium analyses of slope stability[16]), and stochastic finite element analyses that consider random and spatially correlated soil properties[17] and address complex problems such as liquefaction potential.[18] Finite element methods have also been used in conjunction with formal back-analysis techniques (*e.g.*, optimized estimation of soil properties from field measurements of ground movements[19]). Despite these advances, there is currently little application of these techniques in geotechnical engineering practice. This lack of use reflects conceptual difficulties associated with non-deterministic methods of analysis and also the need to demonstrate practical benefits in design.
- Substantial and sustained educational efforts are needed to train geotechnical engineers in the most effective use of finite element analyses. This process is already occurring in the form of professional short courses related to the recent spread of commercial geotechnical finite element programs. However, there is also need for adequate training in finite element methods in most geotechnical graduate degree programs (in the United States). This training requires providing both background (theoretical basis and techniques of numerical analyses) and application knowledge (modeling nonlinear soil behavior, parameter selection, etc.). This situation will certainly change now that high-quality (user-friendly and robust) software is available.

NOTE — *This article is based on a presentation at a technical session entitled, "What Has the Finite Element Method Done for (or to) Geotechnical Engineering?" held at the ASCE National Convention in Boston in October 1998.*

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REFERENCES

1. Zienkiewicz, O.C., *The Finite Element Method in Structural and Continuum Mechanics*, McGraw-Hill Co., London, 1995.

2. Desai, C.S., & Abel, J.F., *Introduction to the Finite*

Element Method, van Nostrand Reinhold Company, NY, 1972.

3. Roscoe, K.H., & Burland, J.B., "On the Generalised Stress-Strain Behavior of 'Wet' Clay," in *Engineering Plasticity,* J. Heyman & F.A. Leckie, editors, Cambridge University Press, 1968.

4. Duncan, J.M., & Chang, C.Y., "Nonlinear Analysis of Stress and Strain in Soils," *Journal of the Soil Mechanics and Foundations Division, ASCE,* Vol. 96, No. 5, 1970.

5. Lysmer, J., Tabatabaie-Raissi, M., Tajirian, F., Vahdani, S., & Ostaden, F., "SASSI, A System for Analysis of Soil-Structure Interaction," *Report No. UCB/GT81-02,* University of California, Berkeley, 1981.

6. McDonald, M.G., & Harbaugh, A.W., "A Modular Three-Dimensional Finite Difference Groundwater Flow Model," *Techniques of Water Resource Investigations of the US Geological Survey,* Book 6, A1, US Government Printing Office, 1988.

7. Ghantous, I.M., "Prediction of In-Situ Consolidation Parameters of Boston Blue Clay," master's thesis, Massachusetts Institute of Technology, Cambridge, Mass., 1980.

8. Hoek, E., & Brown, E.T., "The Hoek-Brown Failure Criterion — A 1988 Update," *Proc. 15th Canadian Rock Mechanics Symposium,* Univ. of Toronto, 1988.

9. Whittle, A.J., "Why We Need Improved Soil Models," *The Earth, Engineers & Education, Whitman Symposium,* MIT Department of Civil & Environmental Engineering, 1995.

10. Duncan, J.M., "The Role of Advanced Constitutive Relations in Practical Applications," *Proc. 13th Intl. Conf. on Soil Mechanics and Foundation Engineering,* New Delhi, India, 1994.

11. Clough, G.W., & O'Rourke, T.D., "Construction Induced Movements of Insitu Walls," *Design and Performance of Earth Retaining Structures,* ASCE Geotechnical Special Publication No. 25, 1990.

12. Terzaghi, K., Peck, R.B., & Mesri, G., *Soil Mechanics in Engineering Practice,* 3rd Edition, John Wiley & Sons, NY, 1996.

13. Goldberg, D.T., Jaworski, W.E., & Gordon, M.D., "Lateral Support Systems and Underpinning," *Report FHWA-RD-75-128,* Federal Highway Administration, Washington, DC, 1976.

14. Pestana, J.M., "A Unified Constitutive Model for Clays and Sands," Ph.D. thesis, Department of Civil & Environmental Engineering, Massachusetts Institute of Technology, Cambridge, MA, 1994.

15. Baecher, G.B., "Analyzing Exploration Strategies," *Site Characterization and Exploration,* ASCE, 1978.

16. Christian, J.T., Ladd, C.C., & Baecher, G.B., "Reliability Applied to Slope Stability Analysis," *ASCE Journal of Geotechnical Engineering,* Vol. 120, No. 12, 1994.

17. Fenton, G.A., & Vanmarcke, E.H., "Simulation of Random Fields Via Local Average Subdivision," *ASCE Journal of Engineering Mechanics,* Vol. 116, No. 8, 1990.

18. Popescu, R., Prévost, J-H., & Deodatis, G., "Effects of Spatial Variability on Soil Liquefaction: Some Design Recommendations," *Géotechnique,* Vol. 47, No. 5, 1997.

19. Gens, A., Ledesma, A., & Alonso, E.E., "Estimation of Parameters in Geotechnical Backanalysis — II. Application to a Tunnel Excavation Problem," *Computers & Geotechnics,* Vol. 18, No. 1, 1996.

Effective Uses of Finite Element Analysis in Geotechnical Engineering

The greater capabilities of computer hardware and finite element software can produce safer, more economical designs as long as there is adequate training on how to perform these analyses.

W. ALLEN MARR

The guiding rule for doing finite element analysis is: "Know the answer before you start." To many, especially to clients, this statement might seem ridiculous. After all, if the answer is already known, why should more time and resources be spent to perform a finite element analysis? Furthermore, how can the answer for a complex problem be found without doing a finite element analysis?

Finite element analyses involve quite complicated geometric and mathematical models of simplified reality. Analyses of practical cases usually involve more than one mechanism and multiple materials within the same analysis. It becomes almost impossible to check that the analysis is correct by examining the results of the finite element analysis alone. Seemingly subtle changes in parts of the geometric model or in the details of the material models can sometimes lead to sizeable changes in the computed result. Errors in the model definition within the program input can go undetected. An estimate of what the answer should be serves as a benchmark with which the results of the finite element analysis can be evaluated. Without knowledge of what the answer should be, there is little basis to decide whether the finite element model is a reasonable representation of reality or not. Having a finite element model that looks great on paper is quite possible, yet that model may give calculated displacements that are 0.1 to 10 times those of the actual situation. Knowing what the answer should be provides a way to review and modify the finite element model so that it better represents reality.

So, how does one obtain an answer before running a finite element analysis? Simpler

TABLE 1.
Levels of Analysis

Level	Analysis Method	Material Parameters
Simplified Analysis	Semi-empirical calculations from experience & local correlations	Estimated parameters from experience & index tests
Standard Analysis	"Standard practice" methods from geotechnical books, codes & local experience	"Standard practice" testing such as triaxial, direct shear, field vane, SPT & cone
Advanced Analysis	Advanced numerical methods including finite element, finite difference & boundary element	Best available from lab & field tests that consider stress path

methods and experience must be used. Table 1 represents an attempt to classify the levels of analysis. The table also shows that the level of analysis should be matched by an equal level of sophistication in the material parameters used in the analysis. Therefore, the answer to the question is that one tempers simplified and standard analysis methods with experience in order to obtain an estimate of the answer before undertaking a finite element analysis. This preparatory effort results in:

- Developing a sense for what the final answer should be;
- Obtaining insight on what parts of the problem are important and should be carefully modeled; and,
- Defining the objective(s) for the more advanced finite element analysis.

