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Monitoring Completed Navigation Projects Program

Lock Wall Expedient Repair Demonstration Monitoring, John T. Myers Locks and Dam, Ohio River

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Final report

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Abstract: The lock chambers at John T. Myers Locks and Dam are very susceptible to wall damage from barge impact and abrasion. The majority of the damage includes gouges and spalls in the concrete adjacent to a lock wall armor strip. Many of the damaged regions are next to a vertical joint. The majority of the damage occurs in the 1,200-ft lock chamber as opposed to the 600-ft chamber. Innovative expedient repair methods and techniques that will not disrupt river navigation were evaluated and demonstrated. Non-traditional materials for repair and rehabilitation of concrete structures, such as fiber reinforced polymer (FRP) composites, were evaluated for specific application to inland hydraulic navigation locks and dams. This Monitoring Completed Navigation Projects (MCNP) expedient lock wall repair demonstration consisted of five aspects: (1) evaluating the John T. Myers Locks and Dam wall armor system, (2) monitoring the repair of the 600-ft chamber upper land-wall approach vertical joint, (3) monitoring the repair of the 600-ft chamber lower land-wall approach vertical joint, (4) monitoring the repair of the 1,200-ft chamber upper-river approach wall, and (5) evaluating the feasibility of using fiber reinforced polymer (FRP) composites for inland hydraulic structure application.

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Preface

The studies reported herein were conducted as part of the Monitoring Completed Navigation Projects (MCNP) program under MCNP Work Unit A1060, "John T. Myers Locks and Dam." Overall program management of the MCNP is provided by Headquarters, U.S. Army Corps of Engineers (HQUSACE). The Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), is responsible for technical and data management and support for HQUSACE review and technology transfer. The HQUSACE program monitor for the MCNP Program at the time of this study was James E. Walker, Chief, Navigation Branch, HQ. W. Jeff Lillycrop, CHL, was the ERDC Technical Director for Navigation. MCNP program manager during the conduct of this study was Dr. Lyndell Z. Hales, Hydraulic Engineer, Technical Programs Office, CHL.

This research was conducted during the period October 2006 – September 2010 under the general supervision of Thomas W. Richardson, former Director, CHL, and Dr. William D. Martin, present Director, CHL. MCNP Principal Investigator was Dr. Stanley C. Woodson, Research Structural Engineer, Geotechnical and Structures Laboratory, ERDC. MCNP District Team Member was J. Rick Lewis, Chief, Maintenance Section, Technical Support Branch, Operations Division, U.S. Army Engineer District, Louisville. Lewis and Dr. Woodson designed and contributed significantly to the execution of this study. David W. Scott and James E. McDonald performed fundamental background evaluations of the John T. Myers Locks and Dam armor system for designing the expedient repairs. They also developed the technical specifications for performing the repairs. Drs. H. V. S. GangaRao and P. V. Vijay investigated the feasibility of using fiber reinforced polymer composites for making repairs to hydraulic navigation structures.

At the time of publication of this report, COL Kevin J. Wilson was Commander and Executive Director of ERDC. Dr. Jeffery P. Holland was Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02832	cubic meters
feet	0.3048	meters
inches	2.54	centimeters
miles (U.S. statute)	1.609344	kilometers
pounds	453.592	grams
pounds per square inch	70.307	grams per square centimeter
square feet	0.0929	square meters
tons	907.185	kilograms

1 Introduction

Monitoring Completed Navigation Projects (MCNP) program

The goal of the Monitoring Completed Navigation Projects (MCNP) program (formerly the Monitoring Completed Coastal Projects (MCCP) program) is the advancement of coastal and hydraulic engineering technology. The program is designed to determine how well projects are accomplishing their purposes and how well they are resisting attacks by their physical environment. These determinations, combined with concepts and understanding already available, will lead to the creation of more accurate and economical engineering solutions to coastal and hydraulic problems, strengthening and improving design criteria and methodology, improving construction practices and cost-effectiveness, and improving operation and maintenance techniques. Additionally, the monitoring program will identify where current technology is inadequate and where additional research is required.

To develop direction for the program, the U.S. Army Corps of Engineers (USACE) established a committee of engineers and scientists. The committee formulated the objectives of the program, developed its operation philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of the committee's efforts was a prioritized listing of problem areas to be addressed. This is essentially a listing of the areas of interest of the program.

USACE offices are invited to nominate projects for inclusion in the monitoring program as funds become available. The MCNP program is governed by Engineer Regulation 1110-2-8151 (USACE 1997a). A selection committee reviews and prioritizes the nominated projects based on criteria established in the regulation. The prioritized list is reviewed by the program monitors at HQUSACE. Final selection is based on this prioritized list, national priorities, and the availability of funding.

The overall monitoring program is under the management of the Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), with guidance from HQUSACE. An individual monitoring project is a cooperative effort between the submitting District and/or Division office and CHL. Development of monitoring plans, and

conduct of data collection and analyses are dependent upon the combined resources of CHL and the District and/or Division.

Project location

John T. Myers Locks and Dam (Figure 1), originally Uniontown Locks and Dam, is located on the Ohio River at Mile 846.0 about 3.5 miles downstream from Uniontown, Kentucky. The project was approved by the Secretary of the Army on 17 September 1958 under the authority of Section 6 of the Rivers and Harbors Act, approved 3 March 1909, as amended. The locks are located on the right descending bank and have chamber sizes of 110-ft by 600-ft for the landside lock and 110-ft by 1,200-ft for the riverside lock. The lift of the locks at normal pool is 18 ft. The dam consists of a series of 10 gates, each 110-ft-wide by 32-ft-high, located between piers.

Figure 1. John T. Myers Locks and Dam, Ohio River, near Uniontown, Kentucky.

The John T. Myers Locks and Dam is an important facility for providing navigation capability for the Ohio River, a major artery for commercial navigation in the United States. The facility provides 69.9 miles of navigable pool from Newburgh Locks and Dam (Mile 776.1) to its location at Mile 846.0. The purpose of John T. Myers Locks and Dam was to replace the low

lift Locks and Dams 48 and 49 to insure continued efficient operation of the system.

At John T. Myers Locks and Dam, future major repairs to the main 1,200-ft chamber, associated with heavy use and age, will force greater reliance on the inadequately-sized 600-ft auxiliary chamber. This will result in accelerating transit costs. The John T. Myers and Greenup Locks Improvements Interim Feasibility Report, a product of the Ohio River Mainstem Study, recommends a 600-ft extension of the existing 600-ft auxiliary chamber and a miter gate quick-changeout system. This project was authorized for construction by the Water Resources Development Act of 2000.

Statement of the problem

The lock chambers at John T. Myers Locks and Dam are very susceptible to wall damage due to the large amount of traffic passing through these locks. Approximately 75 million tons of commodities are shipped through these locks annually. Consequently, the wall armor system exhibits a large amount of damage. The majority of the damage includes gouges and spalls in the concrete adjacent to an armor strip. Many of the gouges are next to a vertical joint (Figure 2). In addition to the spalls and gouges, wall armor separation exists at several locations (Figure 3).

The majority of the armor damage occurs in the 1,200-ft lock chamber as opposed to the 600-ft chamber. The armor damage is a result of a combination of impact and abrasion by commercial barge traffic that typically uses the 1,200-ft chamber. Wall armor separation is vulnerable to “catching” on protruding metal on barges. This is a special concern when barges have protection for themselves.

The most common type of armor-related damage is the loss of concrete (Figure 4). This type of damage can lead to more serious problems such as broken armor strips, since the concrete provides stability and stiffness to the steel systems. Additionally, gouges in the concrete can cause severe discontinuities in the lock wall surface. These discontinuities often result in protruding armor. If a barge or other vessel strikes these protruding edges in such a manner as to engage the armor itself, serious damage could occur to the vessel, the lock wall, or both.

Figure 2. Concrete spalling at vertical joint, John T. Myers Locks and Dam.

Figure 3. Wall armor separation, John T. Myers Locks and Dam.

Figure 4. Loss of concrete between armor strips, John T. Myers Locks and Dam.

Apart from concrete damage and broken armor, another major concern is abrasion of the armor strips. Long-term wear can result in the removal of as much as 0.5 in. of material from the strip armor. This problem is exacerbated when one piece of armor protrudes farther than an adjoining piece. When this occurs, the load from the passing vessel will not be evenly distributed on the armor itself, causing heavy wear to occur at discrete locations. When the armor is worn "flat," it is no longer effective in protecting the surrounding concrete.

Conclusions concerning the original design and construction of John T. Myers Locks and Dam include the following.

- The design did not provide for wall armor protection at the vertical concrete joints. The use of vertical armor protection has proven to be very effective at other lock facilities, and lack of this protection at John T. Myers is causing considerable damage to the concrete.
- Performance prediction technology was not fully used for this project. The knowledge of the advantages of using vertical wall armor was available prior to the design of these lock chambers and should have

been used on these lock chambers. The wear of other armor protection is expected, and repairing of the armor is expected.

- Construction methods used for this project were satisfactory. However, the design parameters were insufficient, causing extensive wear of the lock walls.

Extensive armor repairs are necessary at John T. Myers Locks and Dam due to the lack of vertical armor protection and the high volume of barge traffic. Innovative repair techniques must be researched and demonstrated to achieve repair methods that will not disrupt navigation traffic. It is of great importance that non-disruptive repair methods be developed and demonstrated due to the number of repairs needed and the heavy volume of barge traffic that traverses the lock.

Additionally, non-traditional materials for repair and rehabilitation of concrete structures, such as fiber reinforced polymer (FRP) composites, should be evaluated for specific application to inland hydraulic navigation locks and dams. Resin systems such as polyesters and vinyl esters that are compatible with glass fiber reinforcement should be evaluated for USACE applications.

Regional extent of the problem[1]

The Operations and Maintenance Division, U.S. Army Engineer District, Louisville (CELRL), has as a primary responsibility the repair and upkeep of six navigation locks on the Ohio River. In recent years, maintenance and inspection personnel reported that the protective steel armor on the lock walls underwent significant localized damage and deterioration at the various projects. This armor provides the primary protection for the locks' mass concrete walls against abrasion and impact damage due to the heavy amount of commercial traffic on the Ohio River. The anecdotal information regarding armor deterioration led to concerns that, at certain locations, the armor could fail to perform its role of protecting the lock walls or, in fact, could pose a hazard to river traffic.

As a result of these concerns, CELRL contacted the Geotechnical and Structures Laboratory (GSL) of ERDC in June 1999. Working jointly, engineers from CELRL and GSL developed a program methodology to

[1] This section is extracted essentially verbatim from Scott and McDonald (2000).

address the maintenance and repair of deteriorated wall armor system. The major activities of the program were to

- evaluate the condition of existing armor systems at selected Ohio River navigation projects,
- identify the most prevalent types of failures and determine their causes,
- develop procedures for wall armor remediation at each location,
- demonstrate the repair technology, and
- develop a sustaining program of armor maintenance and repair

The first step in this process was to ascertain the actual scope of the problem and gather qualitative rather than anecdotal information on the level of deterioration and damage present at the various projects. To gather necessary information, researchers from GSL were tasked with performing a field inspection of six locks on the Ohio River.

Field inspection of selected Ohio River armor systems

The term “armor system" refers to the structural steel and surrounding concrete used to protect the mass concrete walls from abrasion and barge impact. Additionally, the presence of protective lock armor reduces the time required for tows to complete a lockage. In contrast, locks of the Panama Canal do not have wall armor; consequently, extreme care must be taken during locking operations to avoid damage to the lock itself. As a result, traffic progresses through the lock at a much slower rate than would be necessary with an armor system.

For a typical installation, lock armor can be divided into two general categories.

- **Horizontal armor:** These armor strips are often referred to as "rubbing strips." This term refers to the horizontal T-sections that are embedded in the monoliths during concrete placement. In some locations, usually on the upstream and downstream approach walls, the strips run the entire length of a monolith. In other locations, the strips are truncated. These are found typically around lock appurtenances such as floating mooring bitts. The armor is anchored into the monolith using staggered studs.
- **Corner protection:** This type of armor is used to protect the corners of the concrete monoliths at discontinuities in the lock wall, such as at a floating mooring bitt or a line hook. Corner protection is also located along the top of the monolith.

A total of six navigation projects were inspected on the Ohio River system. The projects in order of inspection were (a) Smithland, (b) John T. Myers, (c) Newburgh, (d) Cannelton, (e) McAlpine, and (f) Markland.

The inspection at each lock was performed by GSL personnel, with support from CELRL engineers and lock operations and maintenance personnel. A maintenance craft was used to visually inspect the armor system in the lock at both lower and upper pools. Additionally, the upstream and downstream approach walls and bullnoses were inspected. Photographs were taken at locations of armor damage. The location of broken armor, depth of section loss in the surrounding concrete, and other pertinent observations were recorded.

Smithland Locks and Dam was the first project to be inspected. Smithland is the newest of the structures that were inspected, having opened for river traffic in 1979. Overall, the Smithland locks exhibited the least amount of armor damage of any of the locks inspected. The relatively low level of armor damage might be expected, as these locks were in service for the shortest time. Also, Smithland has two 1,200-ft lock chambers in use, while the other projects have one 600-ft chamber and one 1,200-ft chamber. Since commercial traffic usually requires locking in a 1,200-ft chamber, having two such chambers greatly decreases the relative amount of traffic in each.

The locks at Smithland had some locations where the concrete was gouged out around the armor, most likely due to barge impact. This damage was most prevalent at the upstream and downstream bullnoses. The maximum recorded depth of concrete section loss at Smithland was 3.5 in. Additionally, one instance of broken corner armor was recorded.

John T. Myers Locks and Dam was the second project inspected. This facility was completed in 1975, although the locks were operational a few years earlier. The locks at John T. Myers pass the most traffic of all the projects that were inspected. Consequently, the armor system exhibited the most damage of all the projects. The majority of the damage observed included gouges and spalls in the concrete adjacent to an armor strip. Many of the recorded gouges were located at or adjacent to a vertical joint. The maximum recorded depth of concrete section loss at the John T. Myers site was 5 in. In addition to the spalls and gouges, broken armor was observed at 10 locations.

Newburgh Locks and Dam facility was completed in 1975. The armor system at this project exhibited a moderate amount of deterioration in comparison to the other facilities. As at John T. Myers, the majority of damage at Newburgh consisted of gouges or spalls of concrete adjacent to the armor itself. The maximum recorded depth of concrete section loss was 3 in. In addition to loss of section in the concrete, broken armor was observed at seven locations.

Cannelton Locks and Dam was the next site to be inspected. This project was completed in 1974, but the locks were first opened to traffic in 1965. The inspection revealed that, overall, there were fewer gouges and spalls than were typical at the other sites. However, the maximum recorded depth of concrete section loss was 5.25 in., the largest of any of the sites. Broken armor was recorded at four locations. Obvious physical degradation of the armor itself occurred in one location. This type of damage might be expected from long-term exposure to an aggressively corrosive environment. However, even for this case, it is likely that the observed corrosion was precipitated by impact damage.

McAlpine Locks and Dam was completed in 1964. Only the 1,200-ft lock chamber was inspected at this site, as the 600-ft chamber was currently in the process of being replaced. Broken armor was observed at five locations. At the downstream bullnose on the middle approach wall, McAlpine exhibited the most severe armor damage of any of the projects. According to lock personnel, this damage was not the result of a single event, but of several incidents over a period of years. This type of continuing localized damage underscored the need to develop a program responsible not only for repairing but also for maintaining and monitoring the armor system.

Markland Locks and Dam was also completed in 1964, although the locks were opened to traffic in 1959. This is the oldest of the projects inspected, and as such it exhibited the most concrete damage, with concrete damage ranging from gouges to voids behind the armor. The maximum recorded depth of concrete section loss was 4.5 in. Broken armor was observed at nine locations, most of which was very similar to that observed at the other projects (i.e., horizontal armor broken at the ends). Damage to the corner protection running along the top of the lock chamber was seen only at Markland.

Summary of field inspection of selected Ohio River armor systems

- A total of 36 instances of broken armor were observed at the six projects. The most armor damage was observed at John T. Myers, which passes the most annual tonnage of the six facilities.
- A variety of armor-related concrete damage was observed at each project except Smithland, which was in service the shortest time (approximately 20 years). The maximum depth of concrete section loss was 5.25 in. at Cannelton.
- Six of the 36 (17 percent) areas of broken armor were located within the lock chambers.
- Seventeen of the 36 (47 percent) areas of broken armor were located on the downstream approach walls.
- Thirteen of the 36 (36 percent) areas of broken armor were located on the upstream approach walls.

Causes of armor damage

The inspection results revealed that the vast majority of armor damage occurred in the 1,200-ft lock chambers as opposed to the 600-ft chambers. These results indicate that the armor damage observed was the result of a combination of impact and abrasion by commercial barge traffic that typically used the 1,200-ft chambers. Problems relating wholly to corrosion of the steel were virtually nonexistent, even for armor with 40 years of service. Prior to entering the lock chamber, a tow operator aligns a barge using the guide walls. When moving through the chamber, the barge typically bears to one side, making the barge vulnerable to "catching" protruding pieces of metal. This is especially hazardous for the armor from barges that have protection for themselves.

Typical armor damage

Loss of concrete section: The most commonly observed armor-related damage is the loss of concrete. This type of damage has been discussed previously in the present statement of the problem at John T. Myers Locks and Dam.

Broken armor: Locations where the steel armor system is broken are of special concern because of the potential for serious damage to passing vessels catching on ragged edges. In addition, broken armor is more susceptible to corrosion. The majority of damaged armor observed during

the inspections involved steel strips that were broken at the ends. At the McAlpine and Markland sites, this type of damage was mitigated by the original armor design that did not allow an armor strip to end in a monolith joint. Instead, small pieces of armor were installed on both sides of the joint. The condition of the armor installed in this manner was much better than that of armor that was truncated at a monolith joint. At Cannelton, steel angles were used to protect vertical monolith joints. This design also served to protect the horizontal rubbing armor that runs parallel to the lock wall. In addition to broken rubbing armor, several locations of broken corner protection were noted during inspection.

Other concerns: Apart from concrete damage and broken armor, another major concern is abrasion on armor strips. This type of damage has been discussed previously in the present statement of the problem at John T. Myers Locks and Dam.

Conclusions

In general, the wall armor system installed at the selected navigation projects on the Ohio River performed well in service. The concrete lock walls at each of the projects are generally in sound condition and capable of continuing in service for the foreseeable future. However, localized areas of armor damage at each project are cause for concern.

The armor damage that was observed at the navigation projects is the result of impact and abrasion from commercial traffic passing through the locks. Corrosion of the steel armor due to environmental exposure is not a major concern, as undamaged armor with 40 years of service appears in good shape. Most of the armor damage observed can be classified as a loss of concrete section around the steel. This surrounding concrete supports the steel armor and, when the concrete is removed, the steel itself is much more susceptible to damage. When the steel is damaged, the lock walls themselves become vulnerable to deterioration. Additionally, broken or protruding steel armor could damage commercial traffic passing through the lock. In a worst case scenario, a damaged vessel could sink in the lock chamber, halting traffic for a significant period of time. Given the large volume of traffic passing through these locks each year, such a shutdown would have dire economic consequences.

The results of this inspection demonstrate the need for a comprehensive program of maintenance and repair for wall armor on heavily traveled

locks. More frequent damage to the armor system can be expected as the projects continue to age and traffic on the river increases. Methodologies must be developed for the expedient repair and replacement of broken steel and concrete. In addition, a routine maintenance program should be developed that will mitigate armor-related damage before it becomes more pronounced. Given the economic impact of an unexpected lock shutdown, the benefits of a continuing program of maintenance and repair would far exceed the investment of resources for such a program.

Monitoring plan

Detailed expedient repair demonstration procedures were developed by the Louisville District, ERDC Geotechnical and Structures Laboratory (GSL), and McDonald (2006a, 2008a) to be monitored by MCNP. Repair impacts to the Ohio River barge navigation traffic would have to be limited. Several considerations had to be incorporated. All expedient repair methodologies were considered because of the major economic impact by any interruption of normal river operations. Repair techniques were focused on restoring the system to its original capability. Performance criteria for repair materials were developed that were appropriate for the specific types of repair to be demonstrated and monitored. Various traditional rapid-hardening cementitious materials were considered. Prefabricated stay-in-place steel forms were evaluated. Additionally, non-traditional materials, such as fiber reinforced polymer (FRP) composites, were researched for future consideration for repair and rehabilitation of concrete structures, such as inland hydraulic navigation locks and dams.

This MCNP monitoring study of expedient lock wall repair demonstrations at John T. Myers Locks and Dam consisted of five aspects.

Evaluation of John T. Myers wall armor system for monitoring expedient repair demonstrations: The majority of the observed damage was located where straight-run wall armor was terminated (McDonald 2006a). This was particularly true where the armor terminated near vertical monolith joints. Two such joints existed on the land-wall side of the 600-ft lock, one at the upper approach and a second at the lower approach. Thus, the opportunity existed to demonstrate two fundamentally different repair techniques at these two locations with essentially no disruption to river traffic. The first technique would attach two concrete monoliths together at the vertical joint so as to create a single unit. The second technique would not attach two concrete monoliths together at the

vertical joint but allow them to remain as two separate units. These two distinctly different techniques would allow direct comparison of (a) ease of application and (b) durability and effectiveness. In both cases, the steel plates would act as permanent forms and be backfilled with a rapid-hardening, high-early-strength, low-shrinkage concrete.

Monitoring repair of 600-ft chamber upper land-wall approach vertical joint: This first vertical joint repair technique would fit 0.5-in.-thick steel plates over the vertical joint separating monoliths L-35 and L-36. The plates would measure 36 in. in width over the upstream monolith and 48 in. in width over the downstream monolith. Both sides would be 20 ft (± 6 in.) high. The plates would be welded together at all joints, and 60 concrete anchor bolts would be spot-welded to the plates. This technique would attach the two concrete monoliths together as one unit.

Monitoring repair of 600-ft chamber lower land-wall approach vertical joint: This second vertical joint repair technique would require 12 16-in.-high by 12-in.-wide by 0.75-in.-thick steel plates for monolith L-1, and 17 16-in.-high by 12-in.-wide by 0.75-in.-thick steel plates for monolith L-2. The wall armor plate strips would have a recess cut on the top and bottom to accommodate the steel plates that would then be welded together horizontally at each joint for each independent monolith. Each steel plate would be attached to the wall by spot-welding to four concrete anchor bolts. The vertical steel joints between monoliths L-1 and L-2 would not be welded together. Thus, the two concrete monoliths would not be attached but would remain two separate units.

Monitoring repair of 1,200-ft chamber upper-river approach wall: The area of concrete damage selected by the Louisville District for repair demonstration monitoring was located in monolith R-73. The surface area was 1.75-ft by 6-ft, with an apparent depth of almost 2 ft in some places. McDonald (2008a) provided technical assistance by determining whether available rapid-setting materials could be modified if necessary to extend working times without compromising high-early strength. The form-and-pour technique would be utilized after the cavity was prepared by removing all existing low-strength concrete material down to clean, sound, and suitably roughened surfaces. Reinforcing steel bars would be installed in drilled holes as anchors for the concrete.

Feasibility of fiber reinforced polymer (FRP) composites for inland hydraulic structure application: It is prudent to remain abreast of the latest developments in materials, such as advanced composites, that might be applicable to repair and rehabilitate marine structures. West Virginia University was commissioned to conduct a feasibility review of FRP (GangaRao and ViJay 2010). That study dealt with the design, development, and implementation of FRP composite structural systems that are of interest to USACE, focusing on civil and marine applications. Constituents, short- and long-term properties, and influences of fiber orientation on strength, stiffness, and deformation of composite products were described under combined external and environmental loads. That study focused primarily on glass polymer composites (fiberglass).

2 Evaluation of John T. Myers Wall Armor System for Monitoring Expedient Repair Demonstrations[1]

General classifications for the types of armor damage at John T. Myers Locks and Dam were developed, and potential concepts for their repair were proposed. This information served as a basis for ERDC and CELRL discussions to select a concept(s) for development of details for innovative and expedient demonstration repairs with minimal disruption to navigation traffic on the Ohio River.

John T. Myers Locks and Dam was constructed with two primary types of armor that are classified according to orientation. Horizontal armor includes straight-run wall armor and curved corner armor at the top corners of wall monoliths. Vertical armor includes corner protection in the vicinity of floating mooring bitts, ladders, line hooks, etc. Both types of armor contain anchors that were embedded in the concrete during construction. A typical anchor detail for embedment of straight-run wall armor is shown in Figure 5. Ends of the straight-run wall armor were tapered where it terminated or intersected with vertical armor (Figures 6 and 7).

