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REMR-Designed Precast Concrete Stay-in-Place Forming System Used for Concrete Repair at Troy Lock and Dam

by

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Technology developed under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program was used for repairs to the lock walls, aprons, and related concrete surfaces at the 77-year-old Troy Lock, located on the Hudson River, in Troy, NY. The REMR design for precast stay-in-place forms, placement techniques, installation of anchors, concrete removal techniques, and underwater concrete placement methods enabled the efficient and cost-effective replacement of deteriorated concrete with a durable, abrasion-resistant concrete surface.

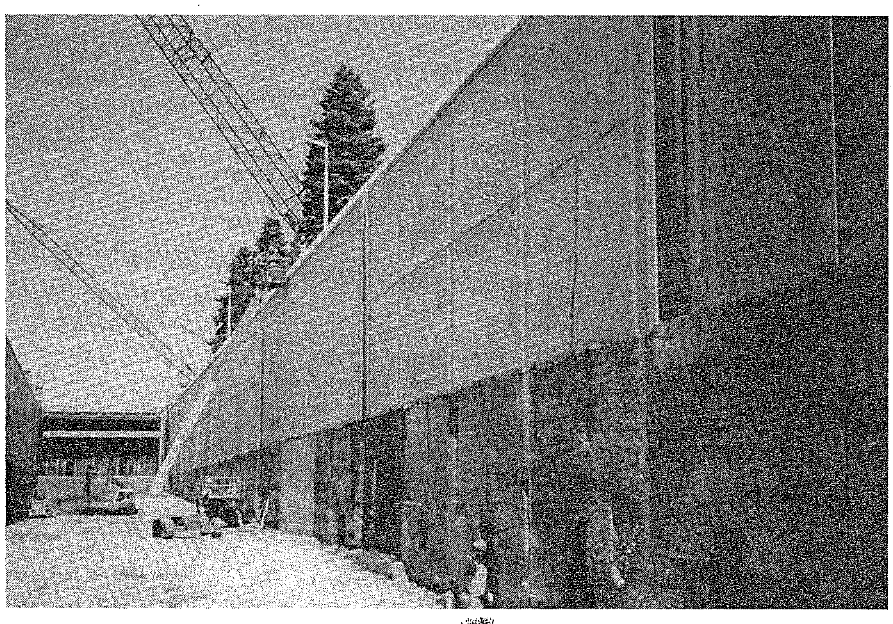
Background

The Troy Lock and Dam is located on the Hudson River Federal Navigation Channel approximately 150 miles upstream from New York City. The only Federal lock on the Hudson River, it is an integral part of a system of waterways that connects the Hudson River with the Great Lakes to the west and with Lake Champlain and Canada to the north via the New York State (NYS) Barge Canal System. Vessels traveling north through the Troy Lock enter the NYS Barge Canal System approximately 2 miles upstream and can navigate westward toward Buffalo, NY, on the Erie Canal or northward toward Lake Chaplain and Canada on the Champlain Canal.

Original construction of the Troy Lock was completed in 1915, and it was opened to navigation in the spring of 1916. The lock has a usable length of 492.5 ft, a usable width of 44.4 ft, and a minimum depth over sills of 16 ft. The mechanical gears that originally operated the gages and valves were replaced with hydraulically operated equipment in the 1970's. The electrical system was upgraded in the 1950's and again in 1986. Prior to 1988, no major concrete repairs had been performed to the lock or dam.

Need for Repair

In 1976, Waterways Experiment Station (WES) performed an



Lock chamber panel anchoring system

engineering condition survey and evaluation to determine the extent of damage to and deterioration of the existing concrete at Troy Lock. In September 1981, the results of this survey as well as a subsequent coring program, laboratory testing, stability analyses, and finite element analyses were published by the New York District in the "Reconnaissance Report for Major Rehabilitation," which recommended a complete rehabilitation of the Troy Lock and Dam and the dam headgates at an estimated cost of $23 million. The cost for this level of repair could not be economically justified.

To ensure continued operation of the lock and dam, New York District authorized the Plan of Study for Concrete Repair Program in April 1983. This study concluded that certain repairs had to be completed to keep the facility safe and operational for its remaining useful life or until such time that major rehabilitation would become justifiable. Based on this study, the Troy Lock Concrete Repair Program was initiated.

The condition survey determined that the interior concrete in the lock was sound and sufficiently strong. Cracking of the concrete was structurally insignificant except for the vertical shafts within the lock, access shaft, and pier sections at the dam headgates, and a horizontal crack at a construction joint in the lock river wall. The majority of the surface concrete was deteriorated to a depth of approximately 1 ft, although at some locations deterioration was found to a depth of 4 ft. The primary cause of this deterioration appeared to be the effects of cycles of freezing and thawing on the non-air-entrained concrete. Some alkali-silica reaction products and ettringite coating of the aggregates were noted, and some deterioration was found at the monolith joints where water was seeping from interior culverts. Surface abrasion and damage from vessels were also present.

Recommended Repairs

In 1985, design of the Troy Lock Concrete Repair Program was temporarily suspended, pending the results of a REMR study on new methods for performing repairs to lock wall concrete. Meanwhile, the Interim Repair Program was initiated to address critical operational and safety items at the lock and dam. Under this program, 10 lock wall monoliths were repaired with conventional cast-in-place concreting methods. Also, damage to the dam headgate piers was repaired, scour holes below the dam were filled with tremie concrete, and cracks at critical locations were repaired.

In 1989 upon completion of the REMR study, the New York District authorized a comparative evaluation of the REMR-developed, precast stay-in-place forming system versus conventional cast-in-place methodology as part of the 30-percent design phase for the Concrete Repair Program. The District's Architectural Engineer (AE) (Bergmann Associates, Rochester, NY) was tasked with evaluating the two methodologies and recommending the most favorable construction alternative based on cost, constructibility, and durability. After analyzing WES reports and observing actual installation of precast panels at Lock 22 on the Mississippi River at Hannibal, MO, the AE concluded that by incorporating several design improvements, the precast stay-in-place forming system was economically comparable to conventional concreting methods.

Since the precast panels were considered superior in strength and durability, the New York District authorized the use of precast concrete stay-in-place forms for repair of the Troy Lock walls. Additionally, based on a recommendation from Mr. James E. McDonald of WES, "jacketing" of the river side of the lock river wall with precast concrete panels was incorporated into the design, thereby eliminating costly concrete removal in this area. The design of the precast panels basically followed that presented in Technical Report REMR-CS-7, "Design of a Precast Concrete Stay-in-Place System for Lock Wall Rehabilitation," with several modifications.

Repair Technique

General

The repairs included removal and replacement of a minimum of 12 in. of concrete from the interior lock chamber walls between elevation -2 and +24 ft NGVD (National Geodetic Vertical Datum) and from 6 to 12 in. of concrete on the aprons and walkways, installation of precast panels over existing concrete (jacketing) on the river side of the lock river wall, removal of the center miter gates that were no longer used, and various associated work.

The AE developed plans and specifications based on the original design by WES for the precast panels with enhancements to the design to facilitate construction and overcome problems experienced at Lock 22 on the Mississippi and at

WES during the demonstration project for the stay-in-place forming system. These enhancements included improvements to the anchorage system (including eliminating the welding detail for attaching panel anchors to wall dowels), revisions to the reinforcement, addition of lifting devices, elimination of jacking devices, and refinements of the concrete mixture, all of which improved the handling, erection, and design strength of the panels.