Why undertake a finite element analysis if one must know the answer before starting such an analysis? There are a number of answers. Finite element analysis can remove many simplifications and assumptions used in simpler analyses. Finite element analysis can help refine the answer to obtain a more precise prediction. Finite element analysis can give better insight into the behavior of the problem. Finite element analysis can help look at alternatives in a systematic way. Finite element analysis can help extend a design beyond the envelope of normal practice. Finite element analysis can be particularly useful in analyzing the causes of failures.

Three cases are presented where finite element analyses were of considerable value to the outcome. These cases were chosen to illustrate the power of finite element analysis in today's engineering practice, and to show that finite element analysis has progressed beyond the position of being a sophisticated tool used by a few academic specialists.

Comparison of Design Alternatives

This case involved the construction of a highway to be placed in a tunnel in the center of a major US city. The final structure was to be a tunnel 90 feet wide with a crown 60 feet below the ground surface. The design called for a 100-foot deep excavation, 100 feet wide, supported by several levels of massive struts. Major structures with foundations within 50 feet of the excavation existed on both sides of the work. Part of the highway had to pass beneath an existing subway station. The contractor wanted to consider replacing the cut-and-cover design for the excavation with a tunnel excavation. Tunneling could potentially reduce excavation and spoil, as well as save time and money.

A principal question dealt with the relative impacts on adjacent structures of the two approaches. Would one approach cause more movement of the existing foundations than the other? Finite element analysis of the two approaches provided a way to examine the size and pattern of movements produced by each approach. By using the same soil profile and soil parameters, the analysis could focus on which excavation method would cause less

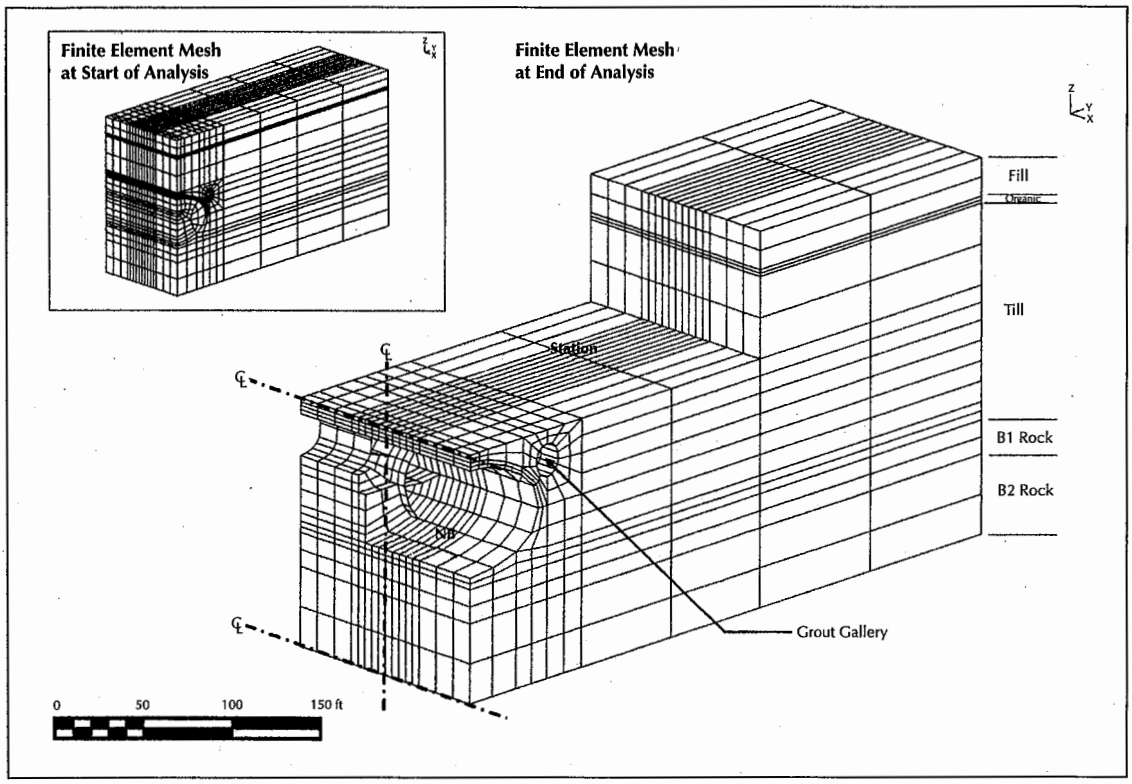

FIGURE 1. Finite element mesh for a tunnel beneath a subway station.

displacement. The soil profile and typical soil parameters had been previously developed for the original design, so developing the input information for the finite element analysis was straightforward.

From an analysis perspective, the big challenge of this project was to have the analysis follow the sequence of construction as closely as possible. Since the size of movements around carefully designed and constructed supported excavations are as much influenced by the construction details as they are by the material parameters, considerable effort was required to develop a finite element mesh that could follow the significant steps of the construction. The mesh had to allow for the removal of soil in a staged manner, the addition of supporting elements and changes in the groundwater level. Additionally, it had to include a realistic representation of the foundations for the existing structures.

Figure 1 shows the typical finite element mesh developed for the tunnel section passing beneath the existing subway station. It shows elements placed into the mesh to model the different soil materials, to model a small tunnel to support grouting activities and to simulate construction of the mainline tunnel. The proposed tunneling method involved the use of the New Austrian Tunneling Method (NATM). The finite element model included considerations for the temporary support provided by the shotcrete and lattice girders used in NATM. Presence and material properties for these various elements were tracked in sequential steps within the analysis, similar to the steps in the actual construction process. A similarly detailed mesh was developed for the cut-and-cover method given in the contract design. The actual analysis was done with the finite element program ADINA.

Figure 2 shows a typical result obtained from this analysis. It shows a section where a high-rise building is close to the excavation. The top half shows the cut-and-cover design method. The bottom half shows the tunneling method, which at this location involved two