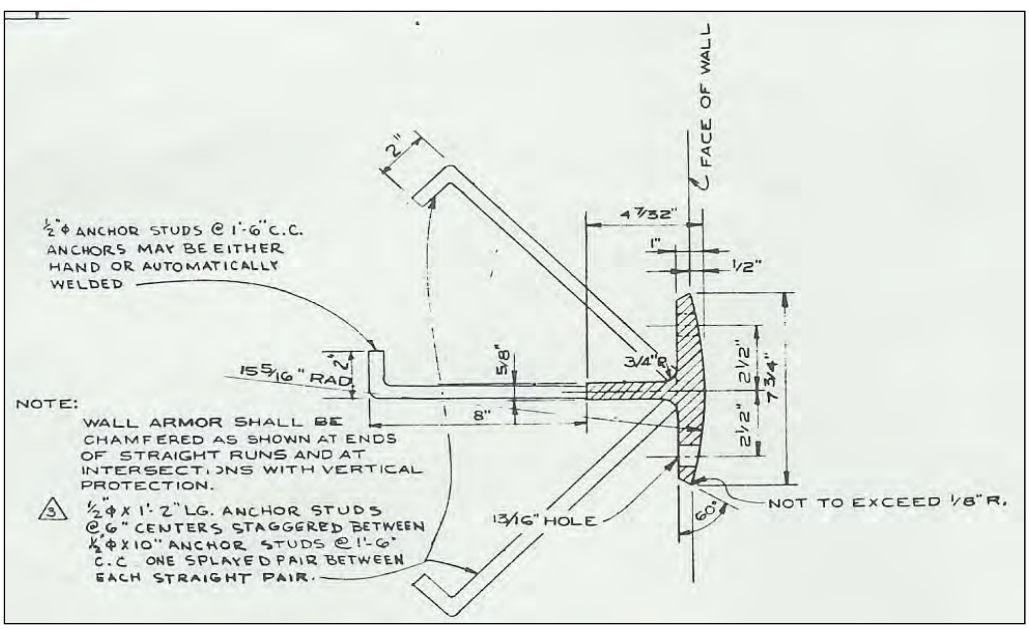


Figure 5. Typical anchor embedment, straight-run wall armor.

[1] Chapter 2 is extracted essentially verbatim from McDonald (2006a).

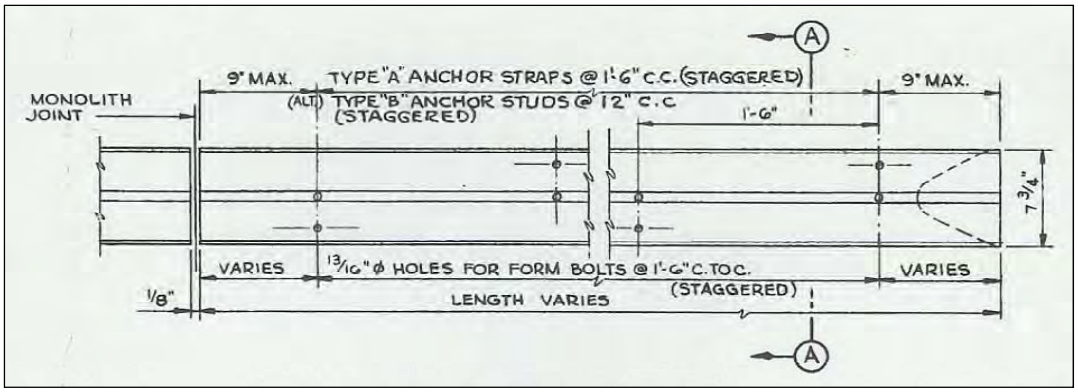


Figure 6. Straight-run wall armor, elevation.

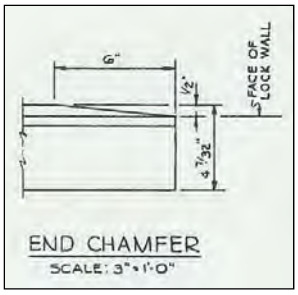


Figure 7. Straight-run armor, Section A-A.

Results of previous visual inspections were examined in detail to identify the most prevalent types of degradation and determine their possible causes. The various types of most prevalent damage at John T. Myers are generally classified according to five locations.

- Type I – Vertical monolith joints, single-side armor
- Type II – Intersection of wall armor and vertical corner armor
- Type III – Intersection of bullnose and lock chamber walls
- Type IV – Wall armor termination within concrete monolith
- Type V – Bullnose

Potential repair strategies were developed based on a variety of repairs previously used by Corps Districts that appear to have potential for application at John T. Myers. Any of the concepts described can be modified as appropriate for current applications.

Type I – Vertical monolith joints, single-side armor

Type I damage

The majority of the observed damage was located where straight-run wall armor was terminated. This was particularly true where the armor was

terminated near a monolith joint. Examples of this type of damage are shown in Figures 8 - 10. The relatively small concrete section between the end of the armor and the monolith joint is inadequate to resist the impact loads transmitted through the armor. Apparent bending of the armor beyond the last anchor also appears to contribute to localized crushing and subsequent loss of concrete section (Figure 9). Significant loss of concrete (Figure 10) makes these areas more susceptible to damage during chamber filling and emptying, particularly where barges have protruding steel plates such as shown in Figure 11.

Figure 8. Overall view of Type I damage, John T. Myers.

Figure 9. Type I armor fractures and concrete spalling, John T. Myers.

Figure 10. Type I loss of concrete section, John T. Myers.

Figure 11. Example of barge protrusions.

Type I repair

Historically, monolith joints in lock chambers were susceptible to damage regardless of the construction or repair materials used. Consequently, a recent trend was to revise joint configurations to minimize the potential for impact loads on the corners of such monoliths. For example, the precast concrete panel stay-in-place forming system initially used to resurface the chamber walls at Mississippi River Lock 22 began to exhibit cracking and spalling at the chamfered joints shortly after being placed into service. Saw cuts at a 5-deg angle from 12 in. on either side of a joint were supposed to provide a recess 1 in. deep at the joint.

The resulting joint appears to have a deeper recess, preventing barges from impacting the joints directly and minimizing impact damage. The condition of typical joints after 7 years in service is shown in Figure 12. In subsequent projects, joint spalling was essentially eliminated (Figure 13) by incorporating a recessed joint into the panels during precasting.

The idea for recessing joints to minimize damage apparently originated in the mid-1980s in a St. Paul District project to repair monolith joints in Upper Mississippi River locks damaged by freezing and thawing, combined with impact and abrasion loads. Four joint repair procedures were developed by Harza Engineering under contract with St. Paul District.

- Alternative 1 - Remove unsound and deteriorated concrete, sand blast or high pressure water blast clean, and patch the joint with polymer cementitious mortar. Do not saw cut.

Figure 12. Recessed joint at Mississippi RiverLock 22 after 7 years of service.

Figure 13. Precast panel joint at Allegheny Lock after four years of service.

- Alternative 2 - Saw cut a minimum of 4 in. on both sides of the joint to a depth of 1.5 in. from top of wall to 2 ft below low water level. Remove concrete within the saw cuts to a minimum depth of 2 in. or to sound concrete. Install bolt-and-nut anchors on 6-in. centers and replace concrete along the joint with glass fiber-reinforced, polymer mortar or other impact resistant material. Recess the joint inward 0.5 in. Maintain a joint width of 0.5 in. to 0.75 in. at the face (Figure 14). Problems with formwork materials and the solution ultimately adopted by St. Paul District are described by Dahlquist (1987).

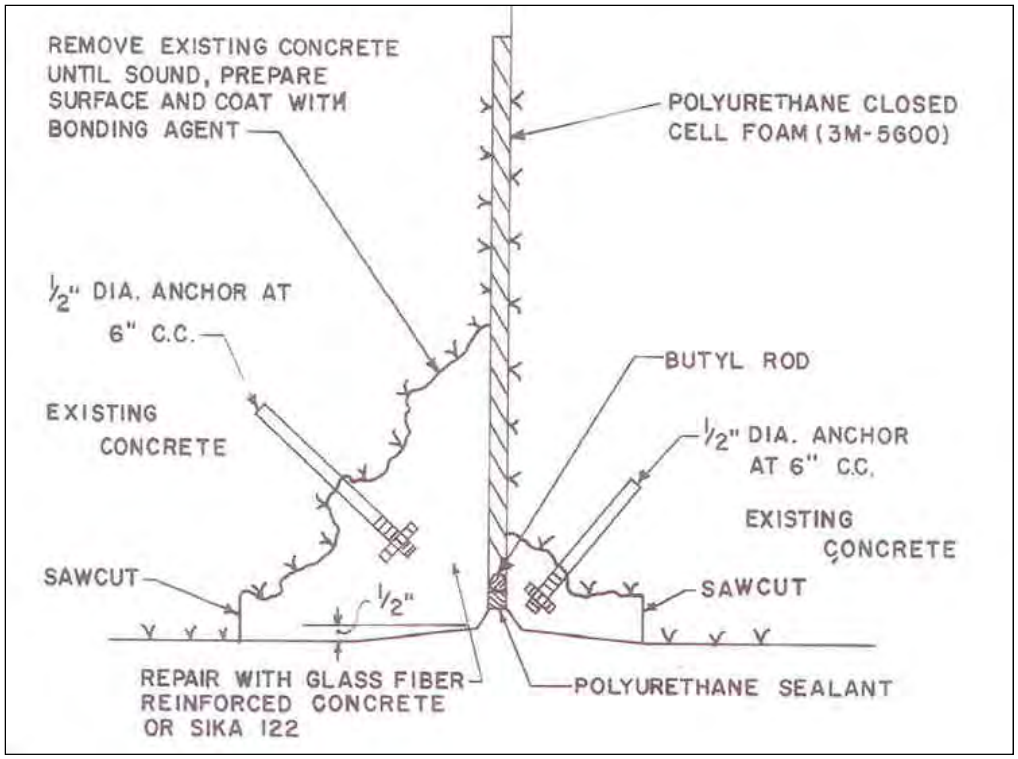


Figure 14. St. Paul District, Alternative 2.

- Alternative 3 - Saw cut a minimum of 8 in. to 10 in. on both sides of the joint to a depth of 1.5 in. from top of wall to 2 ft below low water level. Remove concrete within the saw cuts to a minimum depth of 8 in. or to sound concrete. Install anchors and vertical wall armor on both sides of the joint and grout the void with non-shrink grout (Figure 15).
- Alternative 4 - Perform joint repairs as described in Alternate 3 and install horizontal wall armor for a distance of 5 ft on each side of the joint (Figures 16a and 16b).

Three principal criteria were used to evaluate the alternatives; (a) freeze-thaw resistance, (b) impact resistance, and (c) economy. Freeze-thaw resistance can be obtained for a variety of cementitious materials provided they have a properly entrained air-void system. The angle of impact appears to be the most important consideration in impact resistance. Therefore, it was concluded that the monolith joints should be recessed approximately 0.5 in. away from the plane of the chamber wall. Alternative 1 requires the least amount of work and is the least expensive of the alternatives. In comparison, estimated costs for Alternatives 2, 3, and 4 were 1.2, 1.7, and 4.0 times more expensive, respectively. As a result of the evaluation, Alternative 2 was selected, and the first application was at Lock 2, Mississippi River, near Hastings, MN. Fiber-reinforced, acrylic polymer-

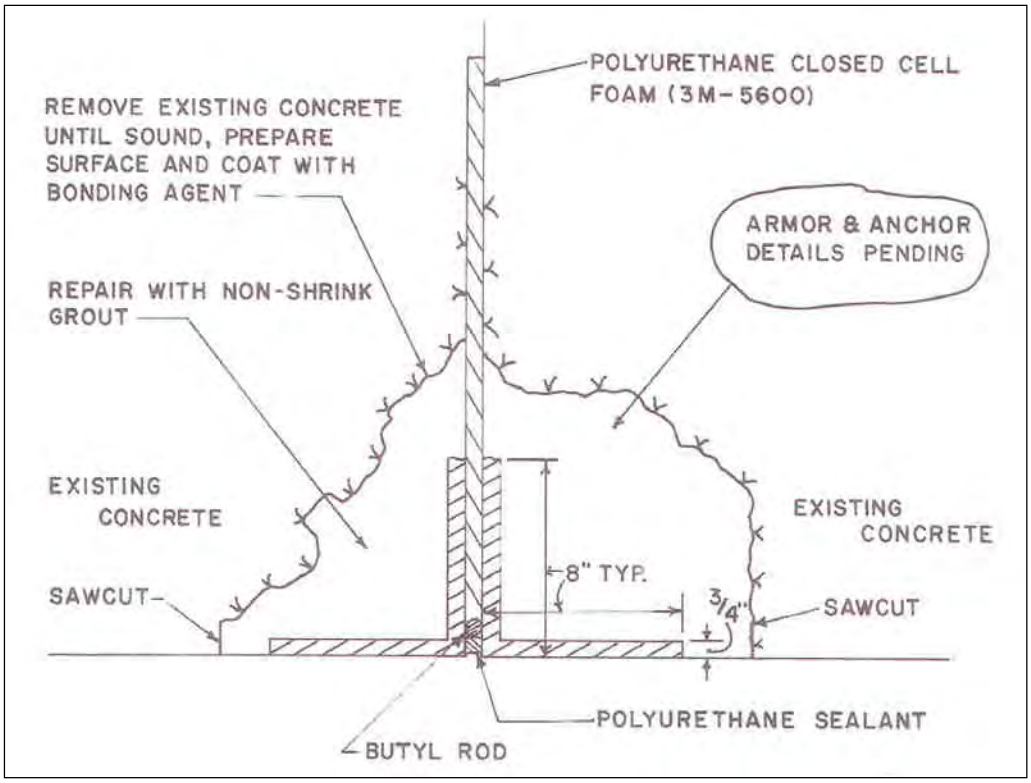


Figure 15. St. Paul District, Alternative 3.

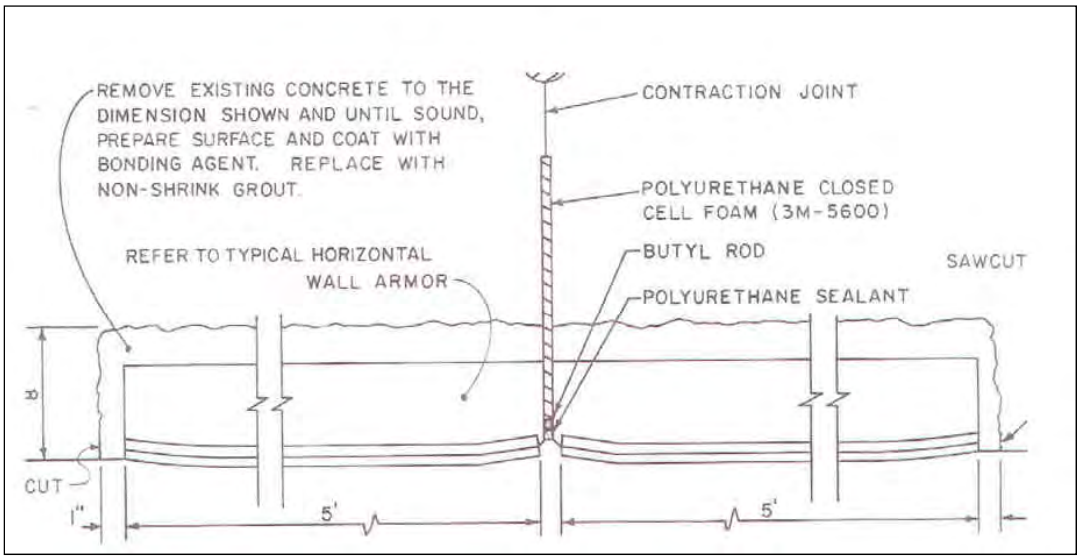


Figure 16(a). St. Paul District, Alternative 4. Plan view (continued).

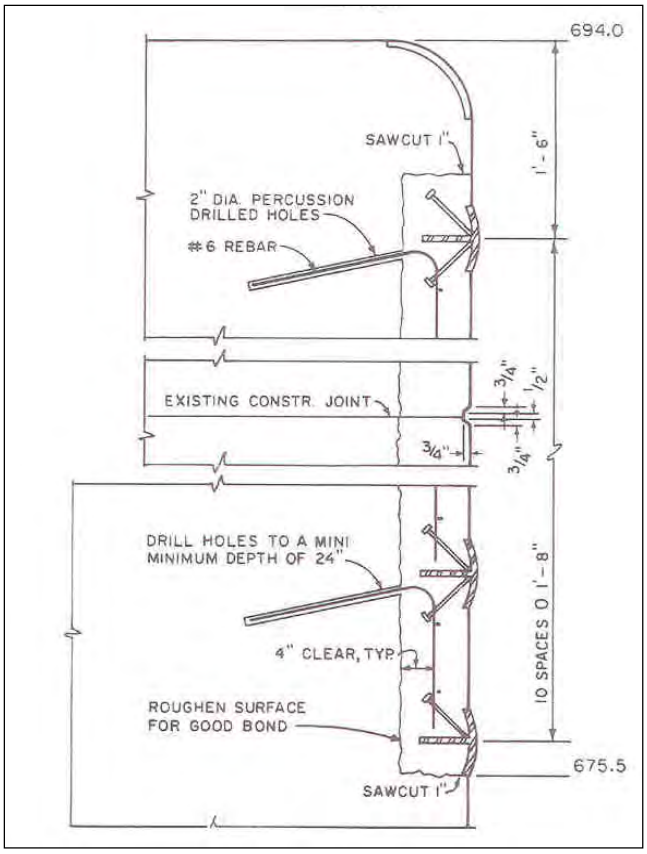


Figure 16(b). St. Paul District, Alternative 4, Elevation (concluded).

modified concrete (FRAPMC) was used to repair 28 monolith joints and selected sections of the lock wall during the winter of 1986-87 (Dahlquist 1987).

A typical joint after 3 years in service is shown in Figure 17. As a result of the continued good performance, this procedure was used in several similar repairs of lock monolith joints on the upper Mississippi River. Current periodic inspection reports indicate that these repairs continue to exhibit good performance.

Alternative 3 was not selected because it did not provide a recessed joint to minimize direct impact on monolith corners; however, a similar detail (Figure 18) was used in construction of Cannelton and McAlpine Locks. Apparently, this design provided satisfactory monolith joint protection (Figure 19). Also, a similar detail was developed by New Orleans District for repair of monolith joints (Figure 20) at Bayou Sorrel Lock (May and McDonald 1989). Also, anchoring of the replacement concrete to the existing concrete should be considered.

Figure 17. Condition of repair after 3 years of service, Lock 2, Upper Mississippi River.

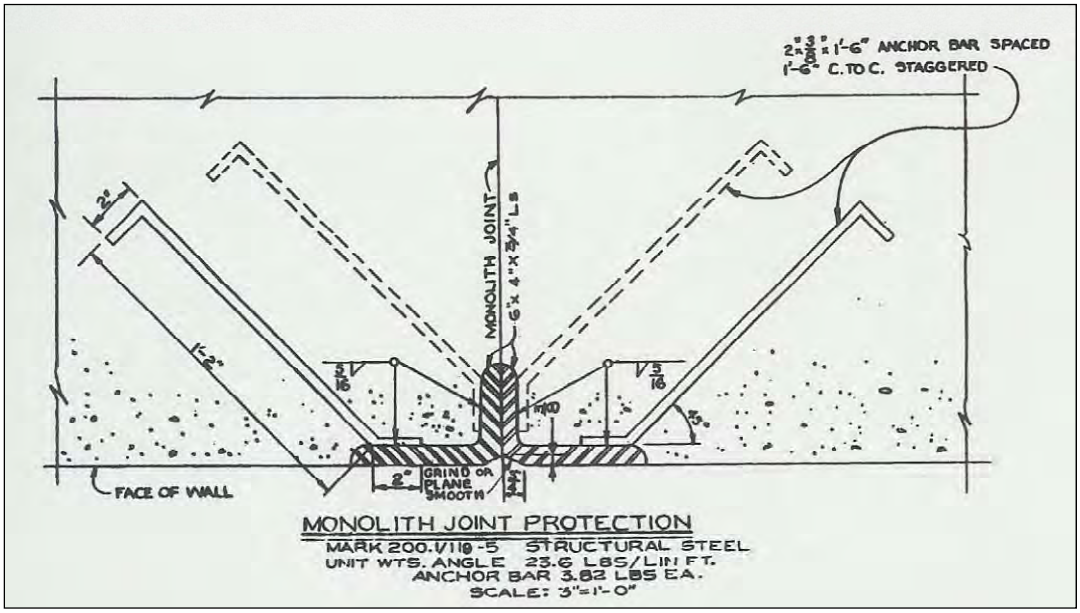


Figure 18. McAlpine Lock detail.

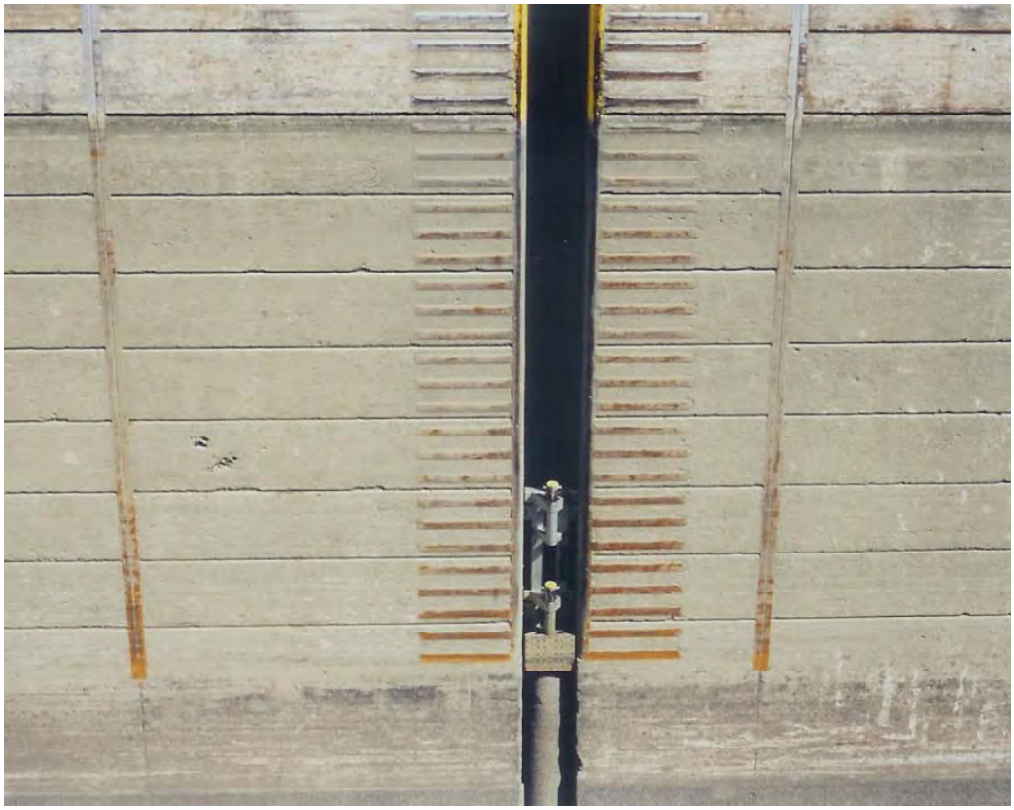


Figure 19. Armor condition, McAlpine Lock, 1999.

The use of an epoxy bonding agent typically results in increased slant-shear bond strengths of less than 15 percent compared to a sound, properly prepared concrete surface without a bonding agent. Therefore, given the increased labor and material costs, plus the potential for actually creating a bond breaker when repair application is delayed, current industry practice is to use no adhesive bonding agents for cementitious repairs that are more than 3 in. deep. Implementation of this detail in a repair would involve removal of significant volumes of concrete, unless an alternative anchor system was developed. Both anchor strap and headed stud wall armor assemblies that would reduce the concrete removal required were previously evaluated by ERDC (McDonald 1988). The potential for drilling and grouting of anchors in lieu of concrete removal should also be evaluated.

Curved armor plate was used to provide a recessed corner protection for selected monolith joints at Algiers Lock (Figure 21). Also, an ultra-high molecular-weight polyethylene liner was used in lieu of steel to armor other joints at Algiers Lock (Figure 22). The liner was fixed on one side and free on the other side to accommodate joint expansion and contraction (May and McDonald 1989).