The prime contractor for this work was Jackie Bombard, Inc., Galway, NY. All work that would interfere with navigation was required to commence no earlier than 28 November 1991 and be completed no later than 15 April 1992 to coincide with the normal winter closure period for the lock. The required completion date for all remaining work was 24 December 1992.

Concrete Removal

A 10-in.-deep horizontal sawcut was made at the bottom repair elevation for the vertical walls. The deteriorated concrete on the vertical wall was then line drilled 14 in. from the face and removed by controlled blasting. Concrete remaining after blasting as well as concrete on horizontal surfaces and restricted areas adjacent to openings was removed by conventional mechanical methods (i.e. jack hammers, air drills, and rotary concrete abraders).

Panel Fabrication

All precasting operations were performed offsite by a National Precast Concrete Association certified precasting plant (Fort Miller Company, Schuylerville, NY). Formica-faced forms were used to provide a smooth exterior surface. Panel anchors positioned to align with anchor dowels installed in the lock walls were cast into the panel along with reinforcing bars and erection devices (Figure 1). The panel anchors protruded approximately 4 in. from the back of the interior surface. The interior surface was raked to give it a rough finish to enhance bonding with the infill concrete. Riverside river wall panels were also fabricated with holes to accommodate she-bolts during installation.

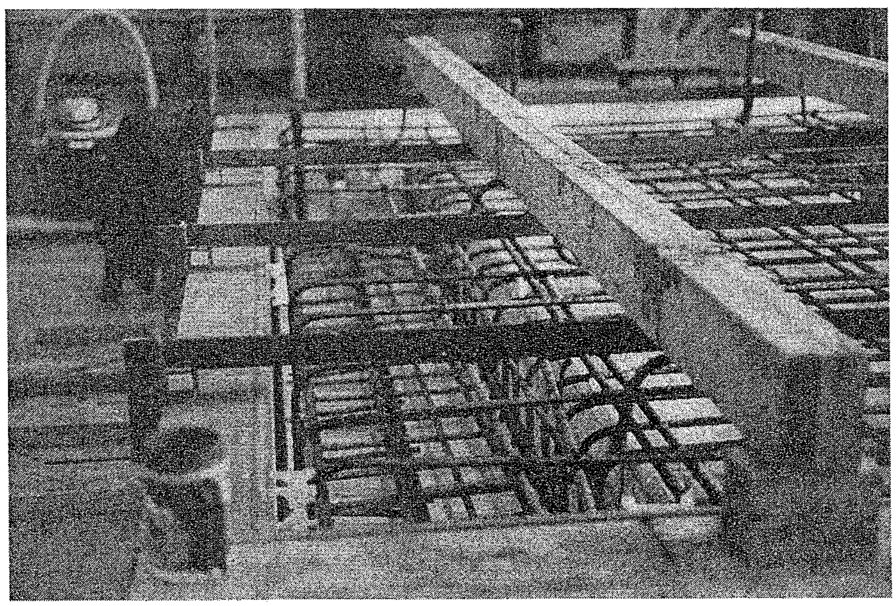


Figure 1. Precast panel fabrication, showing reinforcement and panel anchor.

Completed panels were labeled in accordance with the erection shop drawings and either transported or stored depending on the erection schedule. Lock chamber panels were fabricated 7.5 in. in depth, 11 ft, 10 in. in width (to facilitate highway transport), and various lengths (longest 23 ft, 10 in.) depending on the width of the placement monolith. Riverside river wall panels were fabricated 5.5 in. in depth, 5 ft in width for the first course and 10 ft in width for the second and third courses, and various lengths (longest 22 ft, 8.5 in.) depending on the placement monolith. Specialty panels were cast to accommodate ladders and line poles.

Quality control testing for air, slump, temperature, compression, and yield strength was performed at the plant. The contract specifications included a provision for the optional use of silica fume to meet the required 7,000 psi design strength. The precaster received approval for the use of Type C flyash in lieu of silica fume. Test results indicate that the approved concrete mixture provided high strength (average 9,000 psi) and excellent resistance to cycles of freezing and thawing.

Panel Installation — Lock Chamber

Following installation of dowels, the precast concrete panels were hoisted by crane from the landside of the lock to their proper location with the use of erection anchors cast into the panel. The lower course panels were set onto steel shims placed on the sawcut shelf.

The panels were temporarily held in place by contractor- fabricated adjustable holding devices, which facilitated vertical adjustment of the panels (Figure 2). Upon completion of all adjustments and verification of alignment by means of a laser, vertical reinforcing bars were inserted into the opening created between the panel anchors and the L-shaped anchor dowels behind the panel (Figure 3).

The original design called for steel plates and a series of shebolts anchored into the existing concrete at the top and bottom of the panels to resist form pressures during infill concreting operations. An alternate contractor-designed supporting system was approved which used vertical steel strongbacks (two connected C6 steel channels attached to shebolts anchored 1 ft, 5 in. into the existing wall at the top and bottom of the panel, with steel plates in contact with the panel surface at the top, midpoint, and bottom) to resist form pressures during infill placement (Figure 4). Wooden bulkheads were installed at the panel joints to prevent migration of infill concrete out of the forms. Infill concreting of the +/- 4.5-in. void behind the panel was limited to one-half of the vertical panel height (6 ft) by design to avoid excessive form pressures on the panels and resultant cracking; however, several panels could be filled during the same pouring operation.

Following completion of infill concreting, wooden bulkheads, strongbacks, panel hangers, and shebolts were removed and holes were filled with nonshrink grout.

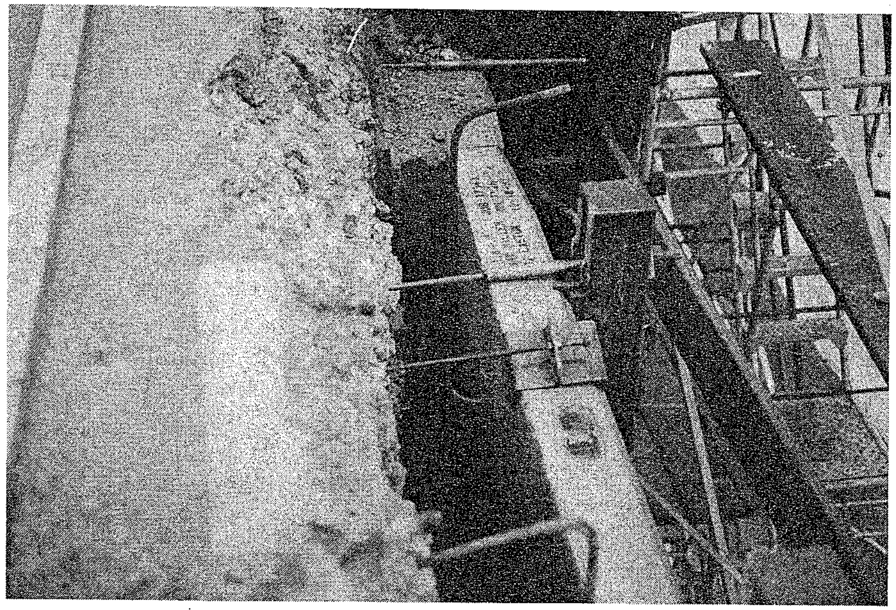


Figure 2. Installation of lock chamber panel, showing adjustable holding device, lifting inserts, strongback, shebolt, panel anchor, wall dowels, vertical reinforcing rod.