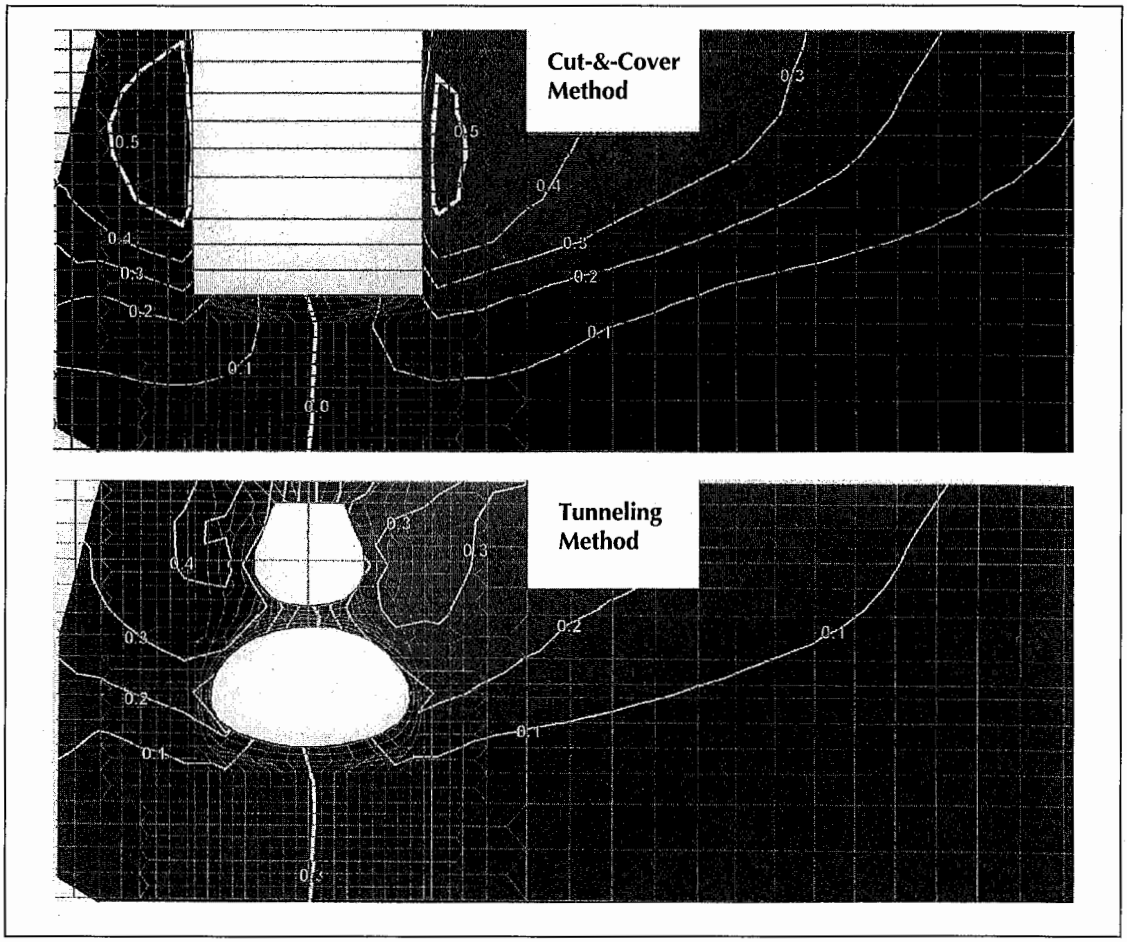

FIGURE 2. Computed horizontal displacements (inches).

tunnels, one over the other. The contours show the predicted horizontal displacement resulting from the excavation. The key question being addressed with the finite element analysis is the potential impact of the excavation on the adjacent facilities. Figure 2 reveals that the predicted horizontal displacements beneath this building from tunneling are approximately one half of those predicted for cut-and-cover tunneling. The differential horizontal movement across the base of the foundation is approximately 30 percent less for tunneling than for cut and cover. The differential horizontal movement across the foundation is important because it stretches the building foundation in tension. Similar reductions occurred for vertical deformations. The finite element results showed that the tunneling method would cause less impact on the building foundation from deformations than the cut-and-cover method. The analytical study of the finite element analysis performed did not consider risk factors associated with NATM, such as the availability of skilled laborers in the United States, requirements for close coordination of field measurements and reaction for contingency plans associated with this method, and other factors.

In this situation, the same method, with consistent parameters and assumptions, was used to analyze the different cases. This approach can provide considerable confidence that the predicted differences in displacements, strains, forces and stresses are real and reliable. It can also provide an unbiased comparison of the performance benefits of one design over another and present alternatives that may further improve on the design. In these situations, having highly refined soil parameters for the

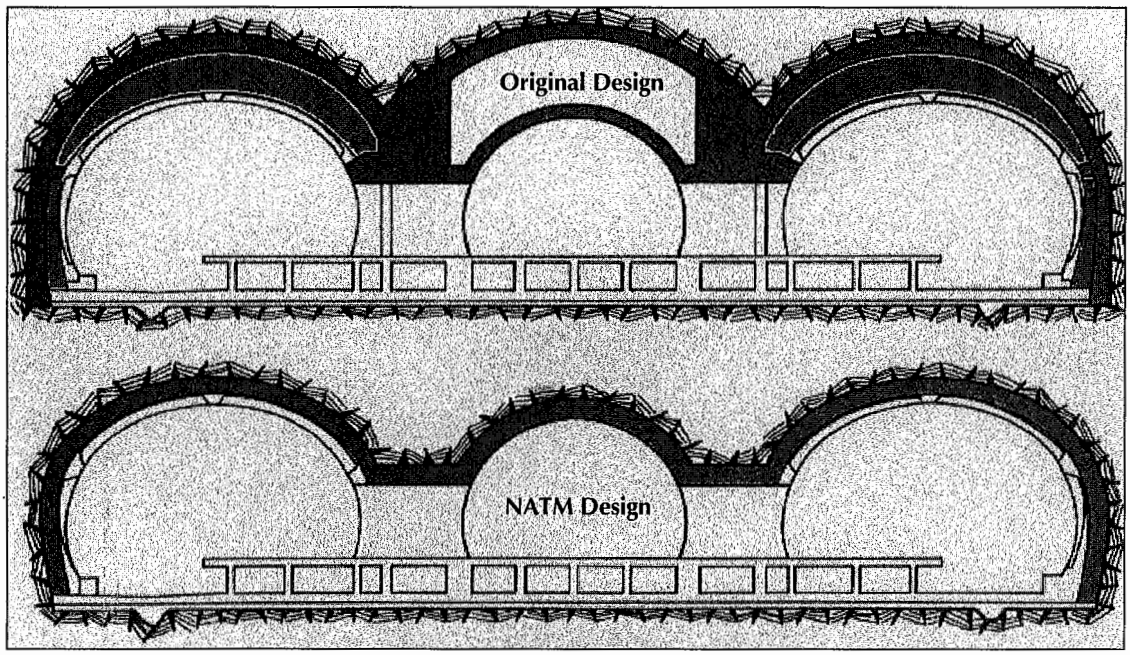

FIGURE 3. Wheaton Station cross sections.

analysis may be less important that having the analysis consider the important details of construction sequence and methodologies. Here, for example, how to model the important influences of initial slack in the bracing system and loss of ground at the tunnel face had to be carefully considered.

Extending the Design Envelope

Geotechnical engineers use design methods that usually encompass past experience. These methods employ considerable conservatism to keep the risk of failure low. Situations frequently occur where one would like to work outside the design envelope to reduce time, save money or accomplish something not previously tried. Advanced analysis can help predict performance outside the usual design envelope.

In the early 1980s, Washington, DC, was engaged in a vigorous effort to build a new subway system. The contractor working on the Wheaton Station for the Washington Metropolitan Area Transportation Authority (WMATA) faced a difficult task to complete a complex intersection of inbound and outbound tunnels with a cross-over tunnel and an inclined escalator shaft. The contractor proposed changing the design to one using NATM and making major reductions in the thickness of the lining system. Figure 3 shows the original design and the proposed NATM design. NATM had been previously used only once in the United States. WMATA had no design codes or methods with which to assess the integrity of the contractor's proposal. A key question was whether the contractor's proposed liner had sufficient strength to support the excavation and avoid overstressing some rock pillars to be left in place between the tunnels and the escalator.

With the assistance of Herb Einstein, of the Massachusetts Institute of Technology, a finite element analysis of the contractor's proposed design was performed. The actual work was a modeling nightmare. A finite element mesh had to be developed that included all of the complicated three-dimensional intersections of the excavation and the lining system and include bar elements for the rock bolts. A mesh processing program called PATRAN was selected to help create the mesh because it had been quite successful in modeling complex geometries for the aircraft, automotive and defense industries. After weeks of effort, and with the help of a PATRAN engineer, a mesh was created. ADINA was then used to do the finite element analysis.