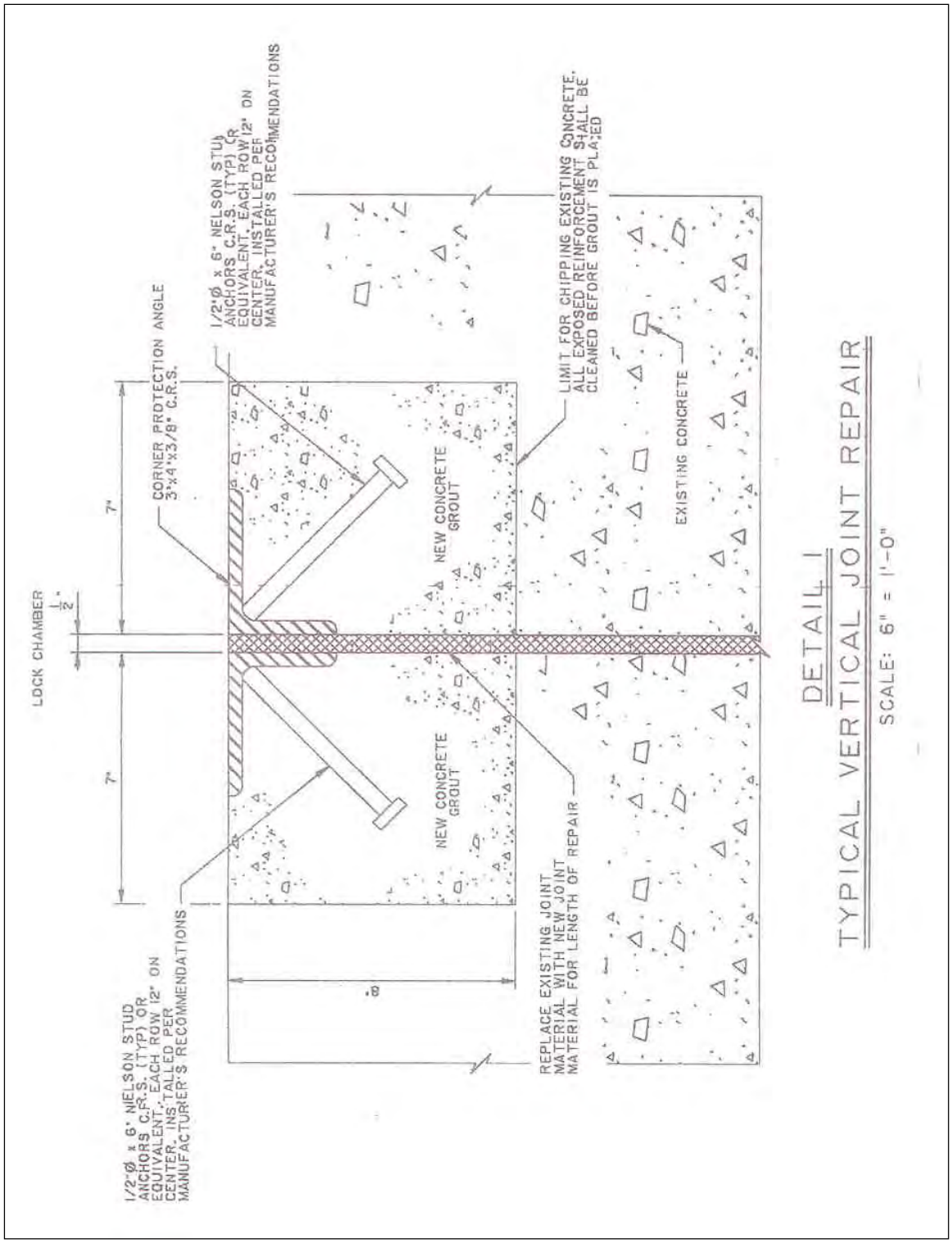


Figure 20. Bayou Sorrel detail.

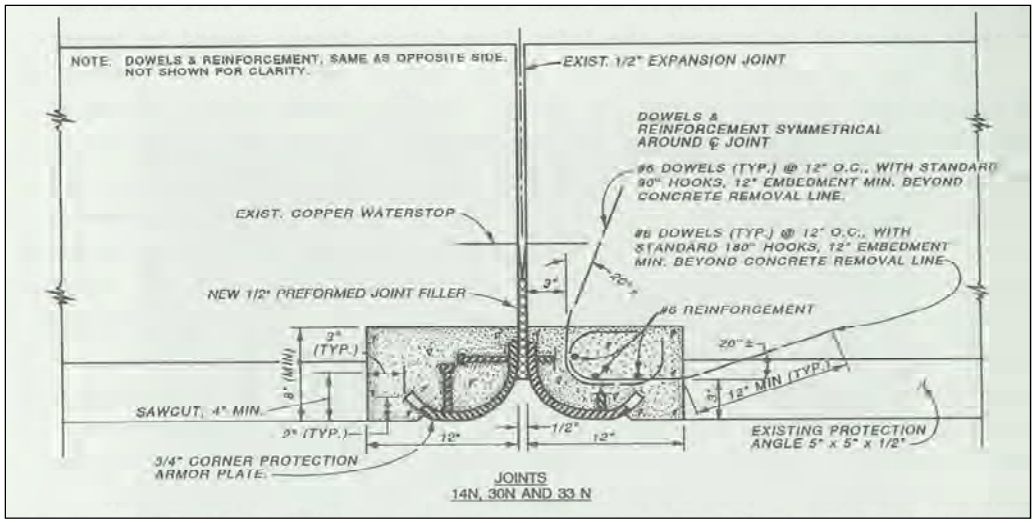


Figure 21. Algiers Lock, curved armor plate.

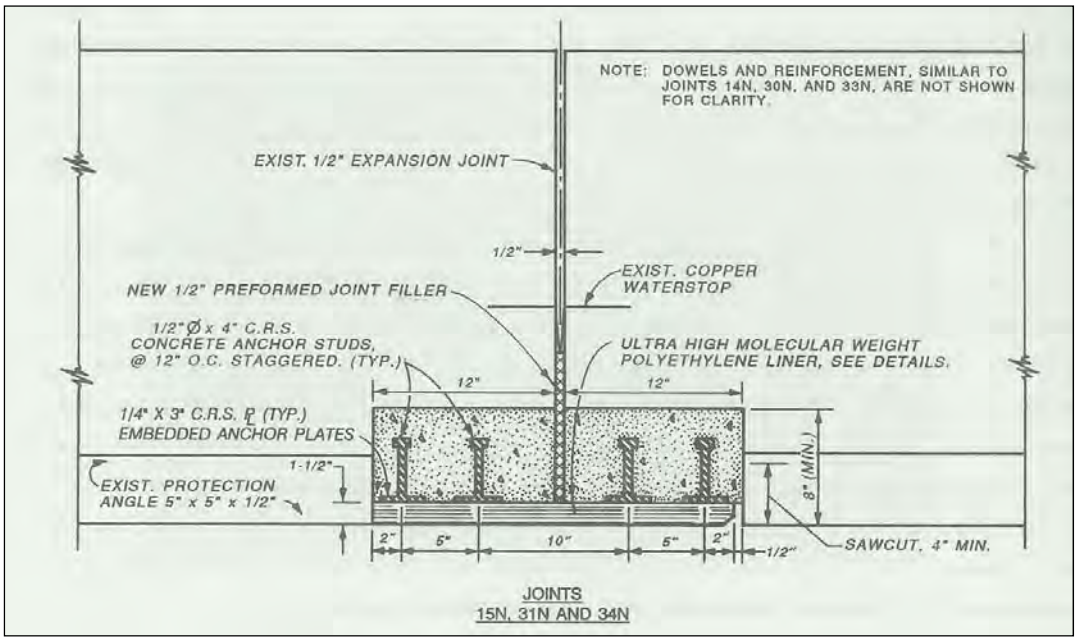


Figure 22. Algiers Lock, polyethylene liner instead of armor plate.

Type II – Intersections of wall armor and vertical corner armor

Type II damage

Ends of the straight-run wall armor were tapered, as shown in Figures 6 and 7, where this armor intersects vertical corner protection armor for floating mooring bits, ladders, etc. As a result of this taper, the vertical corner protection is slightly recessed (about 0.5 in.) behind the plane of the lock wall. In general, this recessed location protected the corner armor from major damage despite localized crushing of the concrete attributed to

impact and abrasion (Figure 23). However, this protection was compromised in some cases because the wall armor was flattened by barge abrasion (Figures 24 and 25). Continued loss of wall armor section combined with loss of concrete section (Figure 26) will exacerbate this type of damage with potentially serious consequences for lock operations.

Figure 23. Typical Type II damage, John T. Myers, localized crushing of concrete.

Figure 24. Typical Type II damage, John T. Myers, impact abrasion.

Figure 25. Typical Type II damage, John T. Myers, barge abrasion.

Figure 26. Typical Type II damage, John T. Myers, loss of concrete section.

Type II repair

Restoration of protection for the vertical corner armor should be a primary objective for repairs of this type. Potential methods for repair include replacement of lost section(s) on existing armor or installation of relatively short runs of new armor between the existing wall armor sections. The feasibility of welding a new crown section to existing armor should be evaluated. Fillet welds around the perimeter of additional steel plate and subsequent grinding to smooth transitions appear to have potential. Since removal and replacement of damaged concrete will be necessary in a number of locations, the feasibility of adding new armor in these locations should be evaluated. Limited nondestructive testing or coring may be necessary to determine the extent of concrete damage for proper evaluation of this approach to repair. As previously discussed for Type I repairs, anchor strap or headed-stud wall armor assemblies that would reduce the need for removal of sound concrete and the potential for drilling and grouting of anchors should be evaluated.

Type III – Intersection of bullnose and lock chamber walls

Type III damage

Areas where adjacent monoliths intersect at an angle exhibit damage both in and above armored zones. The most severe damage, in terms of lost section (area and volume), is located outside of the armored areas, and this damage appears to have increased since 1999 (Figures 27a and 27b). Impact appears to be the primary cause of this damage. There are areas of damage within the armored zone (Figure 28); however, the number of locations appears limited relative to the other types of damage.

Type III repair

The wall armor system in this area performs reasonably well; therefore, any necessary repairs of localized damage should focus on restoring the joint to its original condition. Complete confinement of the concrete within the armor appears to provide good resistance to impact and abrasion (Figure 29).

Figure 27(a). Example of Type III damage, John T. Myers, 1999 (continued).

Figure 27(b). Example of Type III damage, John T. Myers, 1999 (concluded).

Figure 28. Typical Type III damage, John T. Myers.

Figure 29. Steel encapsulated concrete, John T. Myers.

Type IV – Wall armor termination within concrete monolith

Type IV damage

Damage where wall armor terminates within a concrete monolith is generally localized near the end of individual armor sections (Figure 30). Concrete damage appears to be the result of impact forces transmitted through the armor and possible bending of the armor at the end and along the flanges near the end. Loss of concrete section is generally small with the larger spalls in unconfined concrete where armor parallels a lift joint (Figure 31).

Type IV repair

The wall armor system in this area performs well; therefore, repair of the localized damage does not appear necessary at this time. These areas should be monitored periodically because continuing loss of section caused by abrasion will increase armor flexibility that can, in turn, result in increased concrete damage and make the armor itself more susceptible to damage.

Figure 30. Typical Type IV damage, John T. Myers, end of armor section.

Figure 31. Typical Type IV damage, John T. Myers, armor near lift joint.

Type V – Bullnose

Type V damage

The bullnose armor systems generally perform satisfactorily despite the adverse impact and abrasion exposure conditions. Loss of concrete between armor sections is generally of uniform depth and appears to be approaching the maximum abrasion expected, given the spacing of the armor and configuration of typical barge traffic (Figure 32). The armor appears to be in generally good condition.

Type V repair

During the 1999 survey, CELRL personnel stated that the consequences of a catastrophic event such as barge sinking were not as dire in the area of a bullnose as they would be inside the lock chamber; therefore, problems within the lock chamber should be addressed first. Therefore, the need for repair of bullnose damage at this time is questioned. These areas should be monitored periodically to determine the rate of concrete abrasion because continuing loss of section could ultimately affect the armor anchor system. Also, continued abrasion could undermine the edges of the wall armor,

Figure 32. Typical Type V damage, John T. Myers, bullnose.

making it more susceptible to damage. Complete confinement of the concrete within the armor appears to be appropriate for increased resistance to impact and abrasion.

Synopsis

General classifications of observed damage were described and, where repairs are considered necessary or desirable, potential repair strategies were proposed. Detailed repair procedures were developed and prioritized as a joint effort between ERDC GSL and CELRL. Factors that were considered in this process are discussed below.

- Impact on the users had to be limited. The lock operates 24 hr a day, 365 days a year with no seasonal shutdown. Repairs had to be accomplished without shutdown of the lock if at all possible. Repairs within operating lock chambers will obviously have many safety implications. Scheduled shutdowns occur at 5-year and 7-year intervals, and some repairs, particularly those below low-pool elevation, could possibly be accomplished at these times.

- The John T. Myers project has a 600-ft and a 1,200-ft lock chamber. The smaller auxiliary chamber was used to demonstrate and optimize repair techniques to minimize impact on the main chamber. To the extent possible, Corps personnel were used in the development and application of repair techniques to minimize the need for outside contractors.
- Economy is obviously a primary interest; however, all expedient repair methodologies were considered because of the major economic impact of any interruption of normal operations. Any potential for the use of advanced materials, techniques, and equipment was considered.
- Most of the wall armor systems perform reasonably well; consequently, repair techniques were focused on restoring the system to its original capability. However, enhancements were considered in addition to repairs. All repair methodologies were developed to prevent (delay) an extensive rehabilitation that would require closure of the lock system.
- Either conventional concrete or polymer-cement concrete was used as a replacement material in most, if not all, previous repairs. A primary advantage of polymer-cement materials is the enhanced bond between repair and existing substrate concrete. Performance criteria for selection/specification of repair materials were developed that are appropriate for specific type(s) of repair to be demonstrated in the given application and service conditions.
- Most previous repairs were accomplished during lock closures; therefore, proposed repairs in an operational lock present unique requirements. While there are a variety of rapid-hardening cementitious materials that may be appropriate for this type of repair, prefabricated stay-in-place steel forms have merit and were evaluated. Forms were prefabricated to provide a recessed joint, and predrilling holes in the forms provided a template for drilling and installing concrete anchors that were welded to the forms. Also, the potential for minimizing concrete removal and using grout injection to consolidate fractured concrete and fill voids behind the forms was evaluated.
- Concrete replacement materials should be dimensionally compatible with the remaining concrete substrate; therefore, they should comply with the repair material performance requirements set forth by Section 1.4.2 of the Unified Facilities Guide Specification (2004). Use of rapid-hardening repair materials with compressive strengths in accordance with ASTM (2000) C928 was considered to minimize downtime. Also, fiber reinforcement was included to improve toughness and impact resistance.

3 Monitoring Repair of 600-ft Chamber Upper Land-Wall Approach Vertical Joint

Because of the high river traffic volume on the Ohio River, the vertical joint where the land-side upper approach wall intersects the lock chamber wall experiences an exceedingly large amount of impact as barges heading in a downstream direction line up for entrance into the chamber. Barge impacts since the locks were opened in 1969 have been frequent and oftentimes quite severe. According to the Corps' Waterborne Commerce Statistics for 2008, over 71.9 million tons of commodities were shipped through the locks, with a combined value of $13.4 billion. The leading commodity shipped through the locks was coal, comprising 51 percent of the tonnage.

This specific vertical joint between monoliths L-35 and L-36 (Figure 33) was selected by CELRL in coordination with ERDC GSL as significantly important to merit a demonstration repair to be monitored and documented by the MCNP program as an expedient method for lock wall rehabilitation (McDonald 2006b). To minimize impact on river navigation through the main chamber, the 600-ft chamber was selected for a demonstration repair to ascertain the optimum time interval among repair intervals and downtime to be expected by the high-volume 1,200-ft operational lock chamber. Most previous repairs were accomplished during scheduled lock closures that occur at about 5-year to 7-year intervals. The capability to make repairs outside those windows without waiting for scheduled downtime will significantly enhance shipment of commodities through the locks.

Alternative repair techniques

Two alternative repair techniques were proposed by the team of CELRL and ERDC GSL personnel. Alternative 1 (Figure 34) involved replacing damaged concrete with new concrete by using a removable custom form, holding the new concrete in place with anchor bolts, and leaving the new concrete exposed. Alternative 2 (Figure 35) would install steel plates over the new concrete, using the steel plates as stay-in-place forms held by anchor bolts.

Figure 33. 600-ft chamber upper land-wall vertical joint selected for repair monitoring.

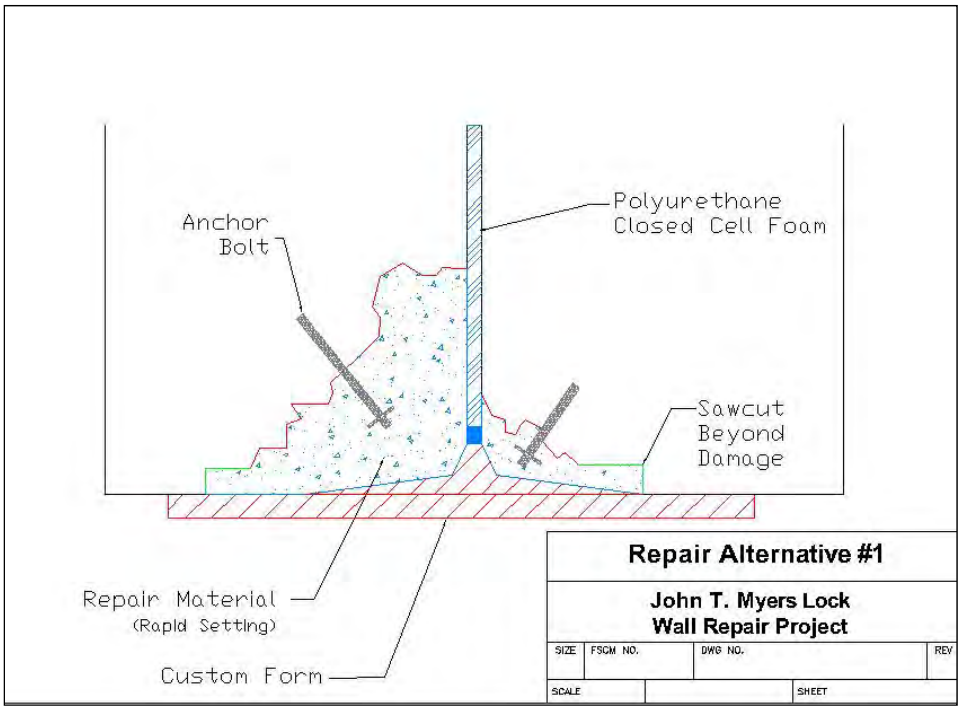


Figure 34. Alternative 1, removable custom form.

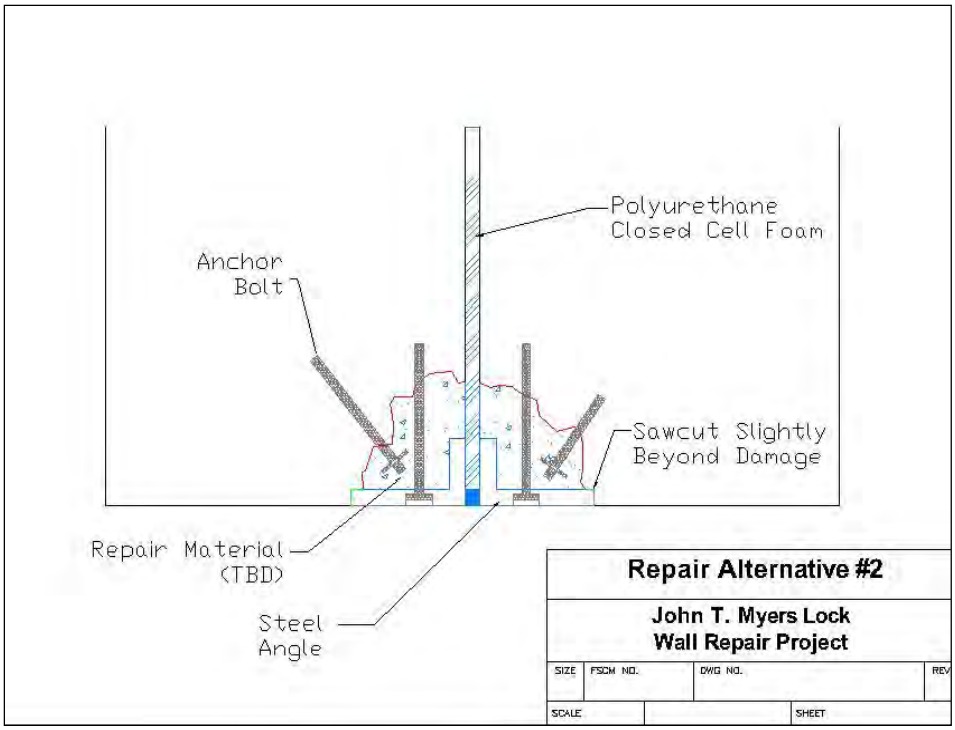


Figure 35. Alternative 2, stay-in-place steel plate armor form.

Alternative 1 specifications

- Saw cut at a minimum of 6 in. outside the joint (both sides) to a depth of 2 in. from top of wall to 2 ft below low water level.
- Remove concrete within the saw cuts to a minimum depth of 2-in. or to sound concrete.
- Install bolt-and-nut anchors on 6-in. centers.
- Seal joint as shown in Figure 34.
- Place forms that recess the joint inward 0.5 in. Maintain joint width of 0.5 in. to 0.75 in. at the face.
- Fill void with material meeting ASTM (2000) C928, Type R3. (A specification sheet for an example acceptable material (RapidSet) is shown in Figure 36).

Alternative 2 specifications

- Saw cut at a minimum of 6 in. outside of joint (both sides) to a depth of 2 in. from top of wall to 2 ft below low water level. It may be necessary to slightly modify the dimensions, depending on the standard size of available armor sections.
- Remove concrete within the saw cuts to a minimum depth of 8 in. or to sound concrete.

Shortform Product Specification for:
Rapid Set® Concrete Mix

SECTION 03 01 30 MAINTENANCE OF CAST-IN-PLACE CONCRETE
SECTION 03 01 40 MAINTENANCE OF PRECAST CONCRETE
SECTION 03 01 50 MAINTENANCE OF CAST DECKS AND UNDERLAYMENT
SECTION 03 01 70 MAINTENANCE OF MASS CONCRETE
(The above were formerly Section 03930)
[Note to specifier: Delete unnecessary Sections.]

Use Rapid Set® Concrete Mix: a cement based, rapid-setting, low shrinkage concrete repair material that can be used from 2 inches to 24 inches thick. Manufactured by CTS Cement Manufacturing Corp., 11065 Knott Avenue, Suite A, Cypress, CA, 90630. Phone: 800-929-3030. Website: www.rapidset.com

1. Set time per ASTM 191 (Mod.) at 70°F:
   Initial set 15 minutes. Final set 35 minutes.
2. Compressive strength per ASTM C39 (Mod.):
   1 hour* 2800 psi
   3 hour 3600 psi
   1 day 5000 psi
   28 day 6000 psi
   * after final set
3. Flexural strength per ASTM C78 (Mod.):
   2 hour* 420 psi
   1 day 650 psi
   28 day 750 psi
   * after final set
4. Shall meet ASTM C928 for packaged concrete repair materials.
5. Shall be non-metallic with no added chlorides and shall be pre-blended requiring only the additon of water.
6. Shall have small, pre-measured packets available to add to the repair material per manufacturer's recommendations to slow down the set time, increase fluidity and strength, speed up the set time, increase bond strength, reduce plastic and crazing cracks, lighten the color of the mix, or darken the color of the mix.

Water shall be potable. Prepare surface, mix, place, and cure per manufacturer's recommendations.

Figure 36. RapidSet, specification sheet (example acceptable material meeting ASTM (2000) C928 Type R3). (www.rapidset.com/Specs2005/PDFdocs/Cement_Spec_Short.pdf).

- Install internal bolt-and-nut anchors on 12-in. centers.
- Seal joint as shown in Figure 35.
- Install anchors for vertical steel wall armor extending into sound concrete on 12-in. centers (staggered off the internal anchor bolts).
- Fill void with material meeting ASTM (2000) C928, Type R3. (A specification sheet for an example acceptable material (RapidSet) is shown in Figure 36).
- Attach steel wall armor plates using Hilti (or equivalent) HVA Adhesive System and HAS anchor rods (www.us.hilti.com/fstore/holus/techlib/docs).
- Weld vertical steel wall armor plates to steel anchor bolts and to existing horizontal wall armor strips, and grind all welds flush.

Alternative 2 was selected for installation because it was believed the steel plates would provide more protection from barge impact at this particular vertical joint location where the lock chamber wall meets the approach wall on the upstream side of the chamber.