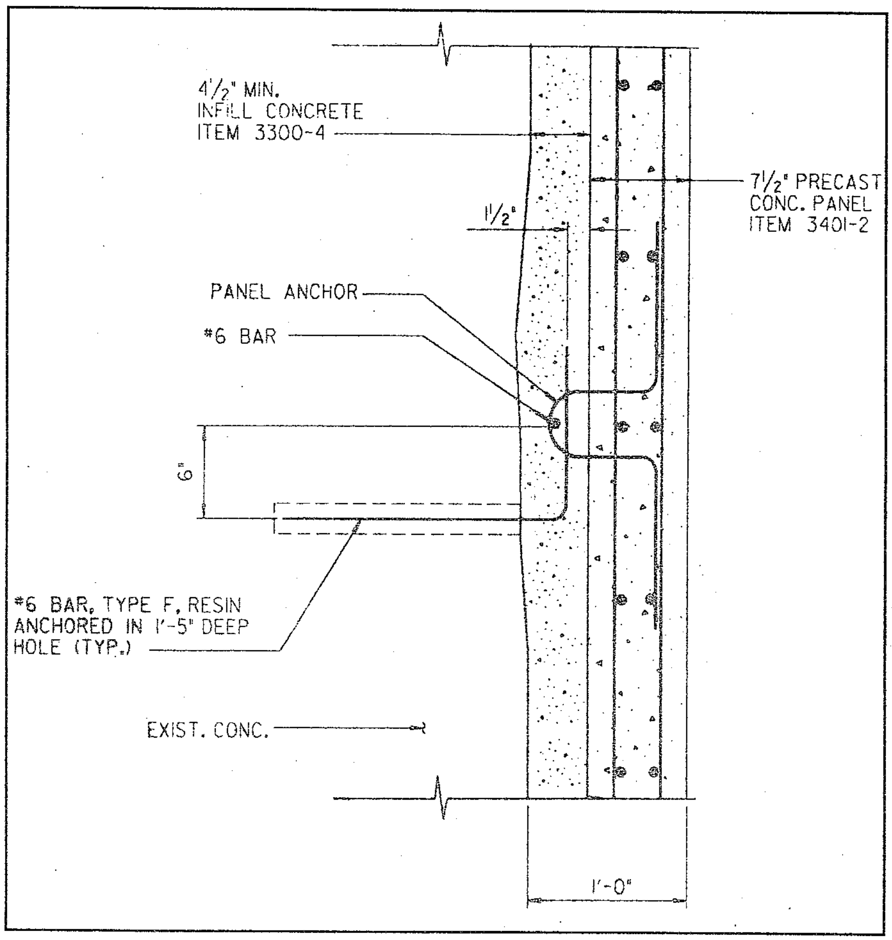


Figure 3. Lock chamber panel anchoring system.

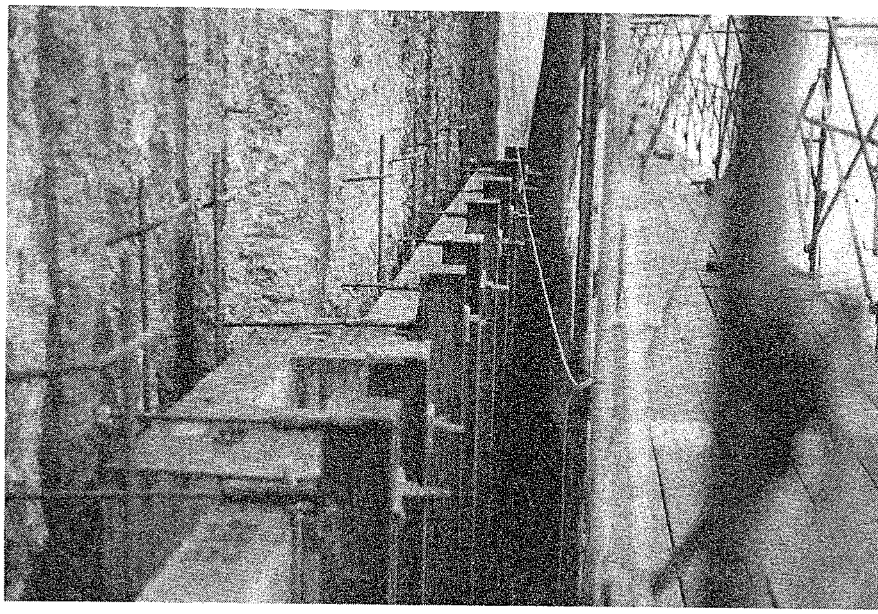


Figure 4. Lock chamber specialty panel, showing strongback system in place prior to infill placement operations.

Upper course panels were erected in the same manner with steel shims placed between the panels to support the upper panels during anchoring and to provide a void for infill concrete. Although as many as 10 panels could be hoisted and temporarily secured during an 8-hr day, final alignment, installation of strongbacks and bulkheads, and related tasks resulted in an overall productivity rate of approximately 2 panels per day.

Panel Installation — River Wall

Prior to the installation of panels, loose concrete was removed from the riverside of the river wall by pressure washing. The contractor fabricated and installed an adjustable steel bulkhead on the bottom of the first course panels to provide a seal with the existing wall. The contractor also fabricated a template to match the slope of the river wall to enable panels to be rigged at the proper angle prior to hoisting them into position. The steel strongback securing system used for the lock chamber panels was not required for the riverside panels; shebolts anchored 2 ft, 5 in. into the wall through prefabricated holes in the panel were used to secure the panels and resist form pressure during infill placement (Figure 5).

The first course panels had to be erected within the tidal zone, necessitating special requirements for underwater dowel installation, panel erection, and infill concreting (including the use of an antiwashout admixture). The first course of panels was designed for a 5-ft height to facilitate erection in the water. The next two courses of panels were fabricated for 10-ft heights. The contractor installed reinforcing bars between the panel anchors and wall dowels and placed infill concrete in the void behind the panels, similar to the lock chamber work.

Completion of Work

All work within the lock chamber was completed prior to the start of the normal navigation season (1 May 1992). All other items of work that did not interfere with navigation, including those on the riverside of the river wall and aprons, were completed by 26 December 1992, as required.