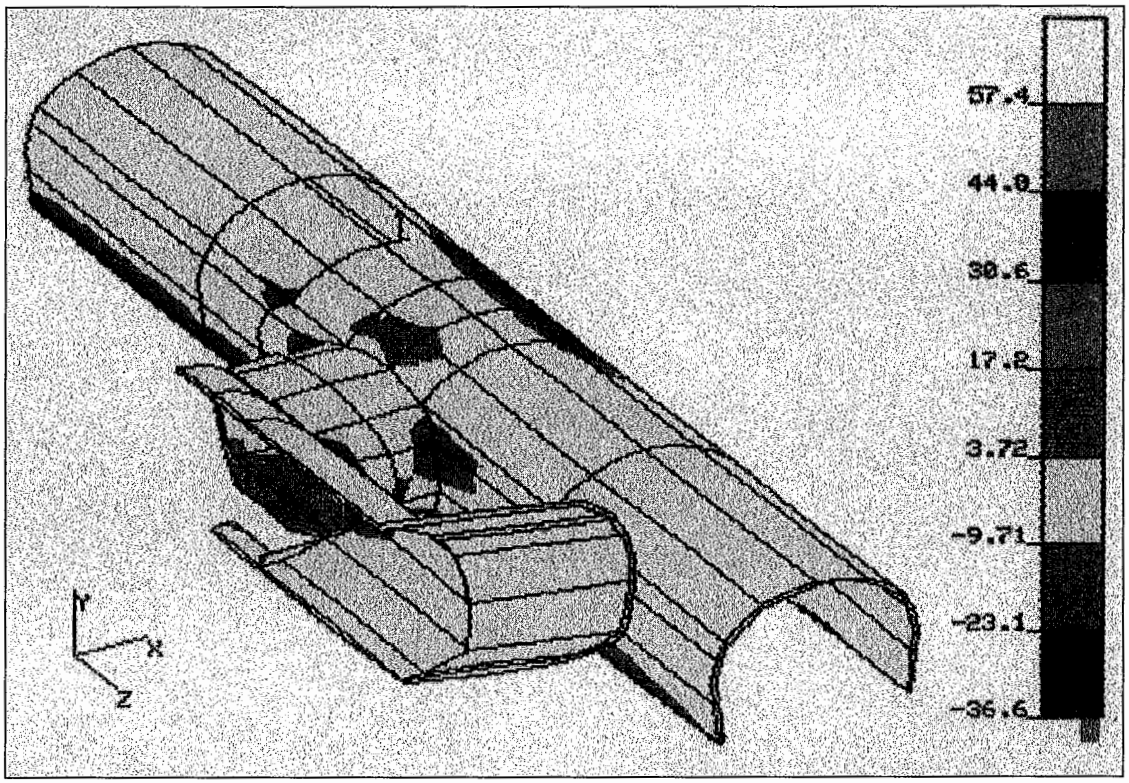

FIGURE 4. Tensile stresses in a shotcrete liner (ksi).

Figure 4 shows the primary result of all of this work. It shows principal stress in the shotcrete liner system at the completion of excavation. The shotcrete provided the initial tunnel support. It would be supplemented with the final cast-in-place liner to provide long-term tunnel support system. Figure 4 indicates that some locations could develop tensile stresses well in excess of the tensile strength of the shotcrete liner. However, no problem with overstressing of the rock pillars and no problems with the final liner system were found. Based on these analyses and other considerations, the contractor's proposal was modified to increase the tensile strength of the shotcrete liner. The project was successfully completed with a savings of millions of dollars accruing to the owner and the contractor. Better water tightness of the final tunnel was achieved as a side benefit.

The finite element analysis helped show that NATM would work on this project, but more reinforcing steel was required to handle the tensile stresses in the shotcrete. The results of the analysis were key in giving the designers and the owner the confidence to accept the contractor's value engineering proposal. Finite element analysis helped the project participants work outside the normal design parameters. The success at Wheaton Station opened the way for more applications of the NATM technique in the United States.

Failure Analysis

Many failures involve performance outside the working zone encompassed by design envelopes. Design methods do not reveal what happens at failure. The results of finite element analyses can give insight to likely failure modes, suggest paths that could lead to failure and help predict performance up to failure.

This case illustrates the use of finite element analysis to help determine the cause of failure. It involves the wheels on cars used to move concrete forms for a tunnel lining in Chicago. Each car had four wheels that rode on the concrete invert. Each wheel consisted of a solid steel hub covered with a 2-inch thick solid polyurethane tire. Less than 2,000 feet into the

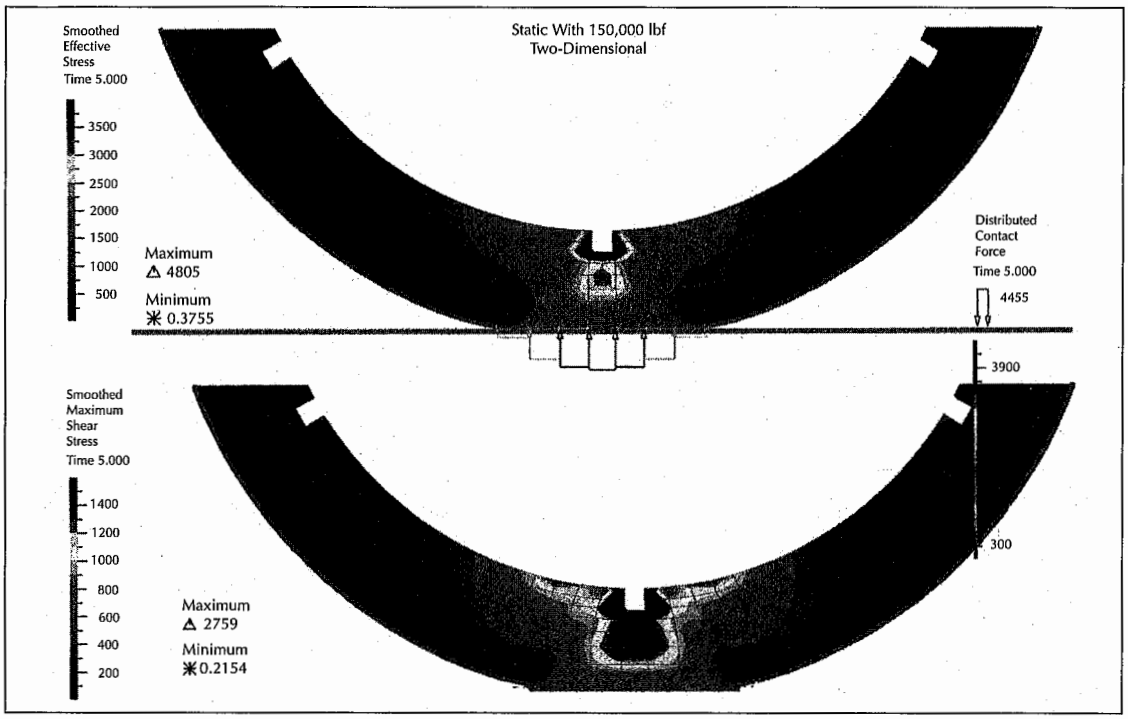

FIGURE 5. Stresses in a solid tire with webs (psi).

50,000-foot job, the tires began to fail. This failure brought the concreting operation to a halt for several hours while a tire was changed. By 2,500 feet, four tires had failed. The contractor recognized that there was a serious problem.

The tires were examined and it was observed that the polyurethane was separating from the steel hub at the bond. However, the visual evidence did not clearly show the cause of the failure. By the time failure was observed, the tire was so badly damaged that the evidence of initial failure was obscured. The polyurethane manufacturer informed the contractor that properly formulated and molded tires should develop a bond stronger than the material itself. However, it was noticed that steel gridwork had been added to the steel hub. The grid consisted of 0.5-inch square bars welded to the circular hub. Two bars were placed around the perimeter of the hub about 2 inches inside the edges. Six bars were placed around the perimeter parallel to the axis of the tire. These bars protruded into the tire and created the potential for concentrating stresses within the polyurethane.