An order for supplies and services was issued to TJC Engineering, Inc., Louisville, KY, on 26 September 2006. The order requested TJC Engineering, Inc., to provide all labor, materials, and equipment for project IDIQ 06-30, John T. Myers 600-ft chamber upper land-wall approach repair in accordance with the Scope of Work dated 13 September 2006, as per the TJC Engineering, Inc. proposal dated 19 September 2006.

Scope of work

The contractor shall furnish and fit steel plate over the damaged approach wall located at the joint between monoliths L-35 and L-36. The replacement steel plate shall be 0.5 in. thick and cover a section on each side of the monolith joint, upstream monolith L-36 measuring 36 in. in width, downstream monolith L-35 measuring 48 in. in width, and both sides 20 ft (± 6 in.) in length. The plate shall extend from the bottom of the horizontal corner protection strip at the top of the wall to the last rub strip above the water line at normal pool (12.0 ft). If normal pool cannot be maintained by the Lock due to high water, the contractor may negotiate more time to complete the project. (The steel plate may be one or two pieces and bent to cover the entire area or two pieces welded together at the contractor's discretion.) The steel plate shall be fitted so that no angle or surface shall protrude past the plane (surface) of the existing corner. Material for the steel plate shall conform to ASTM (2005) A36, minimum yield strength of

36,000 psi and minimum tensile strength of 58,000 psi. The contractor shall provide documentation verifying adherence to this standard.

a. Saw cut at a minimum of 36 in. outside of joint on the upstream side of monolith L-36 and 48 in. outside of the joint on the downstream side of monolith L-35 to a depth of approximately 2.0 in. from the bottom of the horizontal corner protection steel at the top of the wall extending to the last rub strip above the water line at normal pool to cover all damaged areas above the water line.
b. Remove the concrete and rub strips within the saw cuts to a depth of approximately 2.0 in. or to sound concrete (a depth that will allow the top surface of the replacement plate to be flush with the top surface of the existing concrete). Weld and bevel the existing rub strips to smoothly blend with the new armor steel.
c. Layout and drill 60 7/8-in.-diam holes (approximately 2 ft on center) for Hilti (or equivalent) HVA Capsule Adhesive Anchoring System 4.2.1, using HAS anchor rods. (Specification sheets for an example acceptable material (Hilti) are shown in Figures 37a thru 37e.) Follow the manufacturer's recommendations for hole size, depth, and preparation. Install anchor bolts and allow epoxy to cure as per manufacturer's instructions. Using a material meeting ASTM (2000) C928, Type R3 (A specification sheet for an example acceptable material (RapidSet) is shown in Figure 36.), spread/pump a mixture to form a base for the backside of plate. Seal the monolith joint between the plates with a polyurethane closed-cell marine foam.
d. Install plate and torque into position. Follow all manufacturer instructions.
e. The contractor shall weld full length and full depth along top and bottom of horizontal length and at all available points on the vertical surfaces of armor place, and spot-weld all anchor bolts. Grind welds flush. The contractor shall take care not to overheat the metal, distort the armor plate, or damage the grout base. The contractor shall submit a written welding procedure specification for the work and provide verification that the welder is qualified for the work specified in the welding procedural specification.

Repair monitoring

Damage to the lock wall and armor system is caused by barge impact. Thus the damaged areas exist between the water line at normal pool elevation (12.0 ft) and the top of the lock wall (total distance of 20 ft ± 6 in.) (regions of the wall that the barge can strike). The vertical joint to be repaired

HVA Capsule Adhesive Anchoring System 4.2.1

4.2.1.1 Product Description

HVU Adhesive Capsule

HIS Internally Threaded Insert

HAS Anchor Rod Assembly with nut and washer

Rebar (Not supplied by Hilti)

4.2.1.1	Product Description
4.2.1.2	Material Specifications
4.2.1.3	Technical Data
4.2.1.4	Installation Instructions
4.2.1.5	Ordering Information

The Hilti HVA system is a heavy duty, two component adhesive anchor consisting of a self-contained adhesive capsule and either a threaded rod with nut and washer or an internally threaded insert.

Product Features

- High loading capacity
- Small edge distance and anchor spacing allowance
- Excellent dynamic load resistance
- Wide range of installation temperatures
- Excellent performance in holes cored using Hilti DD-B or DD-C diamond core bits
- Excellent elevated temperature performance
- Excellent performance in freezing and thawing conditions
- No hole brushing required—just blow out hole with compressed air—makes installation fast and easy
- Seismic qualified per IBC®/IRC® 2000 and UBC® 1997 (ICC-ES AC58) Please refer to ER-5369

Guide Specifications

Master Format Section:

03250 (Concrete accessories)

Related Sections:

03200 (Concrete Reinforcing Accessories)
05050 (Metal Fabrication)
05120 (Structural Steel)

Adhesive anchors shall consist of an all-thread anchor rod, nut, washer and adhesive capsule. Alternatively, adhesive anchors shall consist of a steel insert and an adhesive capsule.

Anchor Rod Shall be provided with 45 degree chisel or cut point to provide proper mixing of the adhesive components. Anchor rod shall be manufactured to meet the following requirements: **1.** ISO 898 Class 5.8 **2.** ASTM A 193 Grade B7 **3.** AISI 304 or AISI 316 stainless steel meeting the mechanical requirements of ASTM F 593 (Condition CW). **4.** Rebar with chisel or cut point.

Special order HAS Rod materials may vary from standard steel rod product.

Nuts and Washers Shall be furnished to meet the requirements of the above anchor rod specifications.

Adhesive Capsule Shall consist of a dual chamber foil capsule. The resin material shall be vinyl urethane methacrylate.

Steel Insert The internally threaded insert shall have a 45 degree (from central axis) chisel pointed end. The insert shall be carbon steel or stainless steel material which meets minimum ultimate tensile strengths of 66.7 and 101.5 ksi, respectively.

The adhesive anchoring system shall be the Hilti HVA anchoring system, consisting of the Hilti HVU adhesive capsule and the Hilti HAS anchor rod or HIS internally threaded insert.

Installation Adhesive anchors to be installed in holes drilled using the specified diameter of Hilti carbide tipped drill bit or matched tolerance Hilti DD-B or DD-C diamond core bit. Anchors shall be installed in strict accordance to section 4.2.1.4. Do not disturb until cure time has elapsed.

Listings/Approvals

City of Los Angeles
Research Report #25363
NSF/ANSI Std 61
certification for use in potable water
European Technical approval
ETA-05/0255
ETA-05/0256
ETA-05/0257

Code Compliance

IBC®/IRC® 2000 (ICC-ES AC58)
UBC® 1997 (ICC-ES AC58)
LEED®: Credit 4.1-Low Emitting Materials

The Leadership in Energy and Environmental Design (LEED®) Green Building Rating system™ is the nationally accepted benchmark for the design, construction and operation of high performance green buildings.

Figure 37a. Hilti HVA Capsule Adhesive Anchoring System 4.2.1, product description (example acceptable material) (continued).
(www.us.hilti.com/fstore/holus/techlib/docs/4.2_Adhesive_Anchoring_Systems_(150-288)r021.pdf).

4.2.1 HVA Capsule Adhesive Anchoring System

4.2.1.2 Material Specifications

	Mechanical Properties f_y ksi (MPa)	min. f_u ksi (MPa)
Standard HAS-E rod material meets the requirements of ISO 898 Class 5.8	58 (400)	72.5 (500)
High Strength or 'Super HAS' rod material meets the requirements of ASTM A 193, Grade B7	105 (724)	125 (862)
Stainless HAS rod material meets the requirements of ASTM F 593 (304/316) Condition CW 3/8" to 5/8"	65 (448)	100 (689)
Stainless HAS rod material meets the requirements of ASTM F 593 (304/316) Condition CW 3/4" to 1-1/4"	45 (310)	85 (586)
HIS Insert 11SMnPb30+C Carbon Steel conforming to DIN 10277-3	54.4 (375)	66.7 (460)
HIS-R Insert X5CrNiMo17122 K700 Stainless Steel conforming to DIN EN 10088-3	50.8 (350)	101.5 (700)

HAS Super & HAS-E Standard Nut material meets the requirements of ASTM A 563, Grade DH

HAS Stainless Steel Nut material meets the requirements of ASTM F 594

HAS Carbon Steel and Stainless Steel Washers meet dimensional requirements of ANSI B18.22.1 Type A Plain

HAS Super & HAS-E Standard Washers meet the requirements of ASTM F 436

All HAS-E & HAS Super Rods (except 7/8") & HAS-E Standard, HIS inserts, nuts & washers are zinc plated to ASTM B 633 SC 1

7/8" Standard HAS-E & HAS Super rods hot-dip galvanized in accordance with ASTM A 153

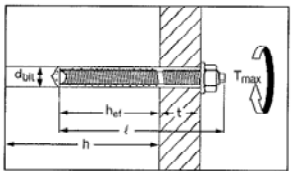
HVU Adhesive—Vinyl Urethane Methacrylate Resin with a Dibenzoyl Peroxide hardener

Note: Special Order steel rod material may vary from standard steel rod materials.

4.2.1.3 Technical Data

HAS Rod Specification Table

Details \ HAS Rod Size		in. (mm)	3/8 (9.5)	1/2 (12.7)	5/8 (15.9)	3/4 (19.1)	7/8 (22.2)	1 (25.4)	1-1/4 (31.8)
d_{bit}: nominal bit diameter[1, 2]		in.	15/32	9/16	11/16	7/8	1	1-1/8	1-3/8
$h_{ef} = h_{nom}$: std. depth of embed.[3] = capsule length		in. (mm)	3-1/2 (90)	4-1/4 (110)	5 (125)	6-5/8 (170)	6-5/8 (170)	8-1/4 (210)	12 (305)
t: max. thickness fastened[4]		in. (mm)	1 (25.4)	1-1/2 (38.1)	1-3/4 (44.5)	2 (50.8)	2-1/4 (57.2)	2-1/2 (63.5)	2-3/4 (69.9)
t_{max}: Max. tightening torque	All Hilti Rods	ft-lb (N·m)	18 (24)	30 (41)	75 (102)	150 (203)	175 (237)	235 (319)	400 (540)
h: minimum base material thickness[5]	$h_{ef} = h_{nom}$	in. (mm)	5-1/2 (140)	6-1/4 (160)	7 (180)	8-1/2 (220)	8-1/2 (220)	10-1/2 (270)	15 (380)
	$h_{ef} \neq h_{nom}$	in. (mm)	1.0 h_{ef}+ 2 (51)	1.0 h_{ef}+ 2 (51)	1.0 h_{ef}+ 2 (51)	1.0 h_{ef}+ 2 (51)	1.0 h_{ef}+ 2 (51)	1.0 h_{ef}+ 2-1/4 (57)	1.0 h_{ef}+ 3 (76)
Recommended Hilti Rotary Hammer Drill			TE 6, 16	TE 16, 25, 35, 46		TE 46, 56, 76		TE 76	



1 Use matched tolerance carbide tipped bits or Hilti matched tolerance DD-B or DD-C diamond core bit.

2 Hilti matched tolerance carbide tipped drill bits

3 Data available for varying embedments; see Load Tables.

4 When using standard length rods at standard embedment (h_{nom})

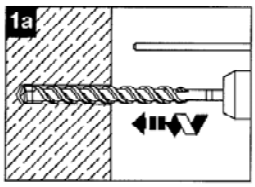
5 Minimum base material thickness given to avoid backside blowout from drilling. Ability of base material to withstand loads applied (e.g. bending of concrete slab) should be determined by design engineer.

Figure 37b. HVA Capsule Adhesive Anchoring System 4.2.1, material specifications and Technical Data (example acceptable material) (continued).
(www.us.hilti.com/fstore/holus/techlib/docs/4.2_Adhesive_Anchoring_Systems_(150-288)r021.pdf).

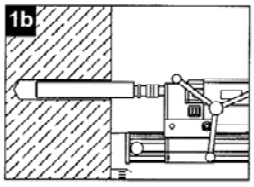
4.2.1 HVA Capsule Adhesive Anchoring System

4.2.1.4 Installation Instructions

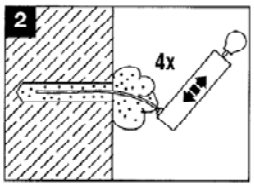
HAS Rod, Rebar and Insert Installation Instructions



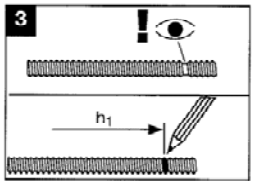
1a. Drilling the hole - Rotary hammer drill: Set the depth gauge to the correct drilling depth.



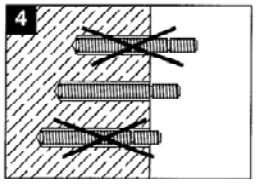
1b. Drilling the hole - Diamond coring: Mark the correct drilling depth on the height adjustment mechanism.



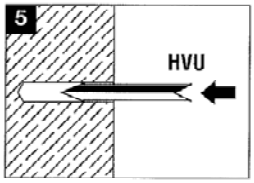
2. Clean the hole immediately before setting the anchor. Remove drilling dust and standing water from the base of the hole by blowing out well with at least 4 strokes of the blow-out pump, or using compressed air or an industrial vacuum cleaner. The anchor holes must be free of dust, water, ice, oil, bitumen, chemicals or any other foreign matter or contaminants. **Poorly-cleaned holes = poor hold.**



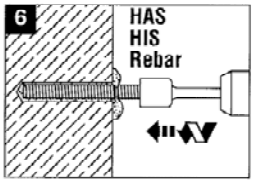
3. Ensure that the specified setting depth is marked on the anchor rod. If not, add an embedment mark, for example with tape or marker.



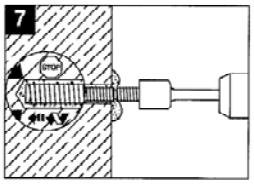
4. Caution! Check that the hole is drilled to the correct depth before setting the anchor. Hole depth is correct when the anchor rod contacts the base of the hole and the setting depth mark coincides with the concrete surface.



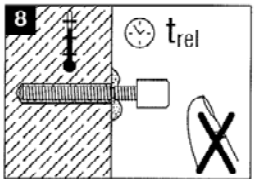
5. Push the anchor capsule into the drilled hole.



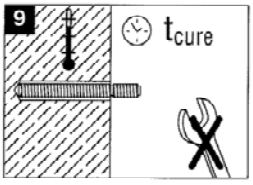
6. Use the setting tool at a speed of 250–1000 r.p.m. to drive the anchor rod into the hole, applying moderate pressure and with the hammering action switched on.



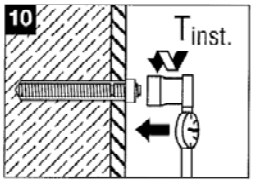
7. Switch off the rotary hammer drill immediately when the specified setting depth is reached (refer to mark on the anchor rod). After setting, adhesive mortar must fill the annular gap completely, right up to the concrete surface. Caution! Prolonged rotary action may cause mortar to be forced out of the hole, resulting in reduced anchor loading capacity.



8. The working time "t rel", which depends on base material temperature, must be observed (see fig. 10). The screwed-on setting tool may be removed only after the time "t rel" has elapsed.



9. After reaching the end of the working time "t rel", do not manipulate or disturb the anchor rod in any way until the curing time "t cure" has elapsed.



10. A load may be applied to the anchor only after the curing time "t cure" has elapsed.

11.

°C	°F	trel	tcure
min. -5°...0°	min.23°...32°	60'	5 h
0°...10°	32°...50°	30'	1 h
10°...20°	50°...68°	20'	30'
20°...max 40°	68°...max 104°	8'	20'

11. The working time "t rel" and curing time "t cure", which depend on base material temperature, must be observed!

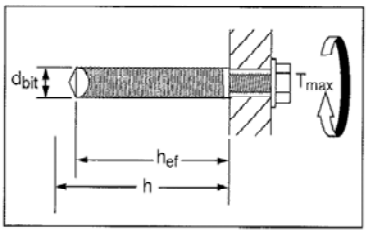
Figure 37c. Hilti HVA Capsule Adhesive Anchoring System 4.2.1, installation instructions (example acceptable material) (continued).
(www.us.hilti.com/fstore/holus/techlib/docs/4.2_Adhesive_Anchoring_Systems_(150-288)r021.pdf).

HVA Capsule Adhesive Anchoring System 4.2.1

HIS Insert Specification Table

Details \ HIS Insert		in. (mm)	3/8 (9.5)	1/2 (12.7)	5/8 (15.9)	3/4 (19.1)
HVU capsule required			1/2 x 4-1/4	5/8 x 5	7/8 x 6-5/8	1 x 8-1/4
d_{bit}	bit diameter[1]	in.	11/16	7/8	1-1/8	1-1/4
h_{nom}	std. depth of embed. = capsule length	in. (mm)	4-1/4 (110)	5 (125)	6-5/8 (170)	8-1/4 (210)
ℓ_{th}	useable thread length	in. (mm)	1 (25)	1-3/16 (30)	1-1/2 (40)	2 (50)
T_{max}	Max. tightening torque	ft lb (N·m)	18 (24)	30 (41)	75 (102)	150 (203)
h	min. base material thickness	in. (mm)	6-3/8 (162)	7-1/2 (191)	10 (254)	12-3/8 (314)
Recommended Hilti Rotary Hammer Drill			TE 6, 16, 25, 35	TE 16, 25, 35, 46	TE 46, 56, 76	

2 Hilti matched tolerance carbide tipped drill bits



Rebar Specification Table

	Rebar Size:		#4	#5	#6	#7	#8
d_{bit}	bit diameter[1, 2]	in.	5/8	13/16	1	1-1/8	1-1/4
h_{nom}	std. depth of embed. = capsule length	in. (mm)	4-1/4 (110)	5 (125)	6-5/8 (170)	6-5/8 (170)	8-1/4 (210)

1 Rebar diameters may vary; the witnessed test was performed using the above mentioned drill bit diameters. Rebar must have a minimum length 4" greater than embedment to accommodate the setting equipment.

2 Hilti matched tolerance carbide tipped drill bits

Metric Rebar Specification Table (Canada Only)

	Rebar Size :		10M	15M	20M	25M
d_{bit}	bit diameter[1]	in. or mm	9/16	13/16	1	32 mm
h_{nom}	std. depth of embed. = capsule length	mm (in.)	90 (3-1/2)	125 (5)	170 (6-5/8)	210 (8-1/4)

2 Hilti matched tolerance carbide tipped drill bits

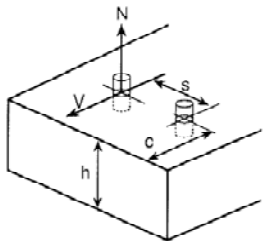
Combined Shear and Tension Loading

$$\left(\frac{N_d}{N_{rec}}\right)^{5/3} + \left(\frac{V_d}{V_{rec}}\right)^{5/3} \leq 1.0 \text{ (Ref. Section 4.1.8.3)}$$

Figure 37d. Hilti HVA Capsule Adhesive Anchoring System 4.2.1, HIS insert, rebar, and metric rebar specification tables (example acceptable material) (continued). (www.us.hilti.com/fstore/holus/techlib/docs/4.2_Adhesive_Anchoring_Systems_(150-288)r021.pdf).

4.2.1 HVA Capsule Adhesive Anchoring System

Anchor Spacing and Edge Distance Guidelines in Concrete

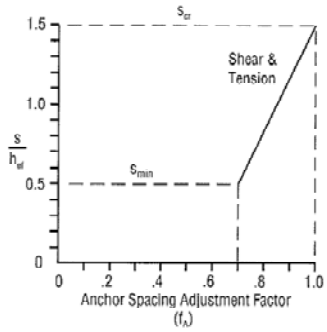


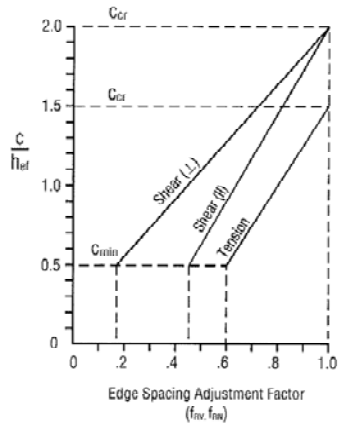
Anchor Spacing Adjustment Factors

s = Actual spacing
h_{ef} = Actual embedment
s_{min} = 0.5 h_{ef}
s_{cr} = 1.5 h_{ef}

Edge Distance Adjustment Factors

c = Actual edge distance
h_{ef} = Actual embedment
c_{min} = 0.5 h_{ef} Tension and shear
c_{cr} = 1.5 h_{ef} Tension
= 2.0 h_{ef} Shear
⊥ = Perpendicular to edge
II = Parallel to edge





Spacing Tension/Shear
s_{min} = 0.5 h_{ef}, s_{cr} = 1.5 h_{ef}
$f_A = 0.3(s/h_{ef}) + 0.55$
for $s_{cr}>s>s_{min}$

Edge Distance Tension
c_{min} = 0.5 h_{ef}, c_{cr} = 1.5 h_{ef}
$f_{RN} = 0.4(c/h_{ef}) + 0.40$
for $c_{cr}>c>c_{min}$

Edge Distance Shear (⊥ toward edge)
c_{min} = 0.5 h_{ef}, c_{cr} = 2.0 h_{ef}
$f_{RV1} = 0.54(c/h_{ef}) - 0.09$
for $c_{cr}>c>c_{min}$

Edge Distance Shear (II to or away from edge)
c_{min} = 0.5 h_{ef}, c_{cr} = 2.0 h_{ef}
$f_{RV2} = 0.36(c/h_{ef}) + 0.28$
for $c_{cr}>c>c_{min}$

Load Adjustment Factors for 3/8" Diameter Anchor

Anchor Diameter	3/8" diameter											
Adjustment Factor	Spacing Tension/Shear, f_A			Edge Distance Tension, f_{RN}			Edge Distance Shear (⊥ toward edge), f_{RV1}			Edge Distance Shear (II to or away from edge), f_{RV2}		
Embedment Depth, in.	3-1/2	5-1/4	7	3-1/2	5-1/4	7	3-1/2	5-1/4	7	3-1/2	5-1/4	7
Spacing (s)/Edge Distance (c), in.												
1-3/4	0.70			0.60			0.18			0.46		
2	0.72			0.63			0.22			0.49		
2-5/8	0.78	0.70		0.70	0.60		0.32	0.18		0.55	0.46	
3	0.81	0.72		0.74	0.63		0.37	0.22		0.59	0.49	
3-1/2	0.85	0.75	0.70	0.80	0.67	0.60	0.45	0.27	0.18	0.64	0.52	0.46
4	0.89	0.78	0.72	0.86	0.70	0.63	0.53	0.32	0.22	0.69	0.55	0.49
4-1/2	0.94	0.81	0.74	0.91	0.74	0.66	0.60	0.37	0.26	0.74	0.59	0.51
5-1/4	1.00	0.85	0.78	1.00	0.80	0.70	0.72	0.45	0.32	0.82	0.64	0.55
6		0.89	0.81		0.86	0.74	0.84	0.53	0.37	0.90	0.69	0.59
7		0.95	0.85		0.93	0.80	1.00	0.63	0.45	1.00	0.76	0.64
7-7/8		1.00	0.89		1.00	0.85		0.72	0.52		0.82	0.69
8-1/2			0.89			0.86		0.78	0.57		0.86	0.72
9			0.91			0.89		0.84	0.60		0.90	0.74
10			0.94			0.91		0.94	0.68		0.97	0.79
10-1/2			0.96			0.94		1.00	0.72		1.00	0.82
12			0.98			0.97			0.84			0.90
13			1.00			1.00			0.91			0.95
14									1.00			1.00

Figure 37e. Hilti HVA Capsule Adhesive Anchoring System 4.2.1, anchor spacing and edge distance guidelines in concrete (example acceptable material) (concluded). (www.us.hilti.com/fstore/holus/techlib/docs/4.2_Adhesive_Anchoring_Systems_(150-288)r021.pdf).

where the bullnose intersects the 600-ft chamber upper land-wall is shown in Figure 38. Close-up photos of the most damaged portion of concrete are shown in Figures 39 and 40.