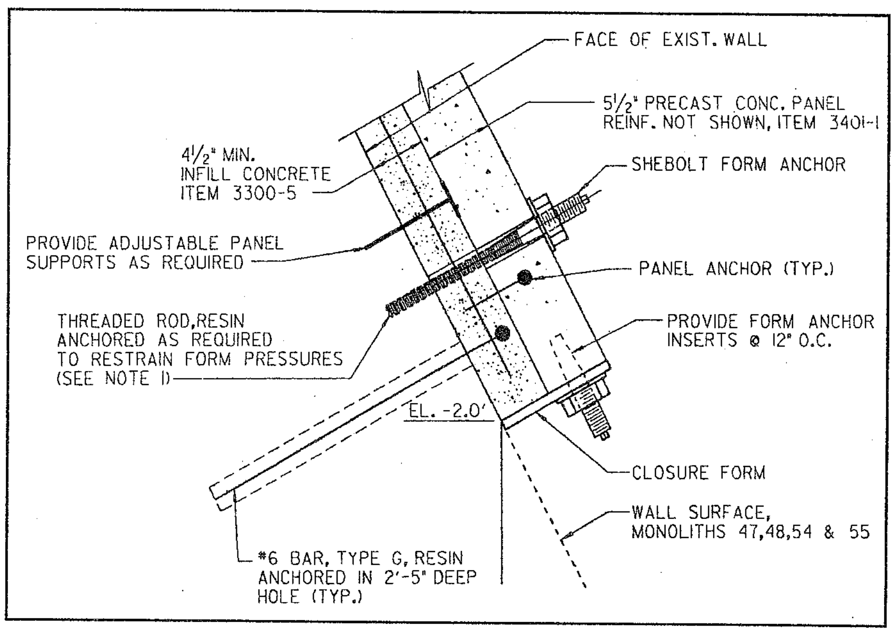


Figure 5. River wall panel support system.

Conclusions

The use of the stay-in-place precast concrete forming system for lock wall repairs provided a cost-effective and time-efficient alternative to conventional cast-in-place concreting. The high-strength, abrasion-resistant, and virtually crack-free surface of the precast panels is far superior to those surfaces previously repaired using conventional concreting methods at the Troy Lock. The reduced forming requirement (erection, insulating, removing, and movement) provided time and cost benefits. Jacketing of the riverside of the river wall also resulted in substantial savings by eliminating concrete removal costs. Additional savings resulted from the ability to install the riverside precast panels without dewatering. The results achieved at the Troy Lock and Dam project demonstrate the advantages that stay-in-place forming can provide for similar concrete repair applications.

For additional information, please contact William Petronis at (518) 273-0870.

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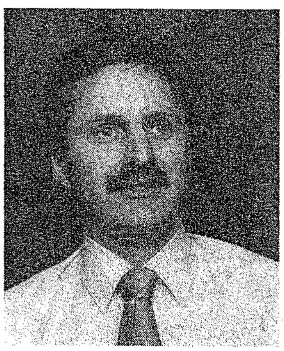

William Petronis *has been Chief of the Operations Division, Albany Field Office, New York District, since 1984. He has primary responsibility for managing civil works O&M activities for projects within the Field Office's geographical area of responsibility. From 1978-1984 he was with the Engineering and Operations Division, New Orleans District. He received a B.S. degree in civil engineering from the University of Buffalo and is a registered Professional Engineer in the States of New York and Louisiana.*

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Video Report Available

Video Report REMR-CS-4, "Comparison of Cast-in-Place Concrete Versus Precast Concrete Stay-in-Place Forming Systems for Lock Wall Rehabilitation," is now available. The report documents details of precast panel production and panel installation, outlines the advantages and limitations of the precast system, and compares the results of rehabilitation with precast concrete and cast-in-place systems. Rehabilitation of Troy Lock is featured.

The videotape is approximately 20 minutes long and is available on 1/2-in. cassette through inter-library loan from the Waterways Experiment Station Library. For additional information, call (601) 634-2355.

Microtunneling Tests at Waterways Experiment Station

by

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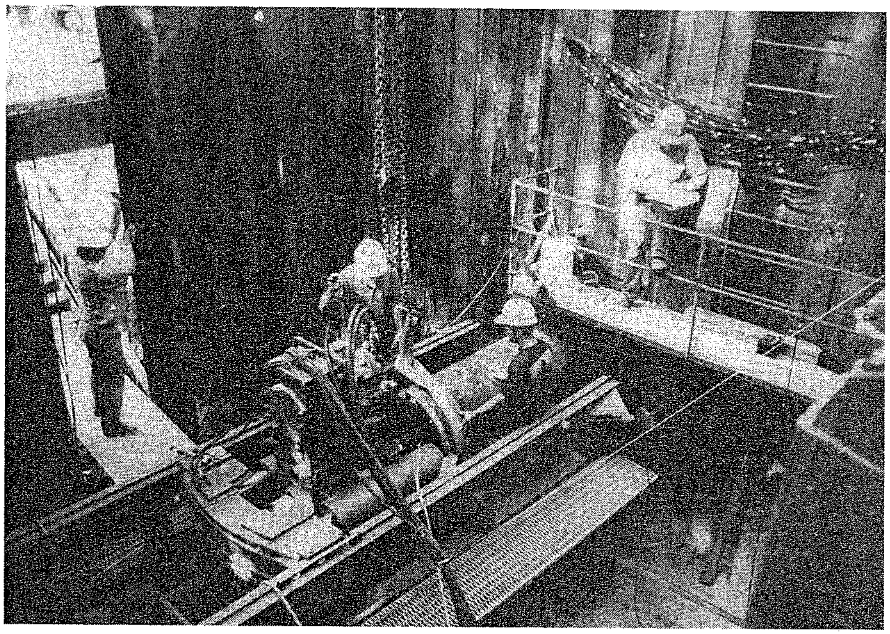


Soltau microtunneling machine

Traditional open trenching methods of installing or rehabilitating underground utilities in developed areas can be costly and disruptive; they frequently interfere with traffic, create inconveniences to adjacent businesses, and may result in the need for expensive replacement of trenched surfaces. To alleviate these adverse conditions, trenchless technology can offer cost-effective and safe alternatives for infrastructure renewal and rehabilitation. One form of trenchless technology currently being investigated at Waterways Experiment Station (WES) is microtunneling.

The microtunneling test program is part of a research effort to evaluate methods and materials to install and rehabilitate underground utilities. This study is a Corps and industry cost-shared project conducted under the Construction Productivity Advancement Research (CPAR) Program. The laboratory partner is the Geotechnical Laboratory, WES, and the industry partner is the Trenchless Technology Center, Louisiana Tech University.

Definition of Microtunneling

Microtunneling can be described as a remotely controlled, guided, pipejacking process. The guidance system is usually a laser mounted in the jacking pit with a target mounted inside the articulated steering head of the microtunneling machine. The microtunneling process does not require personnel entry. Since the same type of system can be used to install almost any size pipe, there is no size constraint on the process. The process can be successfully used under a variety of ground conditions, ranging from soft soils to rock, including mixed face conditions and boulders, both above and below the water table. A typical microtunneling machine is shown in Figure 1.

Test Bed Design and Construction

The design objectives for the experiment and test bed conducted at WES were:

- To provide realistic but challenging ground conditions to test the limits of microtunneling capabilities.
- To provide ground conditions that varied in a controlled fashion and to minimize boundary effects so that performance could be correlated with known ground conditions.
- To allow measurements to be made for evaluation of machine-ground interaction, including cutterhead torque, jacking thrust, effects of lubricants on pipe

loads, stresses and strains, and ground settlement and heave.

- To allow evaluation of two different types of microtunneling systems, i.e. auger and slurry, under the same ground conditions.

The microtunneling tests were performed at a specially constructed test facility built at WES. Figure 2 shows an end view of the test facility, which is 340 ft long, 16 ft wide, and 13 ft deep. It was constructed, by first excavating a trench into which six different types of soils were placed and compacted in approximately 60-ft-long sections. Figure 3 is a view of the top of the test bed. The soil profiles included a lean silty clay (loess), a dry sand, highly plastic buckshot clay, wet sand, clay gravel, and silt.