Finite element analyses were conducted to figure out the stresses in the tire for various loading conditions. The total force delivered to each tire for in-service conditions was measured by placing strain gauges on the wheel struts and taking continuous measurements during a typical pour cycle. The maximum measured force in one tire was 150,000 pounds and represented approximately half the total weight of the car. This force was used to analyze the tire in different configurations with the finite element program ADINA. With ADINA, the tire could be modeled as a separate body, then lowered onto a solid surface and loaded in steps to the full load. The tire could then be rotated to see what configuration of the steel webs caused the greatest stress concentrations. Figure 5 shows the worst-case condition determined from a two-dimensional analysis, where the tire is considered to have an infinitely long axis.

Figure 5 clearly shows the stress concentrations produced by the steel web. The computed maximum compressive stress was 4,800 psf. This stress was more than twice the design compressive stress of the polyurethane. A

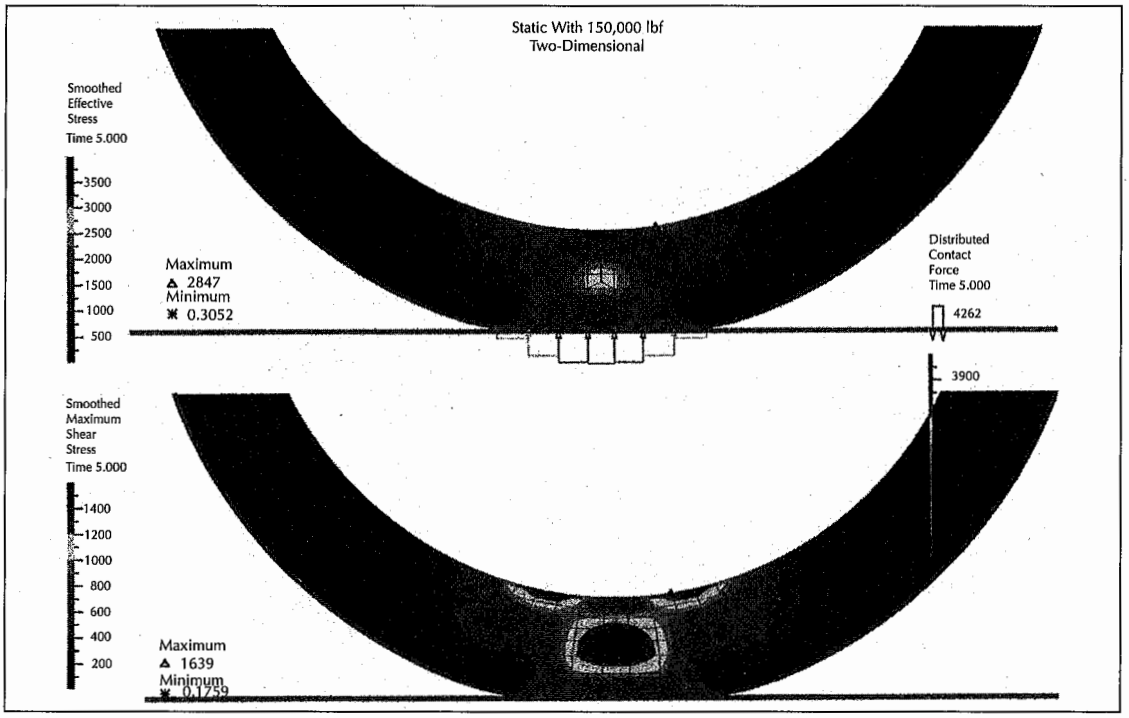

FIGURE 6. Stresses in a tire with webs removed (psi).

three-dimensional analysis for the same loading conditions revealed even worse stress conditions. The tire was also analyzed without the steel webs. Figure 6 presents the result. The maximum compressive stress was reduced to 2,850 psf, a 40 percent reduction. This value was still higher than the recommended design value for the polyurethane.

Based on measurements of the forces developed in the tires and the stresses in the tire computed from the finite element analysis, it was recommended that the webs be removed from the steel hub and a new polyurethane tire be molded onto the hub. The polyurethane manufacturer assured the design consultant that sufficient bond strength would develop if proper molding and curing procedures were followed. The contractor rebuilt every tire. Changes in the carrier hydraulics and operations to reduce the maximum force developed in each tire were also recommended. With the "new" tires, the only other tire failures on this project were a couple of tires that were cut by sharp objects. This success reduced the contractor's potential costs attributable to delays from tire failures by several million dollars.

The results of the finite element analysis played key roles in helping to decide why these tires were failing and in showing the benefits of various alternatives. The analyses indicated that the webs were greatly overstressing the polyurethane and that removing the webs would reduce those stresses. The analyses also permitted looking at the tire in different positions to ensure that the most critical configuration was being examined.

Role of Finite Element Analysis in Practice

Until recently, finite element analysis in geotechnical engineering has been limited to special projects where other alternatives were exhausted or unavailable. The analysis required one or more specialists to obtain a useful answer. This situation, however, is changing.

Powerful microcomputers and easier-to-use operating systems are making it less costly to perform analyses. The WMATA case consumed more than $50,000 of commercial computer time on a minicomputer. It took more than two months to prepare the finite element model. The equally complex Boston case was per-

formed on a microcomputer that cost less than $4,000 to purchase. It took about two weeks to prepare the finite element model using a more user-friendly graphical interface.

A selection of new and upgraded finite element programs are becoming available that are more comprehensive in their capabilities, more robust in their operation and easier to use. These programs make the finite element portion of the analysis transparent to the user. The user defines the geometric model and the material properties without any consideration given to the details of finite element analysis. Many programs automatically create the finite element mesh and apply boundary conditions through a graphical interface. The output is presented as contours or shaded zones of equal stress or displacement.

Whereas previous generations of programs required up to several days to create, correct and refine a finite element model, these new programs reduce the effort to a few hours at most. In the past, a minimum of one week usually would be budgeted to set up and run a finite element analysis for seepage or displacement. Another week would have to be scheduled to run various cases and study the results. With these newer programs, the problem typically can be set up in one day, with another day to run the various cases and study the results. Of course, difficult problems, problems where there is no prior experience and problems where a program is being used for the first time can take much longer to set up and to interpret the results.

Another great advantage of some of the new programs is that they can perform different analyses with the same input information. The user can define the geometry and material parameters once, then continue to do a flow analysis, a consolidation analysis, a deformation analysis and a stability analysis. Previously, each analysis would require a different program, each with its own finite element mesh and material input requirements. This ability can save considerable analysis time and permit these various performance modes to be combined in complex problems.

Some new programs include a variety of elements that permit one to analyze geotechnical problems with structural members, geotextiles and slip interfaces. They provide a much improved analysis of the discontinuities produced by the different properties of these materials. These programs should provide the means to do a much better job analyzing soil-structure interaction.

Finally, most new graphical-user-interface-based programs include improved options for displaying the results of the analysis. These options let the analyst examine large quantities of output quickly and efficiently, as well as let the analyst present the results in ways that non-specialists can understand.

Conclusions

It has been more than thirty years since the first use of the finite element method in geotechnical practice. The development stage of this technology has been left far behind. Practicing engineers can now focus on using the tool rather than fussing with the mechanics of doing the analysis.

Powerful microcomputers, easy-to-use interfaces, better software and more experienced engineers are making it cost effective to use finite element analysis on more routine work. Using a finite element program to analyze many geotechnical problems in a few hours from start to finish is now possible for experienced users. This optimistic statement assumes that the geometry is known and relatively simple, the material parameters are defined and the analyst is very familiar with the software being used.