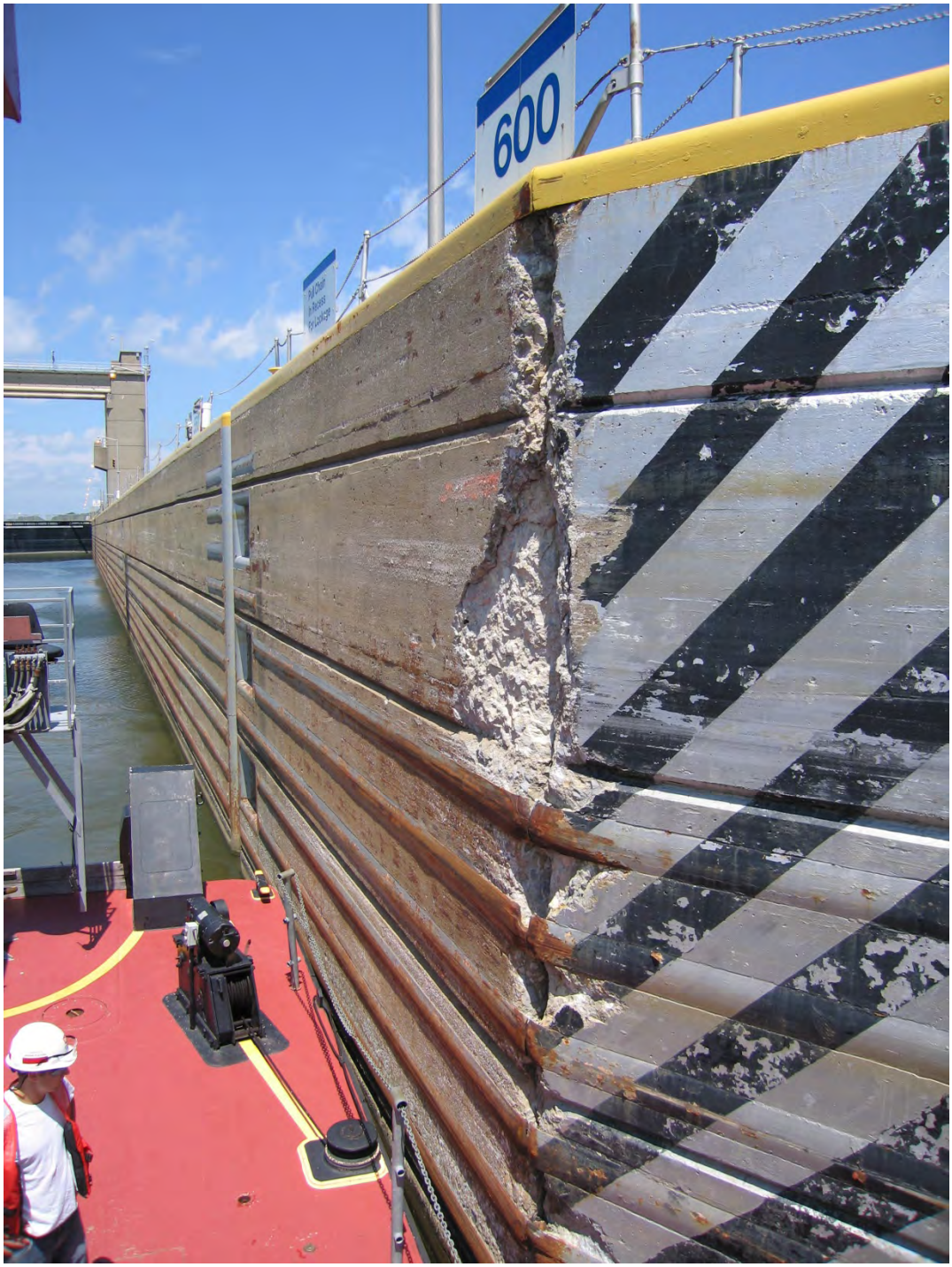


Figure 38. Damaged vertical joint of 600-ft chamber upper land-wall approach to be repaired.

Figure 39. Closeup photo of major concrete damage at the vertical joint to be repaired.

Figure 40. Template lines for identifying changes to damage area during monitoring.

The replacement steel plates would be 0.5 in. thick and 36 in. wide on the upstream monolith and 48 in. wide on the downstream monolith, for a total coverage of 7 ft wide. At the contractor's discretion, this steel plate could be one piece factory bent to fit the profile of the two monoliths, or it could be two pieces placed individually and welded at all seams. The total steel plate coverage would then be 140 sq ft. The contractor chose to install four pieces of steel plate, each 5-ft-high (20 ft total height) and 7-ft-wide, and each factory bent to fit the monoliths. Each individual steel plate covered 35 sq ft, and the total coverage for the four steel plates was the required 140 sq ft. Figure 41 shows the 7-ft-wide by 5-ft-tall factory bent steel plates arriving at the work site.

A pontoon float multi-level work barge was assembled for performing the repairs to the lock vertical joint. The scaffolding on the 20-ft-long by 8-ft-wide self-propelled work barge was two tiers high. Figure 42 shows the contract crew sawing and power chiseling deteriorated concrete from the region to be repaired with high-early-strength, quick-setting concrete. (In this photo, the bottom section of 7-ft-wide by 5-ft-tall steel plate had already been installed, and the contract crew was preparing to install the second section of steel plate.)

Figure 41. 7-ft-wide by 5-ft-tall steel plates arriving at the lock wall repair project.

Figure 42. Pontoon float work barge for performing repairs to lock vertical joint.

Saw cuts by a diamond blade saw were made along the template lines 36 in. outside the joint on the upstream monolith L-36 and 48 in. outside the joint on the downstream monolith L-35 for a minimum of 2 in. deep or to sound concrete. A hand-held impact concrete breaker was used to dislodge concrete within the sawed area being repaired (Figure 43).

Figure 43. Sawed concrete being removed by impact hammer.

The existing wall armor rub strips were beveled to smoothly blend with the new armor steel plate. The concrete surface within the repair region was cleaned by flushing with river water under high pressure to remove any debris that was left after removal of deteriorated concrete. Figure 44 is a close-up photo of the cavity prior to installation of internal bolt-and-nut anchors. Here can be seen the lower 5-ft-high steel plate that was previously installed, with the locations of the steel plate anchor rods visible after the rods were welded to the steel plate and smooth finished down to the surface of the plate.

Internal bolt-and-nut anchors were next installed as shown in Figure 35 (Repair Alternative 2, stay-in-place steel plate armor form) to hold the rapid-set, high-early-strength concrete in place.

Figure 44. Cavity prior to installation of concrete anchor bolts and steel plate anchor rods. Bottom section of steel plate is shown with anchor rods welded to plate and smooth finished.

The vertical joint between the two monoliths was sealed with polyurethane closed cell marine foam. The cavity void was then filled with rapid-set, high-early-strength concrete grout material meeting ASTM (2000) C928, Type R3 (or equivalent) to form a base for the backside of the plate. A specification sheet for an example of acceptable cement-based grout mixture is shown in Figure 36.

Next, 7/8-in.-diam steel plate anchor bolt holes were drilled into sound concrete on approximately 2-ft centers, staggered off the internal bolt-and-nut anchors installed previously to hold the rapid-setting concrete. The steel plate was attached with Hilti (or equivalent) HVA Capsule Adhesive Anchoring System 4.2.1 using HAS anchor rods (Specification sheets for an example of acceptable material are shown in Figures 37a through 37e.) The manufacturer's recommendations pertaining to hole size, depth, and preparation were followed. The steel plate anchor bolts were installed and torqued into position, and the epoxy was allowed to cure as per the manufacturer's instructions. All steel plate anchor rods were then full welded to the steel plate. All horizontal joints between the steel plates were welded full length and full depth, and the steel plates

were full welded to the wall armor rub strips that were cut to butt-weld to the steel plates. All welds were ground flush, and the steel plates were painted to conform to the lock structure appearance.

This repair technique effectively connected two concrete monoliths with one continuous steel plate that was firmly anchored into the sound concrete of both individual monoliths. The completed repair is shown in Figure 45 at low water pool elevation and in Figure 46 at high water elevation on the Ohio River.

Figure 45. Completed repair of 600-ft chamber upper land-wall approach vertical joint at low water pool elevation, John T. Myers Locks and Dam.

Figure 46. Completed repair of 600-ft chamber upper land-wall approach vertical joint at high water elevation on the Ohio River.

4 Monitoring Repair of 600-ft Chamber Lower Land-Wall Approach Vertical Joint

Subsequent to the repair of the 600-ft chamber upper land-wall approach vertical joint (September 2006) by essentially connecting two concrete monoliths together with a steel plate firmly attached to each individual monolith, a weld was found to have cracked due to elemental movement between the two monoliths caused by uneven settlement or thermal expansion/contraction. Hence, the demonstration of a fundamentally different repair technique devised for the 600-ft chamber lower land-wall approach vertical joint was well-founded from the standpoint of an unexpected consequence as well as an alternative repair methodology.

An order for supplies and services was issued to TJC Engineering, Inc., Louisville, KY, on 11 September 2007. The order requested TJC Engineering, Inc., to provide all labor, materials, and equipment for project IDIQ 17-46, John T. Myers 600-ft chamber lower land-wall approach repair in accordance with the Scope of Work dated 14 August 2007, as per the TJC Engineering, Inc. proposal dated 06 September 2007.

The lock wall design at the lower vertical joint is significantly different from the wall design at the upper lock (Figure 47). Here, the wall armor rub strips extend all the way to the top of the wall on monolith L-2 (the main lock chamber wall), but terminate about 13.5 ft below the lock wall on monolith L-1 (the guide wall entrance into the main lock chamber).

Scope of work

The contractor shall furnish, fabricate, and install supplemental wall armor and repair concrete damage/deterioration located on the 600-ft lock chamber's lower land-side approach wall at the joint between monoliths L-1 and L-2. The area of repair to the wall armor may span up to a height of 35 ft and a width of 2 ft 0.75 in. (1 ft on each side of expansion joint, and 0.5 in. to 0.75 in. expansion joint width). The area of repair to the concrete will be similar with exception to filling a 2-ft-wide by approximately 5-ft-long void from missing wall armor near the top of monolith L-2.

Figure 47. 600-ft chamber lower land-wall vertical joint selected for repair monitoring.

The supplemental wall armor consists of the following.

a. Twenty-nine 0.75-in.-thick steel plates with a finished overall face dimension of 16-in.-high by 12-in.-wide, and each plate having four anchor rod holes with each hole located approximately 3 in. from each edge of the four corners of the plates; and also with a 16-in.-high by 4-in.-wide overall dimensioned 0.5-in.-thick flange turned into the structure's expansion joint.
b. One 0.75-in.-thick steel plate with a finished overall face dimension of 22-in.-high by 12-in.-wide, and six anchor rod holes with four of the holes located approximately 3 in. from each edge of the four corners of the plate and two of the holes located approximately 3 in. from each side and approximately 11 in. down from the top edge of the plate; and also with a 22-in.-high by 4-in.-wide overall dimensioned 0.5-in.-thick flange turned into the structure's expansion joint,

c. One 0.75-in.-thick steel plate with a finished overall face dimension of 28-in.-high by 12-in.-wide, and six anchor rod holes with four of the holes located approximately 3 in. from each edge of the four corners of the plate and two of the holes located approximately 3 in. from each side and approximately 14 in. down from the top edge of the plate; and also with a 28-in.-high by 4-in.-wide overall dimensioned 0.5-in.-thick flange turned into the structure's expansion joint.
d. One 0.75-in.-thick steel plate with a finished overall face dimension of 13.5-ft-high by 12-in.-wide, and 16 anchor rod holes with four of the holes located approximately 3 in. from each edge of the four corners of the plate and 12 of the holes located approximately 3 in. from each side and approximately every 1 ft down from the top holes; and also with a 13.5-ft-high by 4-in.-wide overall dimensioned 0.5-in.-thick flange turned into the structure's expansion joint.

Monolith L-1 will require 12 16-in.-high by 12-in.-wide supplemental wall armor plates, and one 13.5-ft-high by 12-in.-wide supplemental wall armor plate.

Monolith L-2 will require 17 16-in.-high by 12-in.-wide supplemental wall armor plates, one 22-in.-high by 12-in.-wide supplemental wall armor plate, and one 28-in.-high by 12-in.-wide supplemental wall armor plate.

Material for the supplemental wall armor plates shall conform to ASTM (2005) A36, minimum yield strength of 36,000 psi and minimum tensile strength of 58,000 psi. The contractor shall provide documentation verifying adherence to this standard.

The 16-in.-high by 12-in.-wide supplemental wall armor plates shall be located in the spaces between the existing horizontal straight-run wall armor. The contractor shall create an accommodating recess by cutting 2-in.-high by 12.5-in.-wide notches in the existing horizontal straight-run wall armor (which are positioned with approximately 12.25 in. of space between them), and saw cutting and removing the concrete to a depth of approximately 2 in., or to sound concrete (a depth that will allow the top surface of the supplemental wall armor plates to be flush with the top surface of the existing horizontal straight run wall armor at the same plane as the edge of the notches).

Layout and drill four holes in the concrete (to match the hole locations of the anchor rod holes of the 16-in.-high by 12-in.-wide supplemental wall armor plates) for Hilti (or equivalent) HVA Adhesive Anchor System using 0.75-in. HAS anchor rods. (Specification sheets for an example acceptable material are shown in Figures 37a thru 37e.) Follow manufacturer's recommendations for hole size, depth, and preparation. Install anchor bolts and allow epoxy to cure as per manufacturer's instructions. The supplemental wall armor plates shall be positioned to maintain a monolith joint width of 0.5 in. to 0.75 in. at the face.

Install the 16-in.-high by 12-in.-wide supplemental wall armor plates onto the anchor bolts utilizing threaded nuts on each side of the wall armor plate, adjusted to maintain the top surface of the supplemental wall armor plates to be flush with the top surface of the existing horizontal straight run wall armor at the same plane as the edge of the cut-out notches.

The contractor shall weld full length and full depth along top and bottom of the horizontal length of the supplemental wall armor plates, including the 2-in. vertical seams at end of the cut-out notches. The outer anchor rod nuts shall then be removed, and anchor rods cut and welded solid to the armor plate and ground flush with the face of armor plate. The contractor shall take care not to overheat or distort armor plate or damage concrete or grout base. The contractor shall submit a written welding procedure specification for the work and provide verification that the welder is qualified for the work specified in the welding procedural specification.

The contractor shall seal the monolith joint with polyurethane closed-cell marine foam maintaining a joint width from 0.5 in. to 0.75 in. wide. (Specification sheets for an example acceptable material (Fomo Products, Inc.) are shown in Figures 48a and 48b).

The contractor shall spread/pump a grout mixture to fill the void and form a base for the backside of the armor plate using a material meeting ASTM (2000) C928, Type R3. (Specification sheets for an example acceptable material (BASF) are shown in Figures 49a thru 49d.)

The 28-in.-high by 12-in.-wide supplemental wall armor plate for monolith L-2 shall be installed in the same manner as described for the 16-in.-high by 12-in.-wide supplemental wall armor plates. This armor plate will span across the area of missing horizontal straight-run wall armor on monolith L-2, second run from the top.

Slow Rise (SR) Two-Component Froth

Handi-Foam® SR is a multiple purpose two-component polyurethane froth, that is designed within the international guidelines for protection of the ozone layer, and with respect to the Montreal Protocol, 1987, and other environmental guidelines, utilizing a non-flammable non-ozone depleting blowing agent to assist in the safety of the end user and the environment. The pre-pressurized, portable two-component froth foam systems are dispensed through the state-of-the-art Handi-Gun® two-component froth dispensing unit. The product designation "SR" stands for "Slow-Rise" to indicate the slower expanding and curing properties, compared to traditional "quick-cure" two-component froth products.

Application Area
Handi-Foam SR is specifically designed for filling cavities, molds, fixtures or holes where a slower curing and expanding polyurethane foam system is required. The foam can be dispensed into clean and dry voids of various size to insulate, fill, seal, deaden sound and reduce vibration.

Properties
The patented and user-friendly II-100 SR and II-200 SR packaging system provides many unique advantages, including:

- factory-attached dispensing hoses. No need to attach hoses prior to use.
- handle is secured to tanks. No more handle popping out of the box. No more tanks falling out of the box when the box gets wet.
- easy to open box for immediate use.
- hoses extend from top of tanks. More reach. More stability.

Handi-Foam Slow Rise two-component froth systems will expand upon chemical reaction of A component and B component to a final volume that is 3 to 5 times the dispensed volume, in typical applications, and may be as much as 8 times the dispensed volume in specific applications, depending on various factors such as cavity size, ambient conditions, etc. The foam will cure to a semi-rigid closed cell foam upon the chemical reaction of component A (polymeric isocyanate) with component B (a polyol blend containing certain additives).

Handi-Foam SR adheres to almost all building materials with the exception of surfaces such as polyethylene, Teflon®, silicone, oils and greases, mold release agents, and similar materials.

Optimal application temperature is 75°F (24°C); however, may be dispensed into colder or warmer substrates, with slight effects on the foam characteristics. See the Product Storage section for temperature information. Cured foam is resistant to heat and cold, -200 to +200°F (–129 to +93°C), and to aging, but not UV rays (i.e. sunlight) unless painted, covered or coated. Cured foam is also chemically inert and non-reactive in approved applications.

Handi-Foam SR systems require no outside mechanical or electrical power source and are available in various refillable and non-refillable sizes to meet specific job application requirements. When dispensed, the foam will create a seamless, continuous seal to insulate and protect against dust, pests, air infiltration, etc.

Preparation For Use
Application area must be dry, firm, free of loose particles, dust, grease and mold release agents. Protect surfaces not to be foamed. For pouring or mold filling applications, clamping of the mold is generally required to provide uniform support against foaming pressure. Extent of this clamping should be determined based on application and desired results. Consult the Handi-Foam SR User Manual (A13356) for more application information.

Shake disposable kits well *before* using.

Application/Use
Slow-Rise foam should be filled into cavities in excess of the theoretical "free-rise" volume. This is important in order that the foam is "packed" to a higher in-place density, thereby achieving optimum physical properties and dimensional stability. A calculated in-place density of 2 lbs./ft.³ (32 kg./m.³) is recommended and shown for specification purposes.

After following instructions for set-up, kits are ready to use. Attach appropriate hose to A and B tanks if needed. Open tank valves as directed. Materials are dispensed through the hoses and mixed in the disposable nozzle. With a nozzle attached to the two-component froth dispensing unit, dispense foam by squeezing the trigger. To interrupt or stop the foaming process, release the trigger. Once foaming process has stopped, the dispensing unit must be reactivated within 30 seconds or a new nozzle must be installed. Fresh foam may be applied in several stages to reduce overfilling of void or damage to non-rigid, confined cavities.

Product Storage
Store in cool dry area. Do not expose to open flame or temperatures above 120°F (49°C). Excessive heat can cause premature aging of components resulting in a shorter shelf life. Handi-Foam SR is reusable by following product instructions. For optimum results of refill systems, chemical temperature must be between 75-85°F (24-29°C). During colder months it may take up to a week or more to warm the chemicals to optimum temperature. Construction of a temperature controlled "hot box" is recommended for all applications in order to store the refill systems at a consistent, controlled temperature prior to and during use. Cured foam is non-toxic. **KEEP OUT OF REACH OF CHILDREN.**

Fomo Products, Inc.
A Member of the FIM Group of Companies
management system registered to ISO 9001:2000

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Figure 48a. Fomo Products, Inc., Handi Foam polyurethane marine foam technical data sheet (example acceptable material) (www.fomo.com/handi-foam_two_component.aspx) (continued).

Technical Data

	FREE-RISE DENSITY	IN-PLACE DENSITY
Density ASTM D-1622	1.75 lb./ft.³ (28 kg./m.³)	2.0* lb./ft.³ (32 kg./m.³)
K-Factor (per inch) ASTM C-518		0.168 BTU/(ft.) (h) (°F) (.024w./m.k.)
Tensile Strength ASTM D-1623		
Parallel		42 psi (286 kPa)
Perpendicular		28 psi (191 kPa)
Compressive Strength, ASTM D-1621		
Parallel @ 10%		14 psi (95 kPa)
Perpendicular @ 10%		15 psi (101 kPa)
Shear Strength ASTM C-273		
Parallel		21 psi (142 k Pa)
Perpendicular		19 psi (128 k Pa)
Dimensional Stability ASTM D-2126		
Heat Age (+ 158 °F/ 70°C; 28 days)		-4.5%
Humid Age (+150°F/ 70°C; 100% RH; 28 days)		-1.0%
Cold Age (-4°F/ -20°C)		-0.3%
Closed Cell Content ASTM D-2856		92%
Elongation		
Parallel		3.4%
Perpendicular		3.9%
Tack-Free / Expansion Time		60-90 sec.
Cuttable		5-10 min
Fire Rating DIN 4102-1		B2

Tank Specifications

	Systems 150 & 250	Systems 550 & 950
Capacity	27 gal. (102 l.)	120 gal. (454 l.)
Dimensions		
Height	54" (137 cm.)	61" (155 cm.)
Diameter	15" (38 cm.)	30" (76 cm.)
Base	20 x 20" (51 cm. x 51 cm.)	30" x 30 (76 cm. x 76 cm).
Empty Weight	120 lbs (54 kg.)	360 lbs. (163 kg.)
Filled Weight*	Sys 150: 265 lbs. (120 kg.) Sys 250: 350 lbs. (159 kg.)	Sys 550: 860 lbs. (390 kg.) Sys 950: 1190 lbs. (540 kg.)

*Filled tank weights are approximate for estimation purposes only. Actual gross weight is formulation specific and may be slightly higher or lower.

Approvals / Standards

Handi-Foam SR® systems meet the Coast Guard specification requirements for flotation in Title 33 code of Federal Regulations, paragraph 183.114 and meet the requirements of DIN 4102-1 for a "B2" building material.

Dispensing unit is patented under U.S. patent number 6,345,776. Other foreign and domestic patents pending.

ODP (Ozone Depletion Potential): Contains non-ozone depleting, non-flammable HFC propellant.

Theoretical Yield*

	2.0 PCF IN-PLACE DENSITY
Non-Refillable	
II-100 SR P10709	7.6 ft.³ (.22 m.³)
II-200 SR P10732	13 ft.³ (.37 m.³)
II-600 SR P10760	43.7 ft.³ (1.24 m.³)
Refillable	
System 150 SR P22045	128 ft.³ (3.6 m.³)
System 250 SR P22245	213 ft.³ (6.0 m.³)
System 550 SR P22445	458 ft.³ (13.0 m.³)
System 950 SR P22645	787 ft.³ (22.3 m.³)

*Physical properties and yields are based on an in-place density of 2.0 lbs /ft.³ Actual yields may vary depending on in-place density, application, and ambient conditions. Consult technical information for recommended application procedures.

Important Note:
Use only in well ventilated areas or with certified respiratory protection. Wear impervious gloves, protective eyewear and suitable work clothing when using. Read all instructions and safety information (MSDS) prior to use of any product. The product contains no formaldehyde

Always read all operating, application and safety instructions before using any products from Fomo Products, Inc. Use in conformance with all local, state and federal regulations and safety requirements. Failure to strictly adhere to any recommended procedures and reasonable safety precautions shall release Fomo Products of all liability with respect to the materials or the use thereof. For additional information and location of your nearest distributor, call Fomo Products, Inc. +1 330.753.4585 or +1 800.321.5585, Outside Ohio.

NOTE: Physical properties shown are typical and are to serve only as a guide for engineering design. Results are obtained from specimens under ideal laboratory conditions and may vary upon use, temperature and ambient conditions. Right to change physical properties as a result of technical progress is reserved. This information supersedes all previously published data. Yields shown are based on theoretical calculations and will vary depending on ambient conditions and particular application. Read all product directions and safety information before use. Consult local building codes for specific requirements regarding the use of cellular plastics or urethane products in construction.

WARNINGS: Follow safety precautions and wear protective equipment as recommended. Consult Material Safety Data Sheet (MSDS) for specific information. Use only with adequate ventilation or certified respiratory protection. NIOSH approved positive pressure supplied air respirator is recommended if exposure guidelines may be exceeded. Contents may be very sticky and irritating to skin and eyes, therefore wear protective eyewear, impervious gloves, and suitable work clothing when operating. If liquid chemical comes in contact with skin, first wipe thoroughly with dry cloth, then rinse affected area with water. Wash with soap and water afterwards, and apply hand lotion if desired. If liquid comes in contact with eyes, immediately flush with large volume of clean water for at least 15 minutes and get medical help at once. If liquid is swallowed, get immediate medical attention. Products manufactured or produced from these chemicals are organic and, therefore, combustible. Each user of any product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage. **KEEP OUT OF REACH OF CHILDREN.**

LIMITED WARRANTY: The Manufacturer warrants only that the product shall meet its specifications: THIS WARRANTY IS IN LIEU OF ALL WRITTEN OR UNWRITTEN, EXPRESSED OR IMPLIED WARRANTIES AND THE MANUFACTURER EXPRESSLY DISCLAIMS ANY WARRANTY OF MERCHANTABILITY, OR FITNESS FOR A PARTICULAR PURPOSE. The buyer assumes all risks whatsoever as to the use of the material. Buyer's exclusive remedy as to any breach of warranty, negligence or other claim shall be limited to the replacement of the material. Failure to strictly adhere to any recommended procedures shall release The Manufacturer of all liability with respect to the materials or the use thereof. User of this product must determine suitability for any particular purpose, including, but not limited to, structural requirements, performance specifications and application requirements prior to installation and after product is applied.