Installation of Strain Gages in Jacking Pipes

Strain gages were installed in eight of the 24-in.-ID glass reinforced plastic (GRP)jacking pipes to measure strains along the tunnel length and around the pipe circumference. Jacking loads generated in the drive pit were resisted by soil shear stresses along the installed pipe string and soil pressure against the face of the tunneling machine. Consequently, the thrust at the face of the machine was only a fraction of the force generated by the jacks in the drive pit. The strain gages were installed in the first two pipes that were to be positioned behind the steering head for each tunnel, in a third pipe to be located at approximately the midpoint of the drive, and in a fourth and final pipe to be installed near the end of the drive. This arrangement was intended to provide continuous strain data on a

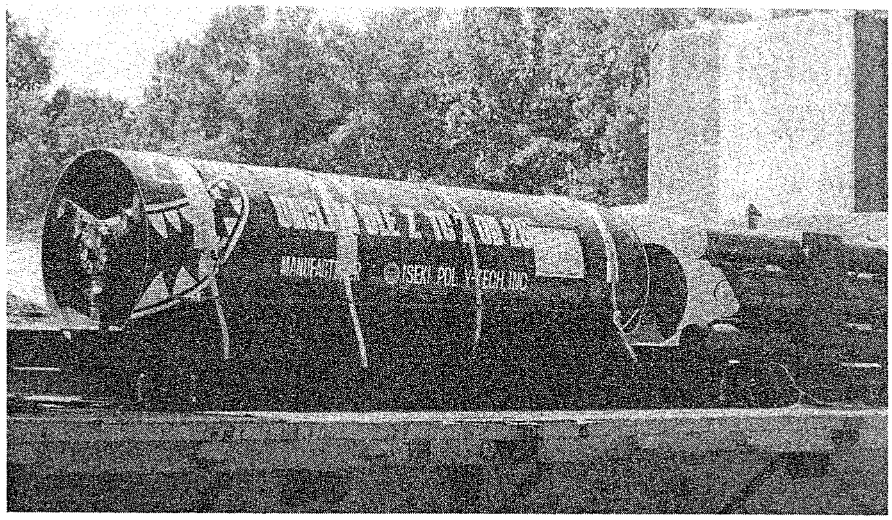


Figure 1. Iseki microtunneling machine

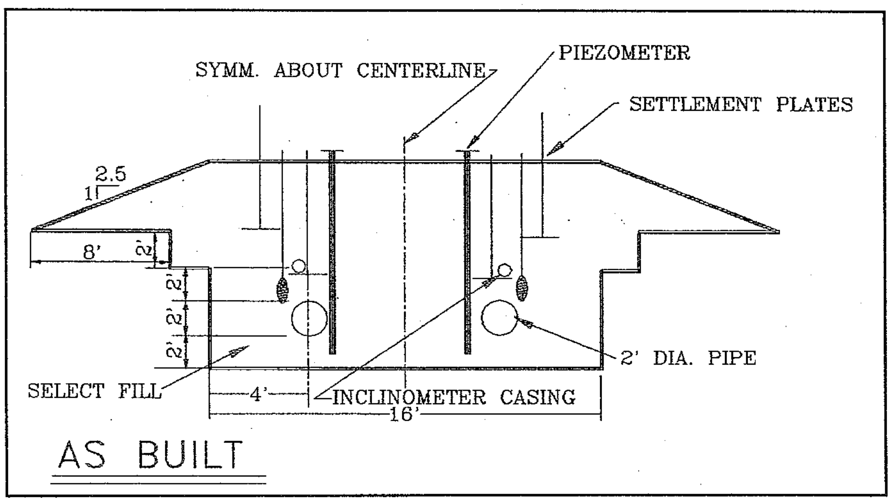


Figure 2. End view of test bed

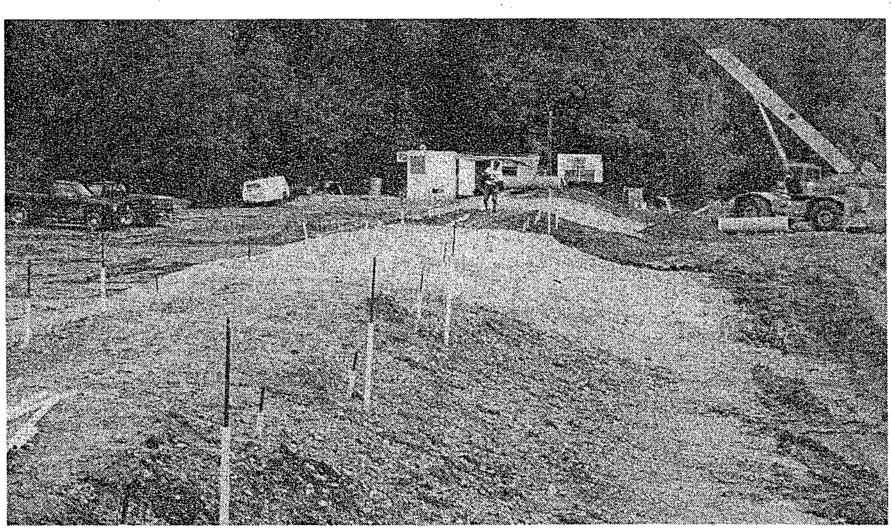


Figure 3. Top of test bed

given pipe as it was pushed from the drive pit and to provide measurements of strain at key points between the drive pit and steering head. Figure 4 shows the GRP pipes with strain gages installed.

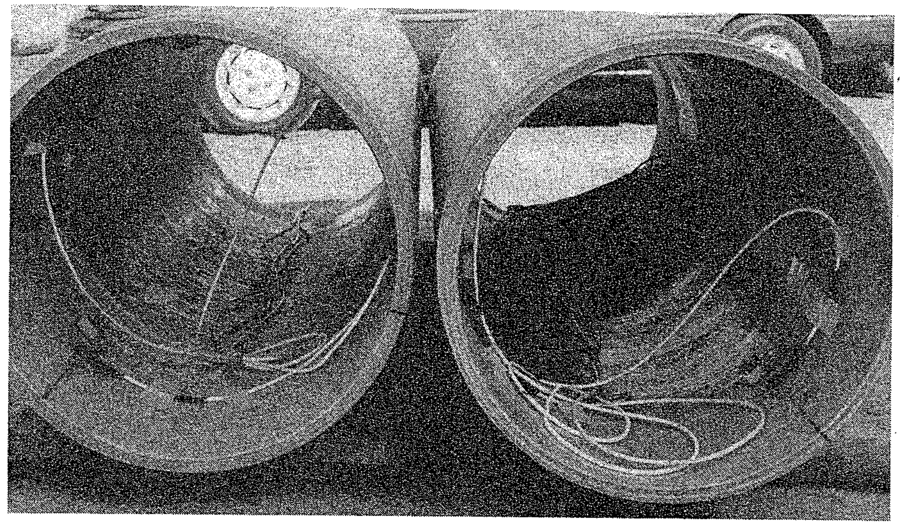


Figure 4. GRP pipes with strain gages installed

Microtunneling Tests and Preliminary Results

Auger Microtunneling Test

The auger microtunneling tests were conducted between 8-26 September 1992, using a Soltau RVA250 machine, with an outer diameter of approximately 26.3 in., furnished by American Microtunneling, South Daytona, FL. The manufacturer's specifications provide a full description of this machine. The control panel was set up adjacent to the drive shaft and was fitted with a computerized data acquisition system to monitor and record machine performance parameters, including position of the steering head, torque, thrust, and steering jack pressures. This data acquisition system was set up to record data at 3-min intervals, keyed to time, date, and distance. The jacking frame was fitted with 250-ton capacity jacks. The jacking pipe used was 10 ft long, 24 in. ID GRP, rated at 115 tons design load. Four of the jacking pipes were strain-gaged. Strain data were recorded continuously during the jacking operation. A computer data acquisition system read all strains 60 times per minute, and the average of the peak strains over that minute for each channel was recorded. This setup was intended to correlate peak jacking thrusts with strains, while avoiding extraneous data from unloading cycles with the jacking system.