The use of finite element analysis in day-to-day geotechnical practice will increase considerably over the next few years. This greater use is due to the presence of tremendous computing power on most engineers' desks, the availability of reliable finite element software that most engineers can learn to use and the increasing computer literacy of young geotechnical engineers.

This widespread capability does cause some concerns. Analysts with inadequate geotechnical knowledge should not use finite element programs to solve complex geotechnical problems. A strong understanding of effective stress principles and of soil behavior is essential to anyone doing finite element analysis of geotechnical problems for design.

There is also the problem of inexperienced persons consuming project resources trying to do finite element analyses without coming to a useful answer. These analytical failures give finite element analysis a bad name. While it is possible to obtain an answer with finite element analysis in a few hours, some geotechnical problems can become quite complex. Getting an appropriate model can become quite involved. Evaluating and interpreting the output can be intellectually demanding and time consuming. Any team working on a complex problem and using finite element analysis should have at least one person on the team who is well versed and experienced in the finite element tools being proposed for the project.

There is also the trend for people to be impressed with nice-looking graphics even though the information presented in those graphics may not make sense or address the key issues of the project. Impressive graphics can be prepared from meaningless information. Engineers will become ever more professionally challenged trying to figure out which of these impressive graphics make sense and help advance a project.

As finite element tools become more sophisticated and easier to use, the emphasis is decreasing on how to do the analysis and focusing more on obtaining meaningful input information. To co-opt a phrase from recent political history to suggest the future of finite element analysis in geotechnical engineering: "It's the input, stupid."

NOTE — *This article is based on a presentation at a technical session entitled, "What Has the Finite Element Method Done for (or to) Geotechnical Engineering?" held at the ASCE National Convention in Boston in October 1998.*

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The New Bedford-Fairhaven Bridge

An innovative and elegant truss design was used on this historic bridge that enabled a shift in the area's economic base from whaling to manufacturing.

FREDERICK M. LAW

Seventy miles south of Boston on the west side of the mouth of the Acushnet River lies the city of New Bedford, Massachusetts. Approximately 150 years ago, New Bedford was known as the "Whaling Capital of the World," the city of Herman Melville's *Moby Dick*. A little over 100 years ago, however, after the collapse of the whaling industry, New Bedford realized that its economic viability depended on having a new bridge that would not only link it to the east side of the Acushnet River but would, at the same time, permit the passage of ocean-going ship traffic. For this reason, in 1893, a city engineer by the name of William Fish Williams was recruited and charged with the task of getting such a bridge built.

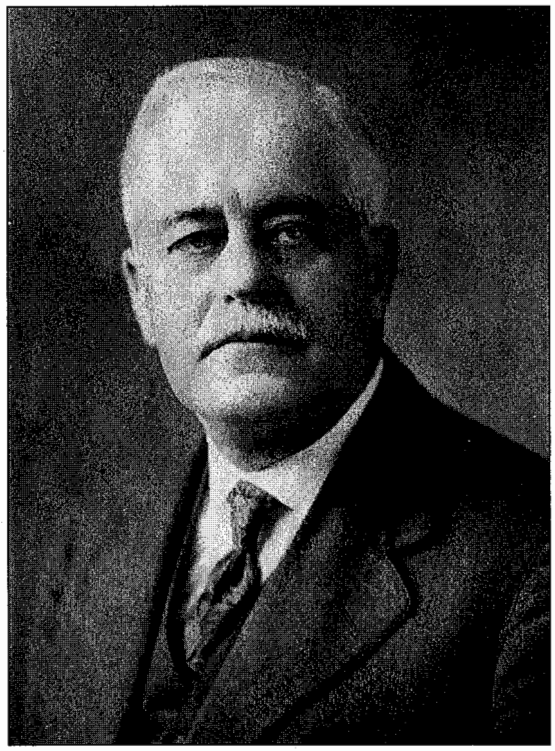
The Engineers

William Fish Williams was, coincidentally, the son of a New Bedford whaling ship captain. He was born at sea and grew up on-board ship and on both coasts of the United States. In 1881 he graduated from Columbia University with a degree in Civil Engineering. Prior to his appointment as City Engineer of New Bedford in 1893, he practiced engineering in Hartford, Connecticut. After directing the construction of the New Bedford-Fairhaven Bridge, Williams directed the construction of the Cape Cod Canal and later became the Commissioner of Public Works for the Commonwealth of Massachusetts.[1]

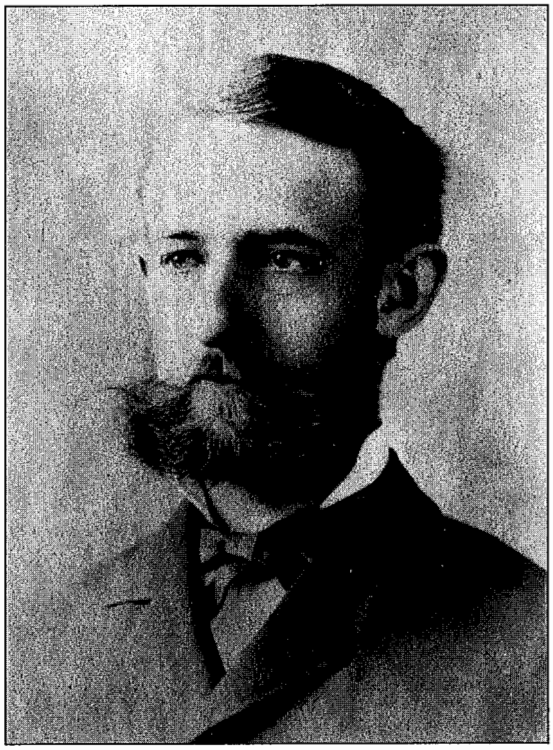
To actually design the New Bedford-Fairhaven Bridge, Williams enlisted the aid of a structural engineer named George Filmore Swain. Swain was also from a New England whaling family, but unlike Williams, he grew up in New England. At age 16 he entered the Massachusetts Institute of Technology (MIT) and graduated in 1877 with a degree in Civil Engineering. A few years later he returned to MIT to become a member of the faculty and by 1887 had risen to the rank of full professor and chair. Professor Swain served as structural engineer for at least ten major bridges, including the New Bedford-Fairhaven Bridge while at MIT. In 1909, he accepted the Gorden McKay Professorship of Civil Engineering at Harvard University and in 1913 was elected president of the American Society of Civil Engineers.[2]

The Bridge

The bridge that these two distinguished engineers created for New Bedford is the 289-foot



William Fish Williams (1889).



George Fillmore Swain (1885).

long, rim-bearing, thru-truss, swing bridge now known as the New Bedford-Fairhaven Bridge (see Figures 1 and 2). The chords of the trusses of this bridge consist of two 20-inch deep steel plates, 20 inches apart, joined by angles and lattice bars. Almost all of the verticals consist of two 12-inch channels, 12 inches apart, joined by lattice bars. The diagonals and counters are double eye bars ranging in depth from 3.5 inches to 7.5 inches. All the joints of the trusses are pin connected, with pins ranging in size from 3.5 inches to 8.5 inches in diameter.