HANDI FOAM

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Figure 48b. Fomo Products, Inc., Handi Foam polyurethane marine foam technical data sheet continued (example acceptable material) (www.fomo.com/handi-foam_two_component.aspx) (concluded).

BASF

The Chemical Company

PRODUCT DATA

3 03 62 13 **Non-Metallic Non-Shink Grouting**

MASTERFLOW® 928

High-precision mineral-aggregate grout with extended working time

Description

Masterflow® 928 grout is a hydraulic cement-based mineral-aggregate grout with an extended working time. It is ideally suited for grouting machines or plates requiring precision load-bearing support. It can be placed from fluid to damp pack over a temperature range of 45 to 90° F (7 to 32° C). Masterflow® 928 grout meets the requirements of ASTM C 1107, Grades B and C, the Army Corp of Engineers' CRD C 621, Grades B and C, at a fluid consistency over a 30-minute working time and ANSI/NSF 61 approved Suitable for use with potable water.

Yield

One 55 lb (25 kg) bag of Masterflow® 928 grout mixed with approximately 10.5 lbs (4.8 kg) or 1.26 gallons (4.8 L) of water, yields approximately 0.50 ft³ (0.014 m³) of grout.

The water requirement may vary due to mixing efficiency, temperature, and other variables.

Packaging

55 lb (25 kg) multi-wall paper bags

3,300 lb (1,500 kg) bulk bags

Shelf Life

1 year when properly stored

Storage

Store in unopened bags in clean, dry conditions.

Features

- Extended working time
- Can be mixed at a wide range of consistencies
- Freeze/thaw resistant
- Hardens free of bleeding, segregation, or settlement shrinkage
- Contains high-quality, well-graded quartz aggregate
- Sulfate resistant
- ANSI / NSF 61 approved

Benefits

Ensures sufficient time for placement

Ensures proper placement under a variety of conditions

Suitable for exterior applications

Provides a maximum effective bearing area for optimum load transfer

Provides optimum strength and workability

For marine, wastewater, and other sulfate-containing environments

Suitable for use with potable water

Where to Use

INDUSTRIES

- Power generation
- Pulp and paper mills
- Steel and cement mills
- Stamping and machining
- Water and waste treatment
- General construction

APPLICATIONS

- Where a nonshrink grout is required for maximum effective bearing area for optimum load transfer
- Where high one-day and later-age compressive strengths are required
- Applications requiring a pumpable grout
- Compressors and generators
- Pump bases and drive motors
- Tank bases
- Conveyors
- Grouting anchor bolts, rebar and dowel rods
- Nonshrink grouting of precast well panels, beams, columns, curtain walls, concrete systems and other structural and non-structural building components
- Repairing concrete, including grouting voids and rock pockets

LOCATIONS

- Interior or exterior
- Marine applications
- Freeze/thaw environments

How to Apply

Surface Preparation

1. Steel surfaces must be free of dirt, oil, grease, or other contaminants.
2. The surface to be grouted must be clean, SSD, strong, and roughened to a CSP of 5 – 9 following ICRI Guideline 03732 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Etchant (see Form No. 1020198) to achieve the required surface profile.
3. When dynamic, shear or tensile forces are anticipated, concrete surfaces should be chipped with a "chisel-point" hammer, to a roughness of (plus or minus) 3/8" (10 mm). Verify the absence of bruising following ICRI Guideline 03732.
4. Concrete surfaces should be saturated (ponded) with clean water for 24 hours just before grouting.
5. All freestanding water must be removed from the foundation and bolt holes immediately before grouting.

Figure 49a. BASF MasterFlow 928 cement-based mineral-aggregate grout product data sheet (example acceptable material) (www.basf-cc.com.jo/en/products/Grouting/masterflow928uw/Pages/default.aspx) (continued).

Technical Data

Composition

Masterflow® 928 is a hydraulic cement-based mineral-aggregate grout.

Compliances

- ASTM C 1107, Grades B and C, and CRD 621, Grades B and C, requirements at a fluid consistency over a temperature range of 40 to 90° F (4 to 32° C)
- City of Los Angeles Research Report Number RR 23137
- ANSI / NSF 61 for use with potable water

Test Data

PROPERTY	RESULTS			TEST METHODS
Compressive strengths, psi (MPa)				ASTM C 942, according to ASTM C 1107
		Consistency		
	Plastic[1]	**Flowable**[2]	**Fluid**[3]	
1 day	4,500 (31)	4,000 (28)	3,500 (24)	
3 days	6,000 (41)	5,000 (34)	4,500 (31)	
7 days	7,500 (52)	6,700 (46)	6,500 (45)	
28 days	9,000 (62)	8,000 (55)	7,500 (52)	
Volume change*				ASTM C 1090
	% Change	**% Requirement of ASTM C 1107**		
1 day	> 0	0.0 – 0.30		
3 days	0.04	0.0 – 0.30		
14 days	0.05	0.0 – 0.30		
28 days	0.06	0.0 – 0.30		
Setting time, hr:min				ASTM C 191
		Consistency		
	Plastic[1]	**Flowable**[2]	**Fluid**[3]	
Initial set	2:30	3:00	4:30	
Final set	4:00	5:00	6:00	
Flexural strength,* psi (MPa)				ASTM C 78
3 days	1,000 (6.9)			
7 days	1,050 (7.2)			
28 days	1,150 (7.9)			
Modulus of elasticity,* psi (MPa)				ASTM C 469, modified
3 days	2.82×10^6 (1.94×10^4)			
7 days	3.02×10^6 (2.08×10^4)			
28 days	3.24×10^6 (2.23×10^4)			
Coefficient of thermal expansion,* in/in/° F (mm/mm/° C)	6.5×10^{-6} (11.7×10^{-6})			ASTM C 531
Split tensile and tensile strength,* psi (MPa)				ASTM C 496 (splitting tensile) ASTM C 190 (tensile)
	Splitting Tensile	**Tensile**		
3 days	575 (4.0)	490 (3.4)		
7 days	630 (4.3)	500 (3.4)		
28 days	675 (4.7)	500 (3.4)		
Punching shear strength,* psi (MPa), 3 by 3 by 11* (76 by 76 by 279 mm) beam				BASF Method
3 days	2,200 (15.2)			
7 days	2,260 (15.6)			
28 days	2,650 (18.3)			
Resistance to rapid freezing and thawing	300 Cycles RDF 99%			ASTM C 666, Procedure A

[1]100 – 125% flow on flow table per ASTM C 230

[2]125 – 145% flow on flow table per ASTM C 230

[3]25 to 30 seconds through flow conc per ASTM C 939

*Test conducted at a fluid consistency

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

Figure 49b. BASF MasterFlow 928 cement-based mineral-aggregate grout technical data sheet (example acceptable material) (www.basf-cc.com.jo/en/products/Grouting/masterflow928uw/Pages/default.aspx) (continued).

Test Data, continued

PROPERTY	RESULTS			TEST METHODS
Ultimate tensile strength and bond stress				ASTM E 488, tests*
Diameter in (mm)	**Depth in (mm)**	**Tensile strength lbs (kg)**	**Bond stress psi (MPa)**	
5/8 (15.9)	4 (101.6)	23,500 (10,575)	2,991 (20.3)	
3/4 (19.1)	5 (127.0)	30,900 (13,905)	2,623 (18.1)	
1 (25.4)	6.75 (171.5)	65,500 (29,475)	3,090 (21.3)	

*Average of 5 tests in ≥ 4,000 psi (27.6 MPa) concrete using 125 ksi threaded rod in 2" (51mm) diameter, damp, core-drilled holes.

Notes:

1. Grout was mixed to a fluid consistency.
2. Recommended design stress: 2,275 psi (15.7 MPa).
3. Refer to the "Adhesive and Grouted Fastener Capacity Design Guidelines" for more detailed information.
4. Tensile tests with headed fasteners were governed by concrete failure.

Jobsite Testing

If strength tests must be made at the jobsite, use 2" (51 mm) metal cube molds as specified by ASTM C 942 and ASTM C 1107. DO NOT use cylinder molds. Control field and laboratory tests on the basis of desired placement consistency rather than strictly on water content.

6. Anchor bolt holes must be grouted and sufficiently set before the major portion of the grout is placed.

7. Shade the foundation from sunlight 24 hours before and 24 hours after grouting.

Forming

1. Forms should be liquid tight and nonabsorbent. Seal forms with putty, sealant, caulk, polyurethane foam.

2. Moderately sized equipment should utilize a head form sloped at 45 degrees to enhance the grout placement. A moveable head box may provide additional head at minimum cost.

3. Side and end forms should be a minimum 1" (25 mm) distant horizontally from the object grouted to permit expulsion of air and any remaining saturation water as the grout is placed.

4. Leave a minimum of 2" between the bearing plate and the form to allow for ease of placement.

5. Use sufficient bracing to prevent the grout from leaking or moving.

6. Eliminate large, nonsupported grout areas wherever possible.

7. Extend forms a minimum of 1" (25 mm) higher than the bottom of the equipment being grouted.

8. Expansion joints may be necessary for both indoor and outdoor installation. Consult your local BASF field representative for suggestions and recommendations.

Temperature

1. For precision grouting, store and mix grout to produce the desired mixed-grout temperature. If bagged material is hot, use cold water, and if bagged material is cold, use warm water to achieve a mixed-product temperature as close to 70° F (21°C) as possible.

Recommended Temperature Guidelines for Precision Grouting

	MINIMUM °F (°C)	PREFERRED °F (°C)	MAXIMUM °F (°C)
Foundation and plates	45 (7)	50 – 80 (10 – 27)	90 (32)
Mixing water	45 (7)	50 – 80 (10 – 27)	90 (32)
Grout at mixed and placed temp	45 (7)	50 – 80 (10 – 27)	90 (32)

2. If temperature extremes are anticipated or special placement procedures are planned, contact your local BASF representative for assistance.

3. When grouting at minimum temperatures, see that the foundation, plate, and grout temperatures do not fall below 40° F (7° C) until after final set. Protect the grout from freezing (32° F or 0° C) until it has attained a compressive strength of 3,000 psi (21 MPa).

Mixing

1. Place estimated water (use potable water only) into the mixer, then slowly add the grout. For a fluid consistency, start with 9 lbs (4 kg) (1.1 gallon [4.2L]) per 55 lb bag.

2. The water demand will depend on mixing efficiency, material, and ambient-temperature conditions. Adjust the water to achieve the desired flow. Recommended flow is 25 – 30 seconds using the ASTM C 939 Flow-Cone Method. Use the minimum amount of water required to achieve the necessary placement consistency.

3. Moderately sized batches of grout are best mixed in one or more clean mortar mixers. For large batches, use ready-mix trucks and 3,300 lb (1,500 kg) bags for maximum efficiency and economy.

4. Mix grout a minimum of 5 minutes after all material and water is in the mixer. Use mechanical mixer only.

5. Do not mix more grout than can be placed in approximately 30 minutes.

6. Transport by wheelbarrow or buckets or pump to the equipment being grouted. Minimize the transporting distance.

7. Do not retemper grout by adding water and remixing after it stiffens.

8. DO NOT VIBRATE GROUT TO FACILITATE PLACEMENT.

9. For aggregate extension guidelines, refer to Appendix MB-10: Guide to Cementitious Grouting.

Figure 49c. BASF MasterFlow 928 cement-based mineral-aggregate grout technical data sheet (example acceptable material) (www.basf-cc.com.jo/en/products/Grouting/masterflow928uw/Pages/default.aspx) (continued).

Application

1. Always place grout from only one side of the equipment to prevent air or water entrapment beneath the equipment. Place Masterflow® 928 in a continuous pour. Discard grout that becomes unworkable. Make sure that the material fills the entire space being grouted and that it remains in contact with plate throughout the grouting process.

2. Immediately after placement, trim the surfaces with a trowel and cover the exposed grout with clean wet rags (not burlap). Keep rags moist until grout surface is ready for finishing or until final set.

3. The grout should offer stiff resistance to penetration with a pointed mason's trowel before the grout forms are removed or excessive grout is cut back. After removing the damp rags, immediately coat with a recommended curing compound com-pliant with ASTM C 309 or preferably ASTM C 1315.

4. Do not vibrate grout. Use steel straps inserted under the plate to help move the grout.

5. Consult your BASF representative before placing lifts more than 6" (152 mm) in depth.

Curing

Cure all exposed grout with an approved membrane curing compound compliant with ASTM C 309 or preferably ASTM C 1315. Apply curing compound immediately after the wet rags are removed to minimize potential moisture loss.

For Best Performance

- For guidelines on specific anchor-bolt applications, contact BASF Technical Service.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by BASF Technical Service.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Hold a pre-job conference with your local representative to plan the installation. Hold conferences as early as possible before the installation of equipment, sole plates, or rail mounts. Conferences are important for applying the recommendations in this product data sheet to a given project, and they help ensure a placement of highest quality and lowest cost.
- The ambient and initial temperature of the grout should be in the range of 45 to 90° F (7 to 32° C) for both mixing and placing. Ideally the amount of mixing water used should be that which is necessary to achieve a 25 – 30 second flow according to ASTM C 939 (CRD C 611). For placement outside of the 45 to 90° F (7 to 32° C) range, contact your local BASF representative.
- For pours greater than 6" (152 mm) deep, consult your local BASF representative for special precautions and installation procedures.
- Use Embeco® 885 grout for dynamic load-bearing support and similar application conditions as Masterflow® 928.
- Use Masterflow® 816, Masterflow® 1205, or Masterflow® 1341 post-tensioning cable grouts when the grout will be in contact with steel stressed over 80,000 psi (552 MPa).
- Masterflow® 928 is not intended for use as a floor topping or in large areas with exposed shoulders around baseplates. Where grout has exposed shoulders, occasional hairline cracks may occur. Cracks may also occur near sharp corners of the baseplate and at anchor bolts. These superficial cracks are usually caused by temperature and moisture changes that affect the grout at exposed shoulders at a faster rate than the grout beneath the baseplate. They do not affect the structural, nonshrink, or vertical support provided by the grout if the foundation-preparation, placing, and curing procedures are properly carried out.
- The minimum placement depth is 1" (25 mm).
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

MASTERFLOW® 928

WARNING!

Masterflow® 928 contains silica, crystalline quartz; portland cement; limestone; calcium oxide; gypsum; silica, amorphous; magnesium oxide.

Risks

Product is alkaline on contact with water and may cause injury to skin or eyes. Ingestion or inhalation of dust may cause irritation. Contains small amount of free respirable quartz which has been listed as a suspected human carcinogen by NTP and IARC. Repeated or prolonged overexposure to free respirable quartz may cause silicosis or other serious and delayed lung injury.

Precautions

Avoid contact with skin, eyes and clothing. Prevent inhalation of dust. Wash thoroughly after handling. Keep container closed when not in use. DO NOT take internally. Use only with adequate ventilation. Use impervious gloves, eye protection and if the TLV is exceeded or used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable Federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs or if swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

Waste Disposal Method

This product when discarded or disposed of, is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains material listed by the State of California as known to cause cancer, birth defects or other reproductive harm.

VOC Content

0 g/L or 0 lbs/gal less water and exempt solvents.

For medical emergencies only, call ChemTrec (1-800-424-9300).

BASF Construction Chemicals, LLC – Building Systems
889 Valley Park Drive
Shakopee, MN, 55379
www.BuildingSystems.BASF.com
Customer Service 800-433-9517
Technical Service 800-243-6739

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Figure 49d. BASF MasterFlow 928 cement-based mineral-aggregate grout application data sheet continued (example acceptable material) (www.basf-cc.com.jo/en/products/Grouting/masterflow928uw/Pages/default.aspx) (concluded).

The 13.5-ft-high by 12-in.-wide supplemental wall armor plate for monolith L-1, and the 22-in.-high by 12-in.-wide wall armor plate for monolith L-2, shall be installed in the same manner as described for the 16-in.-high by 12-in.-wide supplemental wall armor plates, with the exception that their top edges shall form a butt joint with the corner protection armor on top of

the guide wall. The corner protection armor shall NOT be notched to accommodate the supplemental wall armor.

Repair monitoring

An example of the severity of the damage to the concrete at the lower land-wall approach vertical joint that required repair is shown in the close-up photograph of Figure 50. The contractor was directed to saw-cut and chisel to remove damaged concrete to a depth of at least 2 in. or to sound concrete. The cavity shown in Figure 50 is approximately 12 in. deep. The lateral extent of the concrete removal would be only 12 in. on each side of this vertical joint, as contrasted with 36 in. and 48 in. on either side of the upper land-wall approach vertical joint. To accommodate the repair wall armor steel plates, the contractor would be required to cut 12.5-in.-long by 2-in.-high notches in the existing wall armor rub strips. The repair steel plates would be fully welded to the existing wall armor rub strips, as well as attached to the concrete walls by anchor rods. Figure 51 shows template markings for removal of the damaged concrete prior to the installation of the repair steel plates.

Figure 50. Closeup photo of major concrete damage at the vertical joint to be repaired.

Figure 51. Template lines for identifying changes to damaged area during monitoring.

A closeup view of a section between two armor wall rub strips is shown in Figure 52 after the damaged concrete was saw-cut and chiseled down to solid concrete. The top and bottom of the armor wall rub strips had been removed before this photograph to allow for installation and welding of the repair steel plates to the rub strips. Next, four anchor rods would be installed for each repair steel plate, as per manufacturer's recommendations in Figures 37a through 37e (specification sheets for an example acceptable material).

Four holes were previously drilled in each of the repair steel plates, each hole being located approximately 3 in. from each edge of the four corners of the plates. Anchor rod holes were drilled into the concrete cavity to match the hole locations in the steel plates, following manufacturer's recommendations for hole size, depth, and preparation. The anchor bolts were installed, and the epoxy was allowed to cure. Next, the contractor sealed the monolith joint with polyurethane closed-cell marine foam. Then, a grout mixture was spread to fill the cavity void and form a base for the backside of the repair steel plates. Specification sheets for an example acceptable mineral-aggregate grout mixture are shown in Figures 49a thru 49d.

Figure 52. Cavity prior to installation of steel plate anchor rods.

The repair steel plates were placed onto the anchor bolts by utilizing threaded nuts on both sides to adjust the plates flush with the top surface of the existing wall armor rub strips. The steel plates would be full welded along the horizontal joints between the plates and the rub strips, and the outer anchor rod nuts would be removed. This would allow the anchor rods to be welded solid to the steel plates and ground flush with the face of the plates. Figure 53 shows four of the repair steel plates being attached to the wall armor rub strips, with the outer rod nuts removed but prior to completion of the welding. Figure 54 shows a section of the vertical joint repair with the lower repair steel plates fully welded to the wall armor rub strips and the anchor bolts welded and ground down to the plates. Figure 55 is a wider view of the repair activities, showing the vertical extent from the pool water line to the top of the lock wall.

The top 13.5 ft of monolith L-1 on the downstream side of the vertical joint undergoing repair did not have wall armor rub strips. Hence, the repair steel plate for this area was not 16-in.-high by 12-in.-wide as were those placed below. This section of repair steel plate required dimensions of 13.5-ft-high by 12-in.-wide. The anchor bolts embedded into the concrete monolith were spaced appropriately for this dimension, as were the holes drilled into the plate. Figures 56 and 57 are photos of the vertical joint showing both sides of the finished repair.

Figure 53. Repair steel plates being welded to the wall armor rub strips, prior to welding the anchor bolts to the steel plates.

Figure 54. Repair steel plates fully welded to the wall armor rub strips, and the anchor bolts welded and ground down to the plates.

Figure 55. 600-ft chamber lower land-wall approach vertical joint during repair activities.

Figure 56. Closeup photo of the 13.5-ft-high by 12-in.-wide top repair steel plate of monolith L-1 on the downstream side of the joint and the 28-in.-high by 12-in.-wide top plate of monolith L-2 on the upstream side of this joint.

Figure 57. Completed repair of 600-ft chamber lower land-wall approach vertical joint at high water elevation on the Ohio River.

5 Monitoring Repair of 1,200-ft Chamber Upper-River Approach Wall

Technical assistance was provided to CELRL and ERDC by McDonald Consulting (McDonald 2008a) in the development and demonstration monitoring of an innovative strategy for expedient repair of localized concrete deterioration or damage to lock walls at John T. Myers Locks and Dam. A primary requirement of this repair monitoring was that all work must be done without adversely affecting navigation operations on the Ohio River. Consequently, work had to be accomplished between tows with a maximum period of 3 hr to 4 hr for actual placement and curing of the repair material.

Development of repair technique

The technical assistance by McDonald (2008a) included (a) analysis of available information to determine probable cause and extent of concrete deterioration, (b) laboratory investigation to demonstrate that available rapid-setting materials could be modified, if necessary, to extend working times without compromising high-early strengths, (c) recommendations for concrete surface preparation and repair application procedures, (d) assistance in preparation for repair, including an onsite inspection of repair preparation and monitoring, and (e) a letter report summarizing all activities.

The area of concrete damage selected by CELRL for repair demonstration monitoring was located in lock monolith R-73, upper-river approach wall to the 1,200-ft chamber (Figure 58). The surface area of the damage was about 1.5-ft by 6-ft with an apparent maximum depth of almost 2 ft. Localized concrete honeycomb is the likely cause of this damage. Dropping fresh concrete an excessive distance onto the hardened preceding lift and inadequately consolidating during construction are possible causes for honeycomb. Subsequent cycles of freezing and thawing, combined with barge impact and abrasion, often contribute to removal of the typically weak concrete in such areas. Removal of damaged concrete and replacement with a quality repair material were recommended.

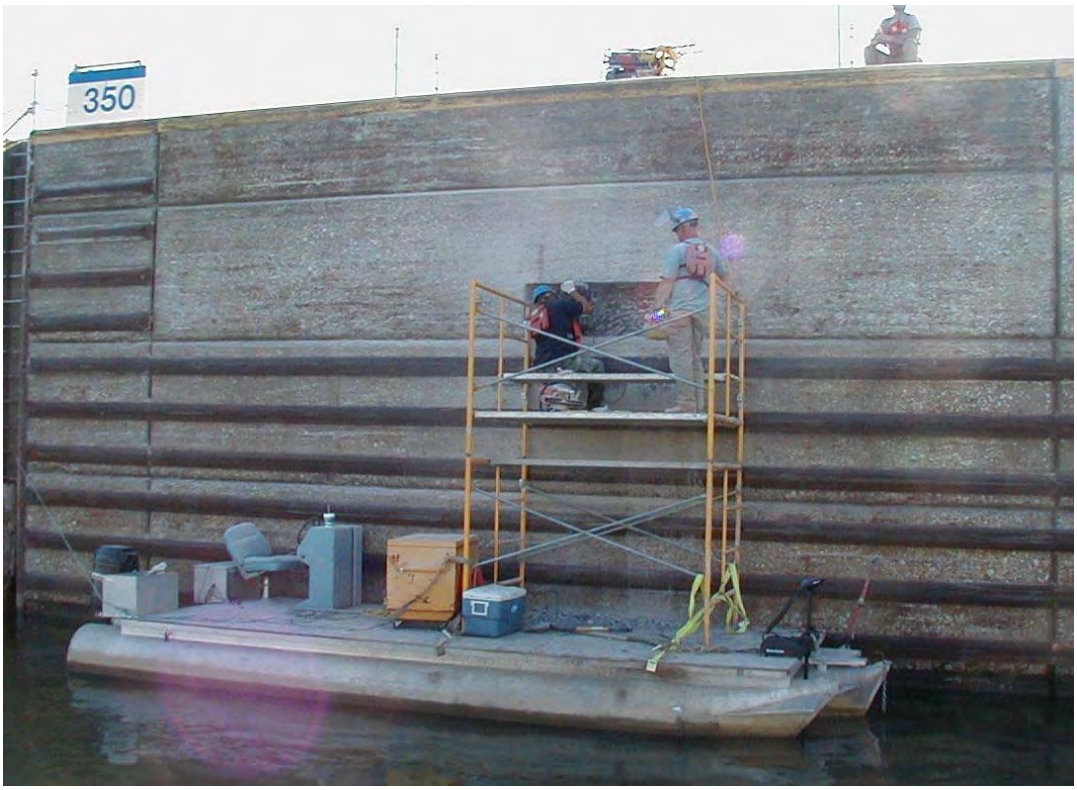

Figure 58. John T. Myers 1,200-ft chamber upper-river approach wall monolith R-73 repair demonstration monitoring.