During the auger microtunneling trials, approximately 312 ft of tunneling was performed over a period of 12 days for an average of 26 ft/day. Mobilization and installation required 6 days. Demobilization and site cleanup required 3 days.

Slurry Microtunneling Test

The slurry microtunneling tests were conducted between 28 September and 31 October 1992, using an Iseki Unclemole Z TCZ, with an outer diameter of 26 in. furnished by Iseki, Inc., San Diego, CA. Again the manufacturer's specifications provide a complete description of this machine. The control panel was set up adjacent to the drive shaft. Machine performance data were displayed on gages on the control panel and recorded manually as each pipe was pushed. Recorded data included penetration rate, torque, thrust, slurry flow rates, and position of the steering head. The jacking frame was fitted with three stage Molemeister 150-tonne (or 165-ton) capacity jacks. The jacking pipe was 8 ft long, 24 in. ID by 25.8 in. OD GRP, rated at 115 tons design load. Four of the pipes had strain gages installed as described previously. As with the auger machine test, strain data were recorded continuously during the jacking operation.

During the slurry machine test, approximately 216 ft of tunneling was performed over a period of 21 days, for an average of 10 ft/day. Mobilization and installation required 6 working days, while demobilization and site cleanup required 4 working days.

Preliminary Results

Alignment and Grade Control

The computerized data acquisition system used with the auger machine recorded and plotted the position of the steering head in relation to the position of the laser at 3-min intervals during the drive. These measurements indicated that maximum horizontal and vertical deviations were less than 1-1/4 in. In fact, deviations were typically less than 1/2 in. The position of the steering head was surveyed at the end of the test and was within 1/2 in. of its intended location, both vertically and horizontally.

The deviations of the slurry machine from the laser beam to the actual position of the steering head were monitored continuously by closed circuit television. The positions were recorded manually at least twice as each pipe was jacked. The deviation from planned line and grade was less than 1 in. in all cases, and typically less than 1/2 in. The position of the steering head, when excavated at Sta 2+16, was within 1/2 in. of its intended position, both vertically and horizontally.

Ground Movements

Ground movements were measured after each pipe was pushed during both the auger and slurry tunneling machine tests, using the settlement plates at three levels, the horizontal inclinometers 2 ft above the crown of each tunnel, and surface survey points. For the slurry machine test, measured ground movements were less than 1/4 in. throughout the 216-ft drive, as measured at all levels, and were typically within the level of instrument precision. For the auger machine test, significant ground movements were measured at a few locations, specifically in the flooded sand section. These ground movements are being evaluated to determine possible causes. It appears that the large settlements at the two locations in the flooded sand are related to poor installation practice of a piezometer that was jetted in approximately 2 ft from the tunnel at Sta 2+32 and a 1-1/2-hr. shutdown during an electrical storm at Sta 1+80. At both locations, water carried sand into the face of the microtunneling machine, causing localized ground settlements of 1 ft or so at the surface. After these incidents, compressed air was applied at the face of the machine to stabilize the flooded sand, and ground control was satisfactory for the remainder of the drive.

Jacking Loads and Pipe Performance

Pipe strain data are being evaluated to determine soil resistance at the face of the machine and along the sides of the pipe in each type of soil. A typical strain data plot is shown in Figure 5. These strain data will be used together

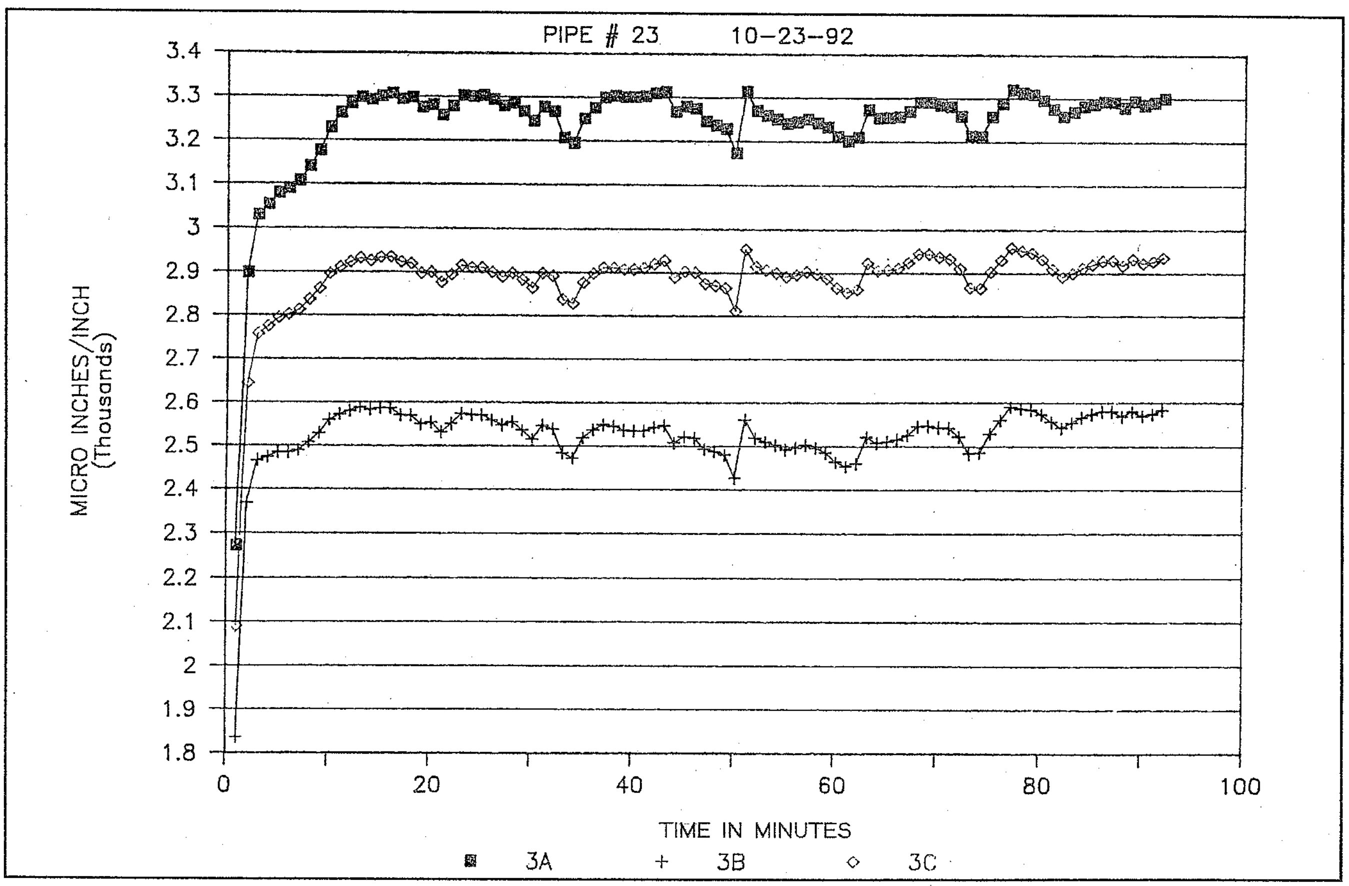


Figure 5. Strains measured in jacking pit while pushing pipe no. 23 during slurry microtunneling test. Gages A, B, and C located at 12, 8, and 4 o'clock, respectively