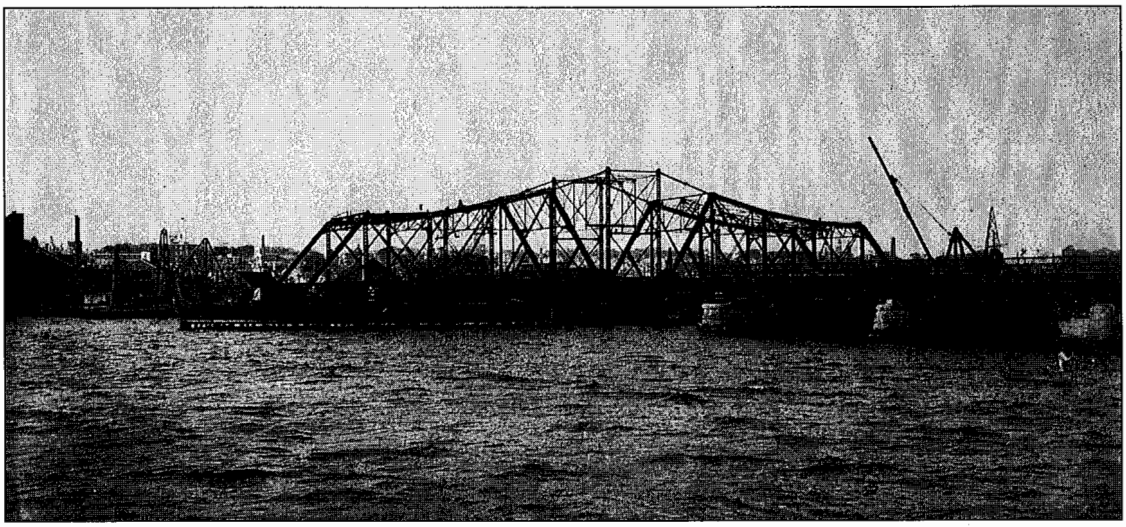


FIGURE 1. The New Bedford-Fairhaven Bridge in open position during construction (1898).

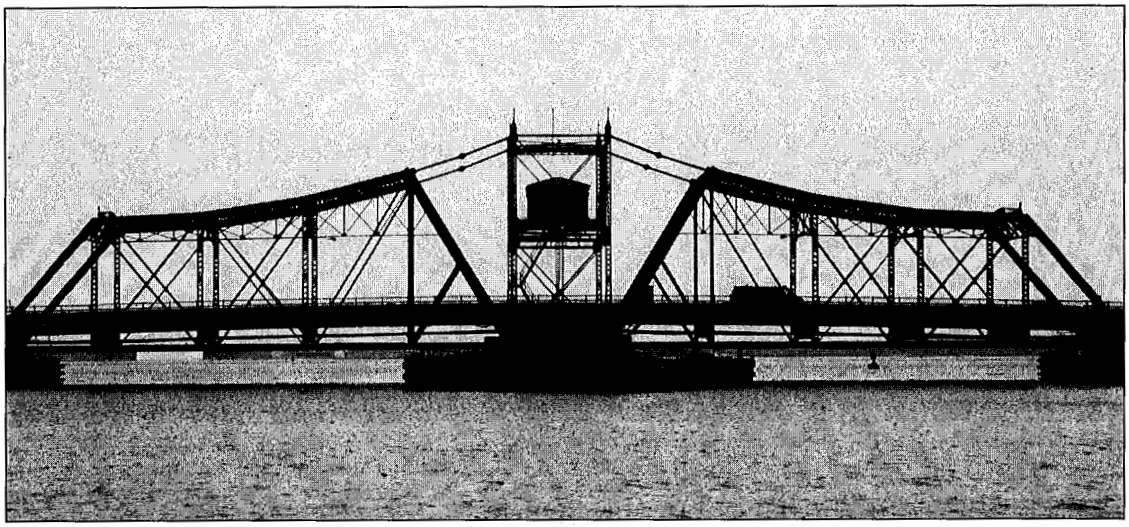


FIGURE 2. The New Bedford-Fairhaven Bridge in the closed position (1998).

The floor of the bridge consists of concrete-filled steel grid deck supported by steel I-beam stringers, which are in turn supported by steel floor beams hung from the verticals of the main trusses.

The fundamental structural action of the bridge depends on whether the bridge is open or closed. When the bridge is open, it rests solely on the center pier and acts as a balanced cantilever. When the bridge is closed, it rests on all three support piers and acts as two simple-span truss bridges with a freestanding tower in the middle.

This complete reversal in the structural action of the bridge is made possible through the innovative use of the hinged upper chord members on either side of the center tower. When the bridge is open, these hinged upper chord members are tensed and carry load. However, when the bridge is closed (and the ends are jacked up slightly), the hinged upper chord members go slack and carry no load (see Figures 3 through 5).

In profile, there is a definite "beauty of form" to the New Bedford-Fairhaven Bridge. In the open position, the bridge is clearly proportioned as a balanced cantilever with the depth of the structure that decreases from the center tower toward the outer ends. In the closed position (through the innovative use of the hinged upper chord members), the bridge becomes two simple-span truss bridges, each of which is appropriately proportioned with a depth of structure that rises from the piers toward the center of each span, albeit not symmetrically. As St. Thomas Aquinas has stated, "The senses delight in things duly proportioned!"

There is also an extraordinary lightness and laciness to the trussing of this bridge. This lightness gives the bridge an almost "gossamer" quality — a quality that is certainly rare in bridges today.

The Reviews

In 1897, when the bridge was nearly completed, the local newspaper, the *New Bedford Standard Times* proclaimed that, "The draw span of the New Bedford and Fairhaven bridge will be one of the longest and finest in New England and one of the greatest draw bridges of the county. Nothing this side of New York will equal it, in some respects it will be the only one of its kind."[3]

In 1990, almost one hundred years later, when the New Bedford-Fairhaven Bridge was selected for the Historic American Engineering Record, historians (perhaps choosing their words more carefully) described the bridge as "noteworthy for its tremendous size and its innovative truss design."[4]

The year 1998 marks the one-hundredth year the bridge has served the city of New Bedford. Many residents of the city embrace this

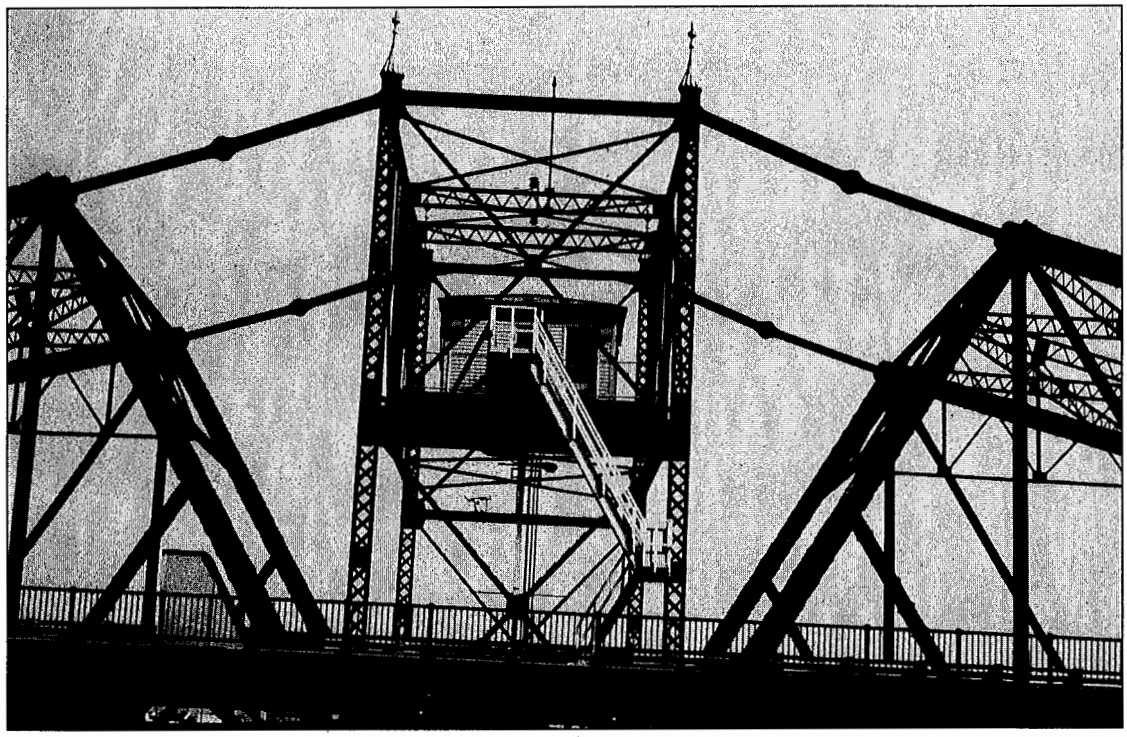


FIGURE 3. The center tower.