An order for supplies and services was issued to TJC Engineering, Inc., Louisville, KY on 03 September 2008. The order requested TJC Engineering, Inc., to provide all labor, materials, and equipment for project IDIQ 08-42, John T. Myers 1,200-ft chamber upper-river approach wall monolith R-73 repair in accordance with Scope of Work dated 13 June 2008, as per the TJC Engineering, Inc. proposal dated 28 August 2008.

Scope of work[1]

The contractor shall furnish, fabricate, and install repairs to damaged concrete in monolith R-73 with rapid-setting, high-early-strength concrete applied with the form-and-pour method described in American Concrete Institute (2003), Rapid Application Procedure 4, "Field Guide to Concrete Repair Application Procedures: Surface Repair Using Form-and-Pour Techniques."

[1] This section is extracted essentially verbatim from McDonald (2008a).

Repair application technique

The form-and-pour method is a technique for replacing damaged/ deteriorated concrete by filling a cavity between the formwork and the prepared substrate with a repair material and consolidating it by vibration or rodding (Figure 59). The form-and-pour technique is generally recommended for vertical surfaces such as walls, columns, and other combinations such as beam sides and bottoms.

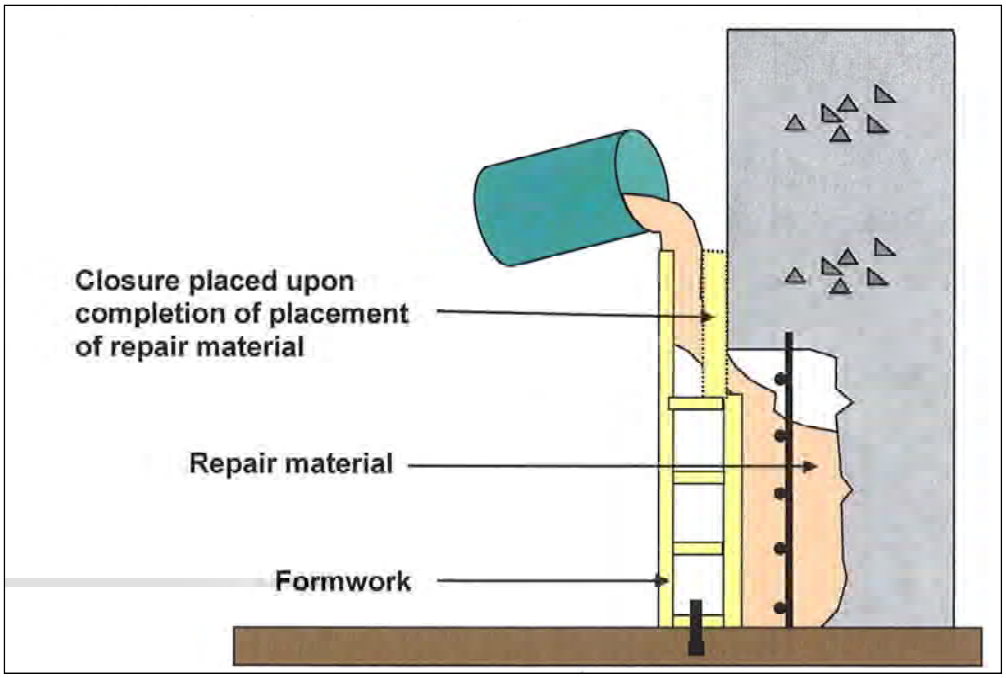

Figure 59. Form-and-pour concrete repair method.

Concrete surface preparation

The importance of surface preparation cannot be overstated. Proper attention to surface preparation is essential for a durable repair. Regardless of the cost, complexity, and quality of the repair material and application method selected, the care with which deteriorated concrete is removed and the concrete substrate prepared will often determine whether a repair project will be successful.

Concrete surface preparation for repair is the process by which sound, clean, and suitably roughened surfaces are produced on concrete substrates. This process includes removing unsound concrete and bond-inhibiting foreign materials from the concrete surface, opening the concrete pore structure, and verifying concrete surface strength if necessary.

Simple layouts that approximate squares or rectangles should be used for repair geometries. Layouts shall be designed to reduce boundary edge length and eliminate acute angles. Excessive or complex edge conditions are usually produced when attempting to closely follow the shape of the deteriorated concrete. Such edge conditions often result in shrinkage stress concentrations and cracking within the repair.

Concrete removal

Saw cuts at right angles to the concrete surface shall be made along the perimeter of repairs that involve concrete removal. The surface area of the monolith R-73 repair is approximately 1.5-ft by 6-ft as shown in Figure 60. Depending on the depth of section loss, it may be desirable to extend the repair area to the right by approximately 2 ft. The perimeter of the repair shall be saw cut perpendicular to the chamber face to a minimum depth of 2 in., except for the lower boundary, which is an existing lift joint.

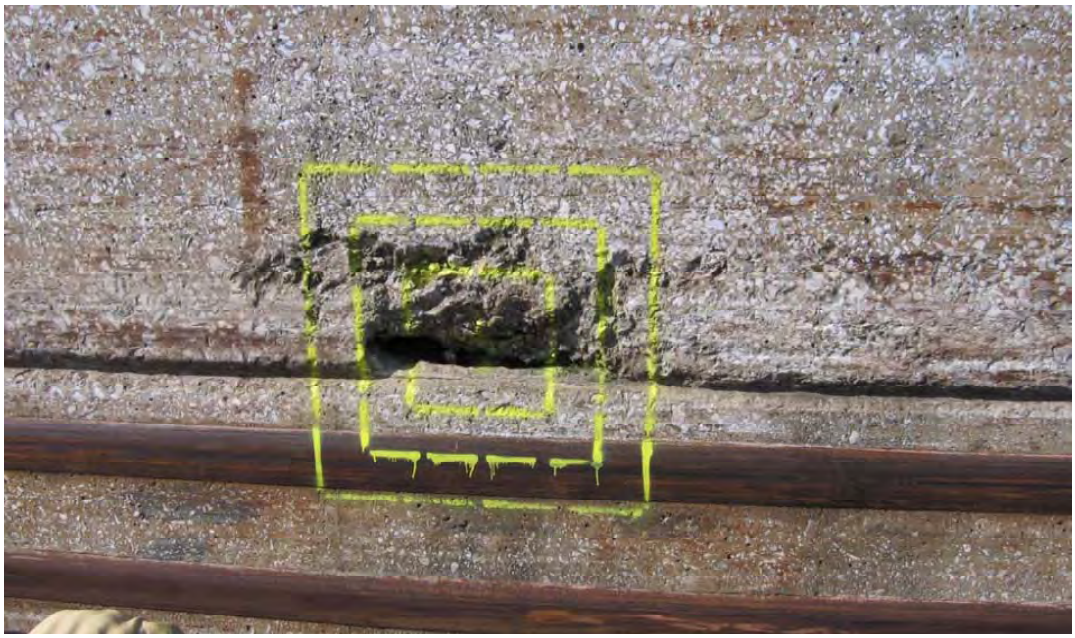

Figure 60. Approximate repair area.

Concrete removal methods are generally categorized as blasting, cutting, impacting, milling, presplitting, and abrading. Detailed information on each technique is provided in the Corps' Engineer Manual 1110-2-2002 (1995) and ACI 546R (2004). Methods used to remove deteriorated and/or damaged concrete and prepare the substrate to receive the repair material shall not weaken or crack the surrounding sound concrete. Concrete shall be removed in such a manner that does not result in microfractures (bruising)

within the remaining near-surface concrete, which would significantly reduce the bond between the repair and substrate.

Existing concrete within the repair area shall be removed to sound concrete with a 4 in. minimum depth of removal (Figure 61). Continue to remove concrete until aggregate particles are being broken rather than simply removed. Concrete shall be removed to provide a gradual transition from the 4 in. minimum depth to the zone of maximum removal. The upper edge of the concrete surface shall be trimmed to eliminate potential entrapped air pockets that would prevent complete filling of the repair cavity. Proper concrete removal is a primary requirement for the outer two-thirds of the repair cavity. Removing concrete from the back of the confined space is difficult, and removal in this area could be limited to loose material.

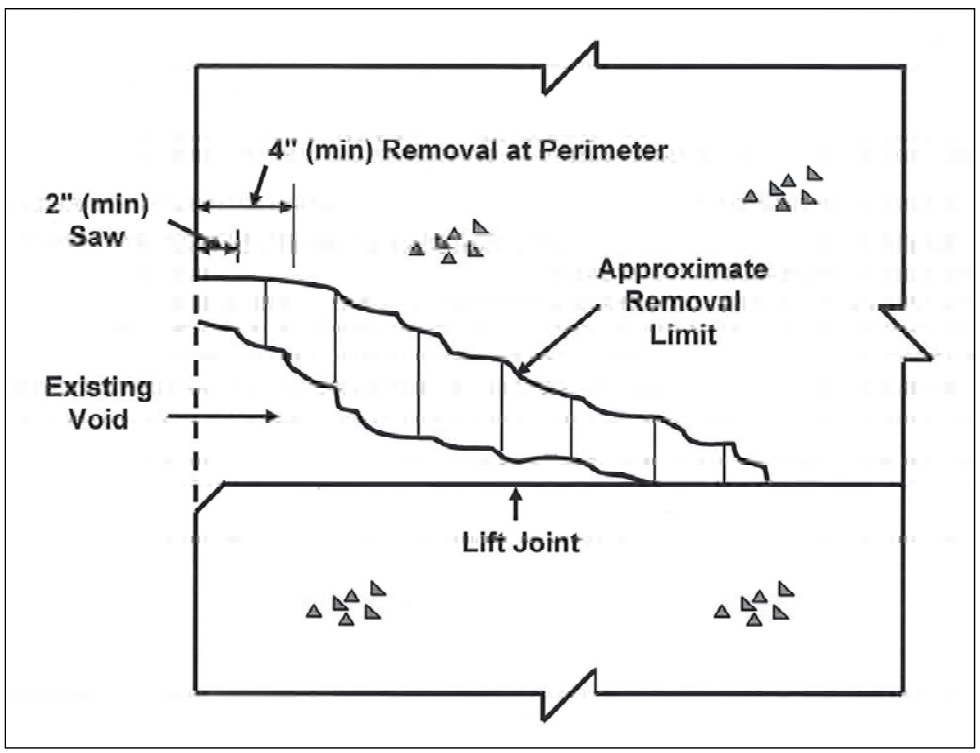

Figure 61. Transverse section through existing void.

Impacting methods are the most commonly used concrete removal techniques. They generally employ the repeated striking of a concrete surface with a high energy tool or a large mass to fracture and spall the concrete. The use of these methods in partial depth removal may produce microcracking in the surface of the remaining concrete. Extensive micro-cracking may produce a weakened plane below the bond line. Hand-held breakers are available in various sizes with different levels of energy and

efficiency. If hand-held breakers (chipping hammers) are used, the maximum size for initial removal shall not exceed the 30-lb class, and the angle of impact with the concrete surface shall not exceed 45 deg. Hammer size for removal of the last 1 in. of concrete and for surface preparation shall not exceed the 15-lb class to reduce the potential for bruising of the substrate.

It is essential to monitor and evaluate the removal operations to limit the extent of damage to the concrete that remains. Near surface evaluation is usually accomplished by microscopic examination or bond testing.

Surface cleaning

Final concrete surface preparation steps must be taken after the removal of damaged concrete. Microcracking and closed surface pores are common when impact tools are used for concrete removal. In this case, abrasive blasting or water-blasting is necessary to open and clean surface pores and remove near-surface microfractures. The existing lift joint shall also be roughened and cleaned to enhance bonding in this area.

Abrading methods remove thin layers of surface concrete by propelling an abrasive medium at high velocity against the concrete surface with abrasive blasters or high-pressure water blasters. Abrasive blasting pneumatically projects abrasives at the concrete surface in the open atmosphere and is the most commonly used method of cleaning concrete.

High-pressure (1,500 psi to 5,000 psi) water blasting with abrasives eliminates the airborne particles that occur when using normal abrasive blasting procedures. Abrasive water blasting results in a clean concrete surface free of dust.

Maintenance of prepared area

After the repair cavity is prepared, it shall be maintained in a clean condition and protected from damage until the repair material can be placed. The contractor shall be required to repeat preparation if a prepared area is allowed to become damaged or contaminated.

In a hot environment, shade shall be provided if practically possible to keep the substrate cool, thereby reducing the effect of high temperature on

working time of the repair material. The material supplier's instructions for conditioning the existing concrete surface shall be followed.

Anchorage

Mechanical connections between repair and existing concrete are required. Approximate anchor locations and orientations are shown in Figure 62. The actual number and locations of anchors will be determined in the field depending on size and shape of repair cavity after concrete removal. Anchor sizes and embedment lengths shall be in accordance with supplier's recommendations. Anchors shall have a minimum cover of 2 in. over the headed ends.

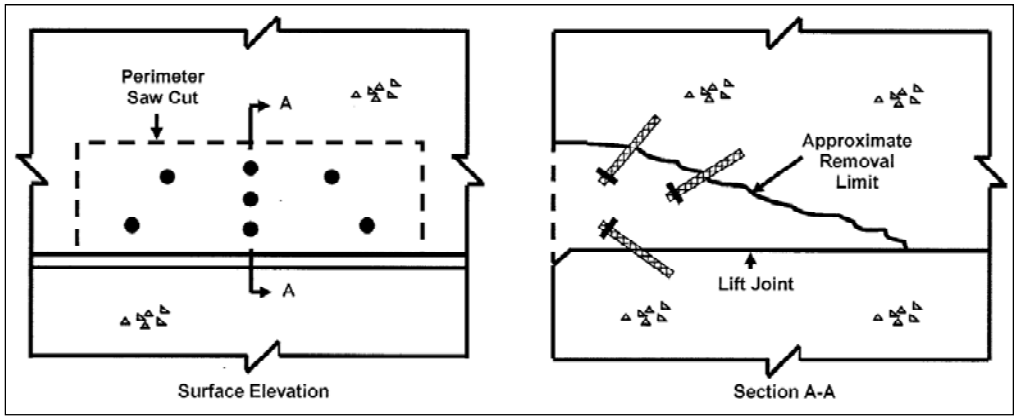

Figure 62. Approximate anchor locations.

Either bonded or expansion anchors installed in holes drilled into the concrete substrate are acceptable. Bonded anchors are headed or headless bolts, threaded rods, or deformed reinforcing bars embedded with portland cement grout or polymer materials such as polyesters, vinyl esters, or epoxies. Expansion anchors are designed to be inserted into predrilled holes and then expanded by tightening a nut, hammering the anchor, or expanding into an undercut in the concrete.

Formwork

Formwork design should follow standard practice for cast-in-place concrete construction. The completed repair surface shall not extend beyond the vertical and horizontal planes of the existing concrete surrounding the repair; therefore, formwork shall be attached securely to the existing concrete surface with anchors that will prevent slippage and form movement during repair placement and consolidation. If expansion/ rock anchors are used, ensure that the anchors are set firmly in place to prevent

slippage under load. A preformed chamfer strip shall be installed within the formwork so that the lift joint in the repair area will match that in the adjacent concrete. Also, a 0.25-in.-thick insert, shaped to match the exposed face of the repair, shall be mounted inside the formwork to ensure that the repair surface does not extend beyond the perimeter concrete.

Placement openings or chutes are required to place the repair material and allow for insertion of a vibrator for internal consolidation of the repair material (Figure 63). Chutes must be constructed to allow overfilling of the repair cavity prior to consolidation. This is necessary to ensure that the upper edges of the repair cavity are completely filled after consolidation. Forms shall be designed to permit rapid removal without hammering or prying against the repair.

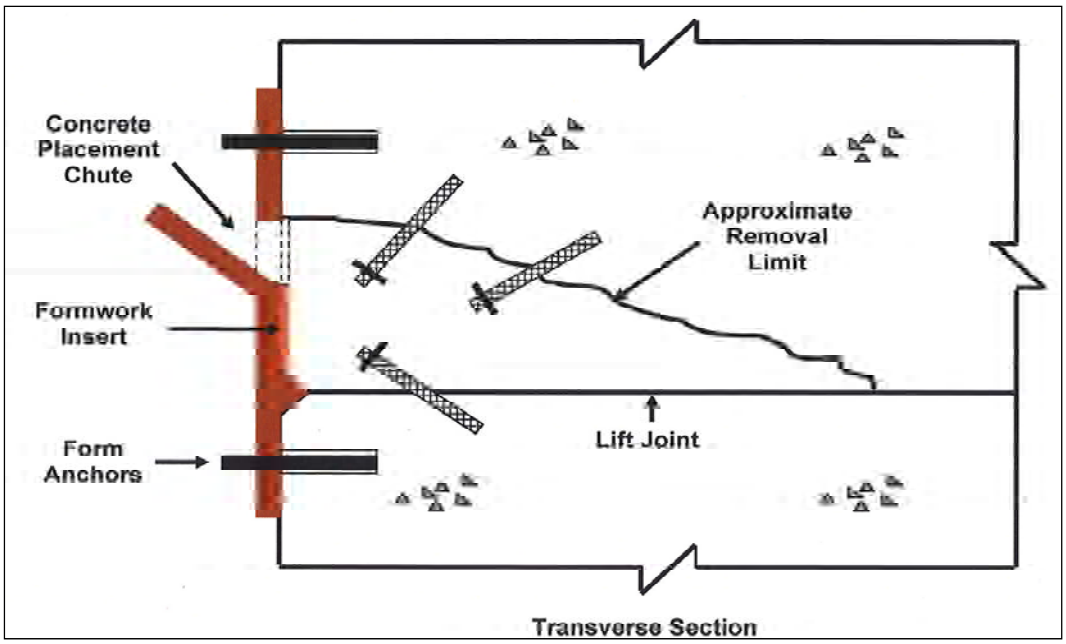

Figure 63. Section through repair cavity showing formwork and placement chute.

Repair material

A rapid-hardening, high-early-strength material will be specified, depending on results of current laboratory tests. Two materials were evaluated (Rapid Set Concrete Mix and Rapid Set DOT Cement). Both are supplied by CTS Cement Manufacturing Corporation, Cypress, CA.

According to CTS, Rapid Set Concrete Mix is a prepackaged blend of rapid-set hydraulic cement and quality aggregates (3/8-in. mechanical sieve analysis) that, when mixed with water, produces a workable concrete material that is ideal where fast strength gain, high durability, and low

shrinkage are desired. The material can be applied in thicknesses from 2 in. to 24 in., sets in 15 min, is ready for traffic in 1 hr, and is durable in wet environments. One 60-lb. bag of Concrete Mix will yield approximately 0.5 cu ft.

According to CTS, Rapid Set DOT Cement is a calcium sulfoaluminate-based hydraulic cement with exceptional workability characteristics. An air entrainment admixture is blended with the cement for freeze-thaw durability. When mixed with aggregates (3/4-in. mechanical sieve analysis) and water, a very fast-setting, concrete repair material is produced that is ideal for the repair of pavement, bridge decks, industrial floors, parking garage decks, freezer floors, and vertical and overhead repairs. DOT Cement may be batched to yield about 0.9 cu ft per 55-lb bag.

Limited tests on these two materials were conducted by ERDC in 2006. Selected results are summarized in the following figures and, in some cases, compared with data from CTS. The Rapid Set Concrete Mix without retarder had an estimated set time of 15 min with an average compressive strength of over 6,000 psi at 2 hr (Figure 64). Results were consistent with data from CTS. The addition of one packet of retarder per sack of material increased initial and final set to 95 min and 100 min, respectively. The average compressive strength was reduced to approximately 4,000 psi at 2 hr. A larger dosage of retarder (6 packets per sack of material) plus an additional gallon of water was used in a mixture batched and mixed at 95° F. Two specimens were broken during demolding for 2-hr strength tests. The third specimen of that set was tested at 5 hr and exhibited a compressive strength slightly over 4,000 psi. The average compressive strength at 6 hr was 4,570 psi. The 28-day strength of the retarded mixture was 7,640 psi at the elevated temperature, compared to 9,160 psi for the retarded mixture at lab temperature.

A concrete mixture was proportioned with Rapid Set DOT Cement, 0.39 water-to-cement ratio, and No. 57 (0.5 in. to 1.0 in.) limestone aggregate. Under laboratory ambient conditions, slump of the concrete with a retarder was 4 in. with initial and final sets of 60 min and 80 min, respectively. Concrete from a second batch, produced under similar conditions about an hour later, exhibited initial and final sets of 62 min and 73 min, respectively. Early-age compressive strengths were also consistent between the two rounds (Figure 65) and similar to data from CTS. Compressive strengths at 28 days averaged 10,790 psi and 11,180 psi for rounds 1 and 2, respectively.

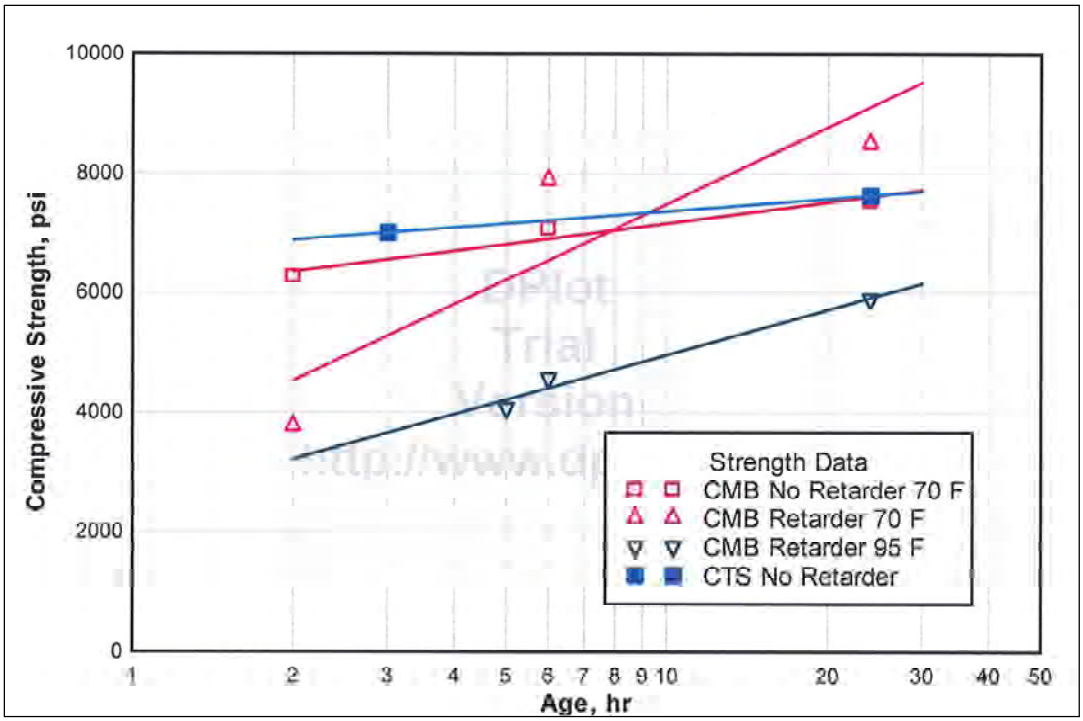

Figure 64. Effect of retarder and ambient temperature on compressive strength of Rapid Set Concrete Mix.

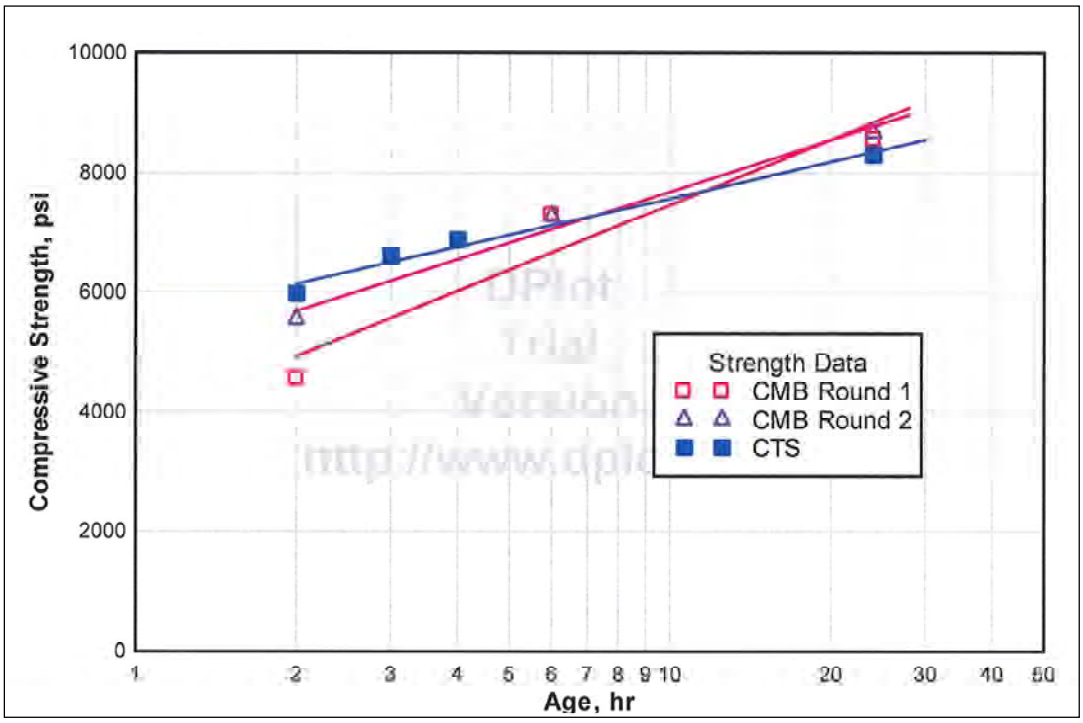

Figure 65. Compressive strengths of Rapid Set DOT Cement mixtures under laboratory ambient conditions.