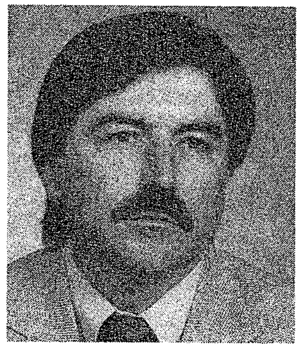
with additional data to improve methods for estimating jacking loads under various ground conditions for the different types of machines evaluated.

Conclusions

The factors that govern successful performance or that can lead to catastrophe on microtunneling projects are not clearly understood. This situation is especially critical in the United States, where this technology is just beginning to emerge.

This microtunneling research was intended to help bridge knowledge and experience gaps. These tests provided a unique opportunity for an impartial documentation of the performance of both auger and slurry systems in ground conditions that varied in a controlled manner, to provide increasing challenges from the beginning to the end of the test bed. Properties of the various soil profiles in the test bed were well documented, and the experiment was designed to allow correlations to be developed between these known ground conditions and machine performance. Even though some problems were encountered in the execution of the tests, a great deal was learned. Extensive measurements and observations were recorded and will form the basis for developing these correlations and predictive methods. Credible case history data will be sought to complement and verify these observations and correlations. Some research has been performed in Japan and Europe that can provide valuable insights, and this information will be used to the maximum extent possible to complement U.S. experience and these tests.

For additional information, contact Dave Bennett at (601) 634-3974.



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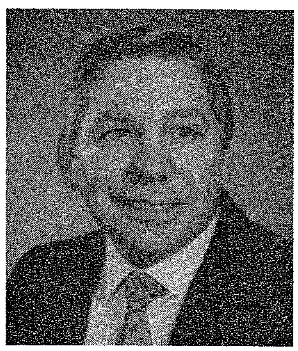

__Tom Iseley__ is an associate professor of civil engineering at Louisiana Tech University and is the Director of the Trenchless Technology Center. He is a registered Professional Engineer in Louisiana and Mississippi and has over 25 years of experience in the design and construction of underground utility systems. Iseley has B.S.C.E. and M.B.A. degrees from the University of Alabama in Birmingham and a Ph.D. in civil engineering from Purdue University. He is a member of the North American Society for Trenchless Technology, the American Undergound Space Association, the National Utility Contractors Association, the National Institute for Engineering Management and Systems, and the Construction Equipment and Techniques Committee of the American Society of Civil Engineers.

__Pat Taylor__ began his career at WES in 1973 as a Civil Engineering Technician. He received a B.S. degree in civil engineering in 1980 and is currently enrolled in the WES Graduate Institute, where he is working toward an M.S. degree in geotechnical engineering. From April 1988 until January 1991, he served a tour of duty with the Construction Division of the Japan Engineering District. He has developed recognized expertise in foundation grouting and geotechnical instrumentation. He serves on both Corps and CAGE grouting committees. He is a registered Professional Engineer in the State of Mississippi.

Rehabilitation of Permeable Breakwaters and Jetties by Void Sealing, Port Everglades, Florida, South Jetty

by

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Stone and concrete units are often used in coastal structures for their durability and wave energy dissipative characteristics. However, large voids may develop between units as the result of cross-sectional design or structural degradation through time (e.g. loss of core stone or settlement) and may impair structure functionality. Excessive wave energy passing through the structure may cause damage to moored vessels, and sediment penetrating voids may shoal navigation channels. Such a structure designed to retain a beach fill may allow fill material to be transmitted through the interconnecting voids.

A grout made from a portland-cement slurry and a sodium-silicate solution was used to seal the voids in the Port Everglades, Florida, south jetty, and a sodium silicate-diacetin mixture was used to stabilize the sand layer beneath the rubble-mound structure. The purpose of sealing the jetty was to eliminate transmission of sand through the structure into the navigation channel. Prior to sealing, "man-sized" voids existed in the structure, impairing its function as a terminal groin for beach fill placed south of the structure. Continual erosion of the beach located immediately downcoast, owned by the State of Florida, prompted Broward County and the State to fund a jetty rehabilitation project and subsequent beach fill of the adjacent south beach. A monitoring plan to ascertain the effectiveness of the project through a field evaluation was conducted by the Waterways Experiment Station (WES), Coastal Engineering Research Center, with the cooperation of Broward County, the State and the sealing contractor. Supplemental monitoring by WES is being conducted to determine long-term durability and effectiveness of the sealant materials.

Project Location

Port Everglades is a Federal navigation project located approximately 23 miles north of Miami Harbor (Figure 1). The south jetty is a 1,000-ft-long rubble-mound structure constructed of 5- to 7-ton stone, approximately 30 ft wide at the base and 11 ft wide at the crest. The structure was constructed in 1926-1927 and rebuilt in 1940. Divers reported in 1985 that the structure was ill-maintained and in disrepair, evidenced by numerous large gaps and voids near the base. It was estimated that at least 5,000 cu yd/year of sediment passed through the structure and accumulated in the navigation channel. Nourishment of the beach south of the jetty with 500,000 cu yd of material was planned, and sealing of the south jetty was determined to be a cost-effective alternative to allowing 5,000 cu

Figure 1. Port Everglades, Florida

yd/year of sand to pass through the jetty into the navigation channel. Sealing was performed in 1988 prior to the beach-fill placement in 1989.

Drilling and Sealing

An asphalt fishing walkway on the crest of the jetty provided an ideal foundation from which to conduct the drilling and sealing operation. The 3-1/2-in.-diam holes were drilled at 3-ft spacings along the center line of the structure to specified elevations beneath the base of the jetty (Figure 2). Initially, every other hole (primary) was sealed with the sodium silicate-cement mixture, which began to "set up" in 70 to 80 sec. After the primary holes were sufficiently strengthened (approximately 24 hr), secondary holes between the primary holes were sealed. Usually the quantity of sealant required for the secondary holes was on the order of only 10 percent of the adjacent primary hole quantity. It was expected that this procedure would result in a 4-ft-wide sealant curtain longitudinally within the structure. After the secondary holes had strengthened, the primary holes were redrilled, and a quick-set sodium silicate-diacetin sealant designed to permeate any sand-filled areas beneath the structure was pumped into the holes. The pumping requirements were at least 30 gal/min of sealant and at least 100 lb/sq in. of pressure.