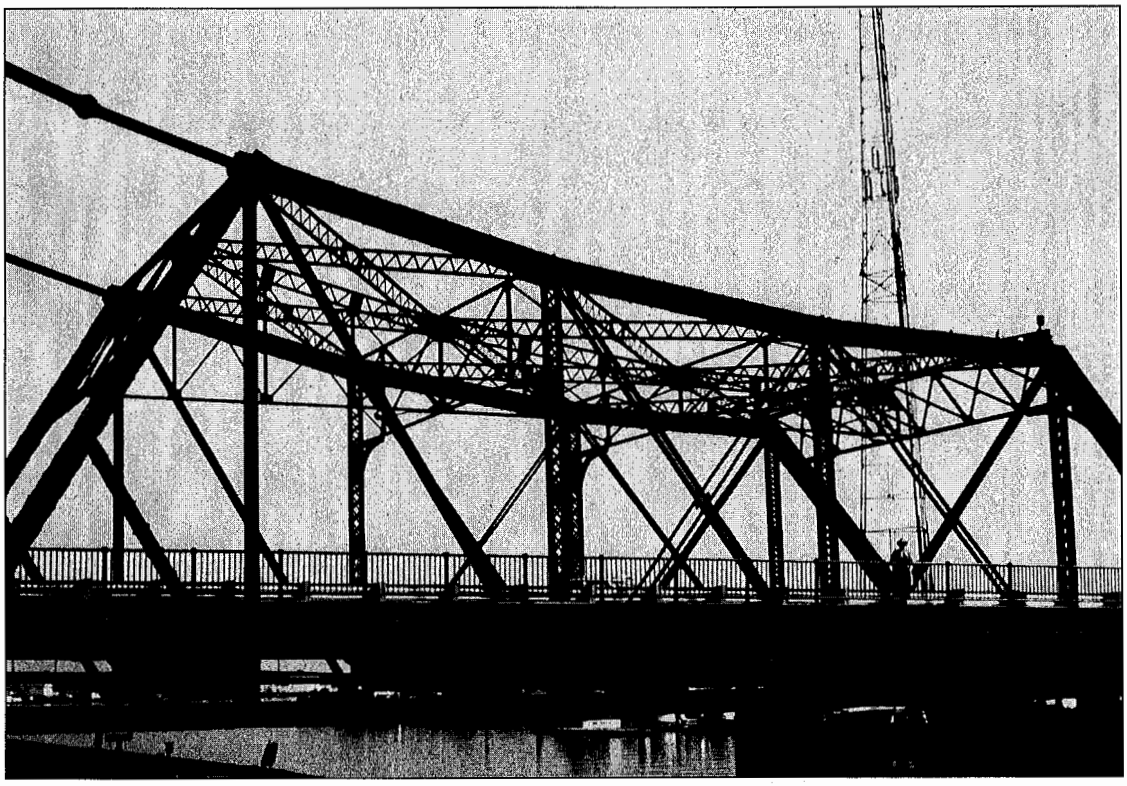


FIGURE 4. The east end of the New Bedford-Fairhaven Bridge.

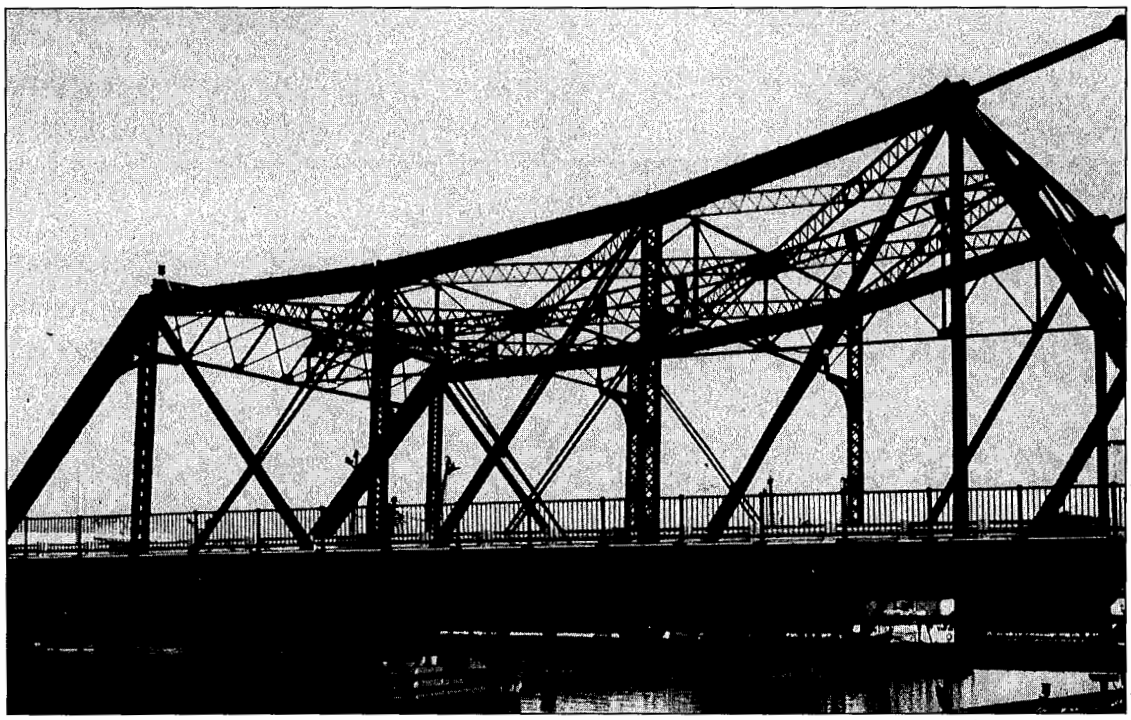


FIGURE 5. The west end of the New Bedford-Fairhaven Bridge.

venerable old bridge for its long service and its historical significance; many residents simply embrace the bridge for its delicate beauty.

NOTE — *Figure 1 is used with the permission of the Board of Trustees of the New Bedford Free Public Library.*

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REFERENCES

1. "Memorial to William Fish Williams," *Journal of the Boston Society of Civil Engineers,* Vol. 19, 1932.

2. "Memorial to George Fillmore Swain," *Journal of the Boston Society of Civil Engineers,* Vol. 19, 1932.

3. "Scrapbook of William Fish Williams," ca. 1890-1900, New Bedford Free Public Library, New Bedford, Mass.

4. "New Bedford-Fairhaven Middle Bridge," *Historic American Engineering Record No. MA-101,* National Park Service, 1990.

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