Monitoring Program

The purpose of the WES monitoring program was to evaluate the degree of structure permeability before and after sealing. Dye observations and current measurements were used to quantify the rate of net fluid movement through the structure. Sand transport could not be quantified because measurable quantities moved through the structure only during storm events. The monitoring program consisted of (a) a preconstruction experiment to qualitatively and quantitatively evaluate the presealing condition of the structure, (b) during construction observations of the drilling and sealing techniques, and (c) a postconstruction experiment

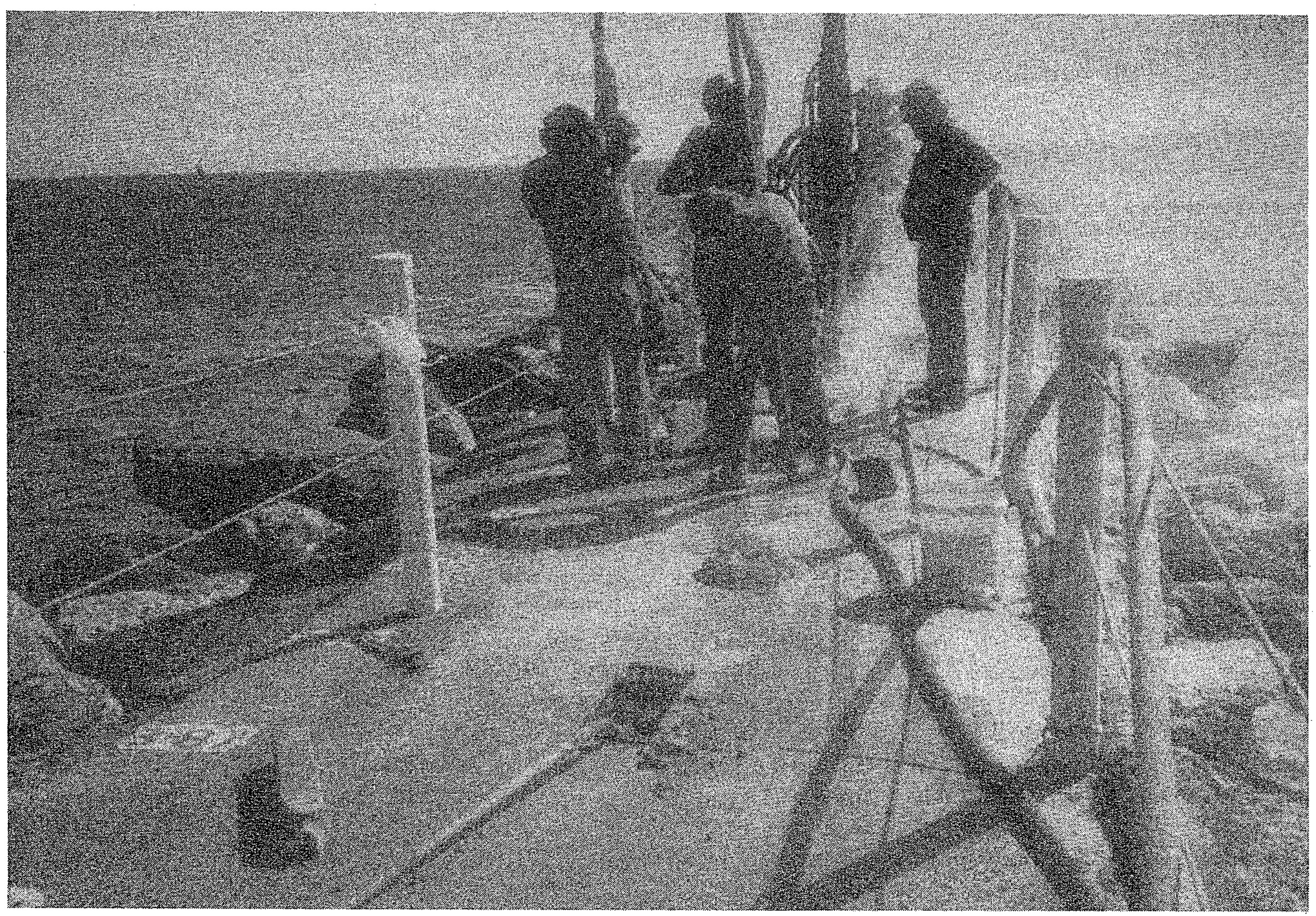

Figure 2. Sealing Port Everglades, Florida, south jetty

Figure 3. Dye being placed down unsealed drill hole

to repeat tests conducted preconstruction so that the degree to which structure sealing occurred could be assessed.

Preconstruction

The preconstruction experiment, conducted during the month prior to actual sealing activities, consisted of collecting current magnitude and direction data in three structure voids and measuring the time rate of dye dispersal through the structure with a fluorometer. Data were collected for an hour, beginning 1/2 hr prior to either peak flood, peak ebb, or slack flow. Rhodamine dye was placed in structure voids on the south side of the jetty during peak flood conditions, and on the north side during peak ebb flow. Surface water samples were taken every 10 min for an hour at three stations on both the north and south side of the structure. Water samples also were taken at the seabed to determine if dye transmission was uniform with depth. One-inch diameter electromagnetic current meters were mounted in three major voids and were cabled to shore, where data were stored in a portable computer.

During Construction

An inspection of the underwater portion of the structure was conducted during sealing operations. Small portions of sealant could be seen in smaller voids located farther in the structure. Dye samples were placed down unsealed drill holes to observe pattern of dye dispersion through the structure voids (Figure 3). The dispersal of dye from an unsealed secondary hole adjacent to two unsealed primary holes (concentration 92 ppm) was compared with the dispersal of dye from an unsealed secondary hole adjacent to two sealed primary holes (concentration 0). Transmission decreased by 100 percent, indicating that sealing of the structure was extremely successful in filling structure voids.

Postconstruction

Because of the nonuniformity in flow conditions between the pre- and postsealing events, flow conditions had to be analyzed individually. It was determined that there was at least a 90-percent probability that a significant difference existed in flow through the structure. Postconstruction evaluations conducted during the monitoring of the Port Everglades, Florida, south jetty sealing project indicated that the transmissibility of the structure has been significantly reduced. The change in average current vector direction, measured during peak flood, high-water slack, and peak ebb conditions, indicated that the structure was much more reflective in the postsealing condition and, therefore, less transmissible.

Continuing Research

Considering the very low compressive strengths of the sodium silicate-cement and sodium silicate-diacetin mixtures in the laboratory, and the observed rapid erosion and deterioration of such specimens at long-term field exposure stations presently being monitored, it is imperative that continued monitoring of completed prototype sealing projects be performed. A re-evaluation of the effectiveness of the cementitious and chemical sealants used at Port Everglades, Florida, will be conducted during FY 93 to try to ascertain the actual useful life of these sealing efforts so that true economic benefits and cost comparisons of all alternatives are realistic. The analysis will include evaluations of beach and offshore bathymetric changes, shoal material accumulations in the navigation channel for correlation with potential transmission through the structure, and dye transmission observation during flood and ebb cycles.

For additional information, call Lyndell Hales at (601) 634-3207.

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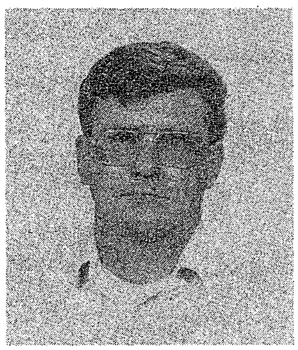

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