A “heavily retarded” mixture was batched and mixed in an elevated temperature (nominal 90° F) room. Initial and final set times were 128 min and 160 min, respectively. Early-age compressive strengths were similar for the two rounds (Figure 66) with strengths approaching 2,000 psi at 3.5 hr and 4 hr. The 28-day strengths for the two rounds averaged 7,870 psi and 8,440 psi.

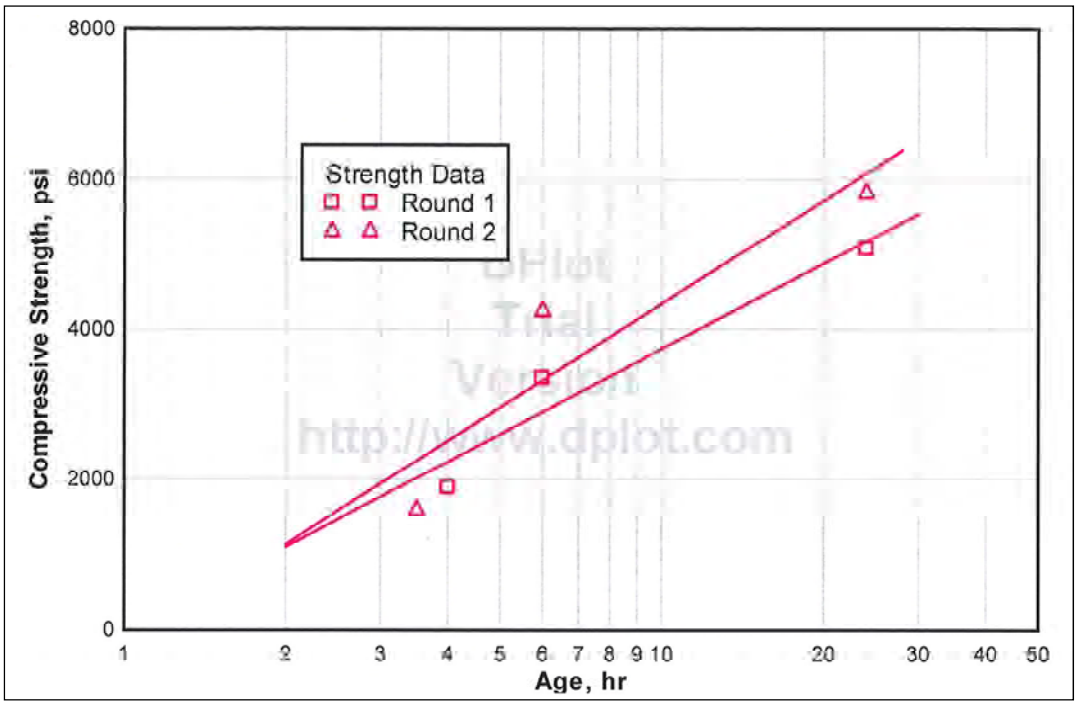

Figure 66. Compressive strengths of Rapid Set DOT Cement mixture under elevated temperature conditions.

Based on the results of these tests, it is possible to develop a repair material that will satisfy the criteria necessary for a successful repair at John T. Myers Lock. These criteria include a mixture with good workability, a 1-hr (minimum) initial time of set, and a minimum 3-hr compressive strength of 3,000 psi. Elevated temperatures similar to anticipated ambient conditions at the project site were emphasized during the lab investigation.

Preconstruction activities

Prior to proceeding with the repair, a preconstruction meeting is recommended. The meeting should include representatives from all participating parties (owner, engineer, contractor, laboratory, materials manufacturer, etc.) and specifically address the parameters, means, methods, and materials necessary to achieve and monitor the repair objectives.

Repair application

Prior to the repair material placement, all preparation, including inspection, shall be completed. This includes moisture conditioning of the substrate concrete to provide saturated-surface dry conditions. The concrete surface shall be allowed to dry following saturation of the concrete. Saturated surfaces will prevent proper bonding because the surface pores are filled with water and are unable to absorb the repair material.

Time is of the essence in batching, mixing, and placing fast-setting repair materials. All personnel and equipment must be in place and organized before mixing of materials begins. (Mixer requirements will be provided based on the type of repair material selected.) The repair material shall be transported from the mixer to the application area as rapidly as practical and by methods that will prevent segregation or loss of ingredients. Placement of the repair material shall follow normal placement practices. Internal vibration is required to consolidate the repair material, to remove the entrapped air, and to bring the repair material into intimate contact with the existing substrate. Forms should remain in place for a minimum of 2 hr after completion of concrete placement. Curing will depend on the repair mixture selected.

Inspection

Slump and entrained air content tests shall be performed onsite in accordance with the appropriate ASTM standards to ensure the repair material quality. A minimum of 12 concrete cylinders for compressive strength tests shall be prepared and tested in accordance with ASTM (1996) C39, and ASTM (1998) C31. Cylinder size and testing age are dependent on the repair mixture selected. Initial tests at 3-hr age will require close coordination with the testing lab or possible onsite testing.

Any surface discontinuities around the repair perimeter shall be removed by grinding to provide a seamless transition between the repair and the surrounding concrete. Also, any excess repair material at the placement chutes shall be removed by non-impact methods such as sawing or grinding.

The complete repair area should be inspected by hammer sounding or evaluated by other non-destructive methods to determine overall integrity. Hollow sounds are indications of voids or delaminations that shall be marked and recorded.

Upper-river approach wall repair monitoring[1]

Given the location, size, and orientation of the damage shown in Figure 60, the form-and-pour procedure was recommended for application of a rapid-setting, high-early-strength concrete repair material. A typical application of the form-and-pour procedure is shown in Figure 67.

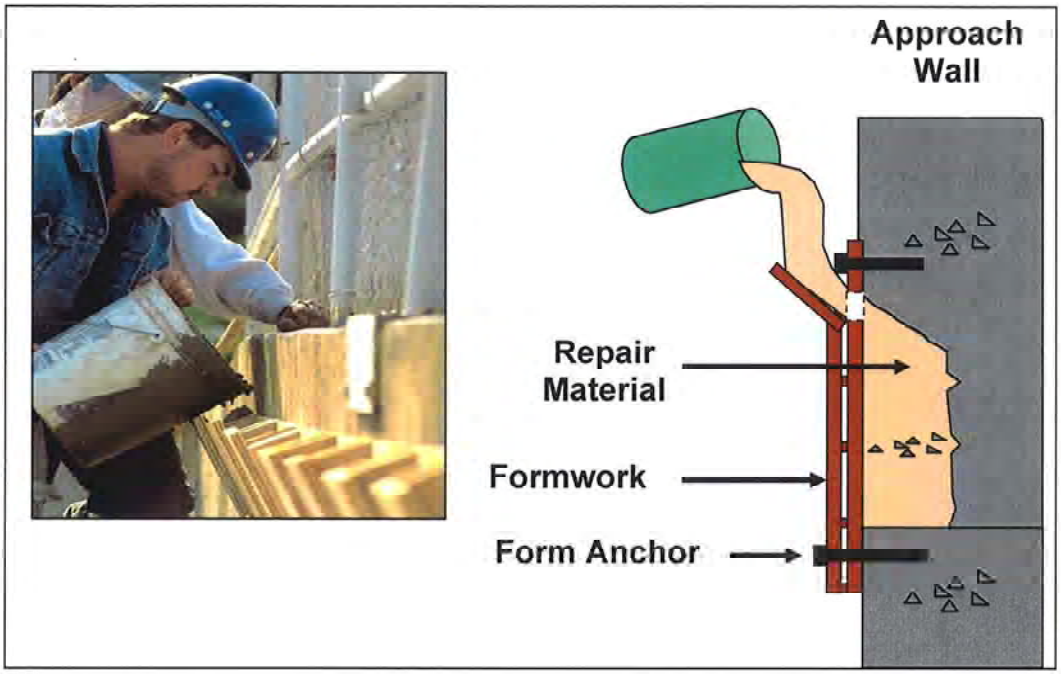

Figure 67. Example of form-and-pour method.

Repair material

To minimize disruptions to operations during repair, a high-early-strength repair material was required. Typically, such materials are also rapid setting with limited working times. The objective of the laboratory evaluation was to demonstrate that available rapid-setting materials could be modified, if necessary, to extend working times without compromising high-early-strengths. The goals of this investigation were to identify a repair material with (a) good workability, (b) approximately 1-hr working time, and (c) minimum 3-hr compressive strength of 3,000 psi. The repair was originally expected to be conducted during the late summer; therefore, effects of elevated temperature similar to anticipated ambient conditions at the project site were also considered during the laboratory investigation.

[1] This section is extracted essentially verbatim from McDonald (2008b).

Repair activities

Two tiers of scaffolding mechanically attached to the deck of a 20-ft-long by 8-ft-wide self-propelled floating work platform were used for access to the repair area (Figure 68). This system allowed contractor employees to easily access the repair area during periods between normal lock chamber operations (Figure 69).

Figure 68. Floating platform for access to repair area.

Figure 69. Contractor employees easily accessing work area via floating platform and removing concrete from the repair area.

A hand-held diamond blade saw was used to make a 4-in.-deep cut around the perimeter of the 1.75-ft by 6-ft repair. An electric-powered hand-held impact breaker was used to remove concrete within the repair area (Figure 70). Typically, concrete was removed to a depth of 1 ft to 2 ft, with a small isolated area extending to a depth of about 4 ft.

No. 5 grade 60 reinforcing steel installed in drilled holes was used as anchors. Lengths of individual anchors varied depending on location and orientation within the repair. Typically, anchors had an embedment depth of 6 in. and minimum cover of 3 in. Anchors were embedded in a medium viscosity epoxy. A total of 18 anchors was installed (Figure 71).

Prior to placement of the repair material, concrete surfaces within the repair cavity were cleaned by washing with river water. After excess water was removed, a plywood form was installed and anchored flush with the lock wall. A 0.25-in.-thick insert, shaped to match the exposed face of the repair and mounted inside the formwork, was previously suggested by

Figure 70. Concrete removed from the 1.75-ft by 6-ft repair.

Figure 71. Reinforcing steel rods installed with epoxy in drilled holes as anchors.

ERDC; this insert was indeed required by the contract. The purpose of the insert was to ensure that the repair surface did not extend beyond the plane of the perimeter concrete. Two access chutes were provided in the top of the form for placement of the repair material. Two anchored 2-in. by 2-in. steel angles were installed to support and stiffen the formwork.

The contractor elected to use a rapid-set grout designated as Mix RSG1, previously proportioned in the ERDC concrete laboratory, as the repair material. Mixture proportions (1-bag mix) were as follows.

Material	Quantity	Remarks
Concrete Mix	60.0 lb	1 bag Rapid Set Concrete Mix
Retarder	12.5 g	1/2 pack SET/ bag concrete mix
Water Reducer	60.0 g	1 pack FLOW/ bag concrete mix
Water	8.33 lb	1 gal/bag concrete mix

Mix RSG1, when batched and mixed at laboratory temperature (73 °F), was essentially self-leveling with a slump and a slump flow shortly after mixing of 11 in. and 27 in., respectively. The air content of the plastic concrete was 8.5 percent. The laboratory mixture was cohesive with no obvious signs of segregation with a working time and time of set of 45 min and 75 min, respectively. Average strengths of Mix RSG1 laboratory specimens were 3,740 psi, 4,180 psi, and 4,460 psi at 3 hr, 4 hr, and 6 hr, respectively. The 1-day and 28-day strengths were 4,660 psi and 6,070 psi, respectively.

Ambient air and water temperatures at the project site were estimated to be about 55° F during the repair. Although the lower temperatures would be expected to reduce early-strength gain, the contractor elected not to revise the dosages of retarder and water reducer. Both admixtures were dissolved in a gallon of mix water prior to the addition of a bag of the prepackaged concrete mix. The repair material was batched and mixed on top of the approach wall immediately above the repair area. Each 1-bag batch was mixed individually in a 5-gal bucket with a mixing paddle and an electric drill (Figure 72). Each bucket, containing approximately 0.5 cu ft of material, was then lowered to the floating work platform (Figure 73) and the contents were poured into the formwork (Figure 74). Contractor personnel observed that the repair material was very flowable and essentially self-leveled within the formwork. A spud vibrator was periodically inserted through the chutes to internally consolidate the repair material. Approximately 14 cu ft of material were placed in slightly more than one hour. Placement was completed at approximately 12:30 pm on October 22, 2008.

There were no Quality Assurance/Quality Control requirements in the contract; therefore, specimens were limited to two containers of mixed material, one each from near the beginning and end of mixing, for visual monitoring of material hardening. The formwork was removed approximately 2.5 hr after completion of placement. The repair cavity was completely filled and the exposed surface was uniform and sound. The excess material in the vicinity of the chutes was removed, and a 1-in.-thick sheet of rigid insulation was installed over the repair area to reduce the

Figure 72. Mixing grout on top of approach wall above repair area.

Figure 73. Mixed grout being lowered to the floating platform for placement.

Figure 74. Mixed grout being poured into the formwork.

temperature differential between the interior and the surface of the repair as heat generated during cement hydration was dissipated. The insulation was removed on the morning of the fourth day after placement. An examination of the repair by lock personnel during the afternoon of the following day revealed a vertical hairline crack at the left and right sides of the repair. The widths of these cracks at the interface between the repair and the existing concrete and a similar vertical crack in the middle of the repair were so small that they are not apparent in photographs of the repair taken at the time (Figure 75).

Figure 75. Completed repair of lock approach wall 5 days after placement.

Overall, the exposed surface of the repair has a generally uniform appearance with the possible exception of a small area in the upper left corner of the repair (Figure 76) where what appears to be small surface voids typical of entrapped air are located. Also, the photograph appears to

indicate that the left side of the repair may protrude slightly beyond the adjacent concrete; however, surface measurements determined this is not a real phenomenon but only an optical illusion.

Figure 76. Closeup of completed repair 5 days after placement.

6 Feasibility of Fiber Reinforced Polymer Composites for Inland Hydraulic Structure Application[1]

Fiber Reinforced Polymer (FRP) composites with fibers/fabrics bonded by organic polymers (resin system) are being referred to as the materials of the 21st century because of their many inherent advantages. In the U.S., application of composites in civil infrastructure projects began in the late 1980's, with major advances in repairing bridges and roads, retrofitting structures, and marine applications. In the last decade, significant efforts were made to develop design guidelines, construction and maintenance standards, and specifications for FRP utilization. These efforts revealed end-user willingness to implement these high-performance materials since they have better durability and cost effectiveness over conventional materials at appropriate applications. FRP composites have a number of advantages over traditional materials. However, extensive usage of FRP in civil infrastructure is inhibited because of lack of data on long-term field performance and design and construction specifications in relation to conventional materials.

The Constructed Facilities Center, West Virginia University (CFC-WVU), was commissioned to conduct a feasibility review of FRP materials for inland hydraulic structure application (GangaRao and Vijay 2010). That study dealt with the possibility of design, development, and implementation of FRP composite structural systems that are of interest to USACE, focusing on civil and marine applications. The resulting report discusses FRP constituents and structural shapes and systems (including their field implementation) in marine and inland hydraulic structures. Responses under in-plane and out-of-plane forces under outdoor service conditions are included. Constituents, short- and long-term properties, and influence of fiber orientation on strength, stiffness, and deformation of composite products are described under combined external and environmental loads.

Essential input to this study was obtained from the European perspective regarding (a) types of applications, (b) constituent material guidelines, and (c) design, test, fabrication, inspection, and repair techniques.

[1] Chapter 6 is extracted essentially verbatim from GangaRao and Vijay (2010).

FRP constituents

The study by GangaRao and Vijay (2010) focuses primarily on glass polymer composites.

Glass

Glass is one of the most common fibers used as reinforcement in composites. Fibers can be short, chopped, milled, or in the form of elongated single crystals. Continuous fibers come in the form of untwisted bundles as strands or twisted bundles as yarns and also as a collection of parallel continuous strands (roving).

Resins

Resins are the polymer binders that hold the fibers together and transfer the loads between the fibers, in addition to protecting them from environmental factors and carrying shear loads. Thermoset resins (e.g., polyester and epoxy) transform into matrix binders after curing through an irreversible chemical reaction. By heating, thermoplastic resins are softened from solid state before processing (making a composite) without chemical reactions. Thermoplastics return to solid state (matrix) once processing is completed.

The primary advantage of thermoplastic resins over thermoset resins is their high impact strength and fracture resistance, which is exhibited by their excellent damage tolerance property. Thermoplastic resins also provide higher strains to failure, a property which is manifested by better resistance to micro-cracking in the matrix of a composite. Some of the other advantages of thermoplastic resins include (a) unlimited storage (shelf) life at room temperature, (b) shorter fabrication time, (c) post-formability (e.g., by thermoforming), (d) ease of repair by (plastic) welding, solvent bonding, etc., (e) ease of handling (no tackiness), (f) recyclability, and (g) higher fracture toughness and better delamination resistance under fatigue. The advantages of thermoset resins over thermoplastic resins include (a) better creep and chemical resistance, (b) lower stress relaxation, and (c) better thermal stability.

Fiber surface treatment (sizing)

Sizing is the treatment of fiber surface with coupling agents that couple resin to fibers to protect the fiber against moisture and reactive fluid attacks. Sizing improves wetability of the fiber surface against the matrix

and, therefore, creates a stronger bond between the fiber and the matrix. This is necessary for the effective transfer of stresses between the fibers and the matrix. Functions of sizing for different fibers include (a) improving the interfacial bond of glass fibers with the matrix to protect the glass fibers from environmental attacks that are of main concern in the strength degradation of glass fibers, (b) promoting good chemical bonding with a binder for carbon-fiber surfaces that are chemically inactive (the sizing treatment creates a porous carbon fiber surface, and hence, increases the surface area by creating pitting to provide a large number of fiber-matrix interfacial contact points), and (c) enhancing weak surface adhesion of Aramid fibers.

Additives and curing

Different kinds of additives and modifiers are added to modify the performance of thermoset polymers. Catalysts, promoters, and inhibitors are used to accelerate or slow the rate of polymerization. Release agents are used to facilitate the removal of the composite from the mold. Other agents, such as plasticizers, are used to improve processability or product durability. Fire retardants are used to extinguish fire upon contact. Viscosity control agents help control the flow of the resin, while air-release agents reduce air voids. Toughness agents increase the toughness of fibers. Electrical conductivity agents shield conductivity from certain fibers, and antistatic agents reduce static or electrical charge. Antioxidants (as additives) keep the polymer from experiencing oxidation.

Coating

Coatings are applied to improve FRP performance against abrasion, fire, and environmental attacks and to improve the adhesion to other construction materials. For high-weatherproof performance along with high-abrasion resistance, a paint system with a high-abrasion resistant intermediate coat and a high-weatherproof topcoat should be used.

Applications, performance history, and feasibility

Since California began using composite jacketing for seismic retrofitting of bridge piers in the late 1980s, FRP materials have gained popularity in various seismic repair and retrofitting applications for both bridges and buildings. Other non-seismic repair and retrofit applications have gained

increasing interest in the last decade, and numerous successful applications using FRP composites in construction have been reported.

Reinforcements/plates/wraps/shapes

FRP can be in the form of reinforcing bars in concrete, structural plates and shapes, or wraps for concrete or timber substrates. The wraps are used for bonding materials to structural members such as concrete and wood to enhance their strength, stiffness, ductility, and durability. Several bridge girders and concrete piers were built with concrete decks and slabs reinforced with non-corroding FRP bars. Since rehabilitation of structures typically involves wrapping concrete or wood members with FRP, bonding between the substrate and FRP is of great significance. Structures that require wrapping may be bond-critical (e.g., flexural members such as beams) or contact-critical (e.g., columns requiring confinement). The effectiveness of wrapping concrete members with FRP composites to repair and rehabilitate damaged members after partially removing chemicals from the concrete and re-alkalizing the members is well researched.

Inland and offshore structures

FRP composites are ideally suited as a quick and effective structural repair tool because of their light weight, high strength, and corrosion resistance. Bridge repair using FRP composites is a major success story. The availability of resins that cure under water extended the wrap application to substructure elements, such as partially submerged piles that are damaged. Also, FRP composites have been used in offshore platforms, where corrosion in the presence of seawater is a major concern. Some of the current FRP applications include (a) buoys and floats, (b) primary steel structures, (c) helicopter landing decks, and (d) walls and floors to providing protection against blast and fire.

Hydraulic applications

In 2000, the French Water Authority introduced a new generation hybrid lock gate designed for use on small inland waterway networks (locks under 6.5 m in width with miter gates). The hybrid lock gate is made of FRP material (two surface laminates 5.2-m-wide and up to 8-m-tall with horizontal stiffeners) and strengthened with a stainless steel frame. Its self weight is only 40 percent steel (Composites News 2000). Different types of hybrid FRP gates (12-in. by 12-in.) with steel cores are available

commercially for water flow control applications, including sluice gates, slide gates, stop gates, weir gates, and flap gates. They are capable of holding water pressure heads up to 100 ft.

Marine applications

FRP pipes required for marine applications are designed with high safety factors (6 to 12), depending on the loading and application parameters such as stress levels and stress concentrations, stress (pressure) surges, operating temperatures, water hammer effects, etc. FRP pipes are typically designed on allowable strain basis with due consideration provided to pressure and temperature such that repeated residual strain accumulation and damage are prevented; however, FRP structures are typically designed for deflection limit states. Large FRP pipe systems with diameters up to 13 ft and a minimum 50-year design life are being manufactured for marine infrastructure applications.

Piling

A full-scale feasibility assessment was conducted on different types of FRP composite-bearing piles at Port Elizabeth, NJ. These FRP composites consisted of recycled plastic reinforced by fiberglass rebar (composite marine piles), recycled thermoplastic reinforced by steel bars, and recycled plastic reinforced with randomly distributed fiberglass. Juran and Komornik (2006) concluded that FRP composite piles could be used as an alternative engineering solution for deep foundations. They recommended that further research regarding time-dependent stress response of composite recycled plastic material be conducted before wider use of these piles. One of the main concerns was that FRP piles may undergo high deformations under sustained loads.

Other applications

FRP composite blast gates with damper/duct requirements are made from vinylester resins with corrosion veils and are used in air-duct systems that require direction/volume control with shaft seals. These blast gates are typically suited for highly corrosive situations found in waste-water treatment plants, pulp and paper mills, and chemical plants. FRP beacons are produced for offshore and ground applications. FRP manholes are commercially available in diameters up to 72 in. Aircraft structures made of composites are gaining popularity because of their high strength-to-

weight ratio. The Boeing 787 *Dreamliner* is the world's first major commercial airliner to use composite materials for most of its construction (www.boeing.com).

FRP development by CFC-WVU

FRP design, manufacturing, field implementation, and monitoring work by the CFC-WVU over the last two decades contributed to the development and publication of design documents, specifications, short courses, conferences, and technology transfer activities.

FRP composite properties

Thermal properties

A lower coefficient of thermal expansion of glass fibers in relation to resin produces residual stresses within the material's microstructure during both temperature drop and composite processing at high temperatures. In cold regions, the difference in curing and operating temperatures of the composite material may be as high as 200° F, resulting in residual stresses that are high enough to cause microcracking within the matrix and matrix fiber interfaces. Matrix tensile strength reductions up to 50 percent may be possible because of residual stress buildup under low temperature effects.

Mechanical properties of FRP composites change when the material is exposed to elevated temperatures (37° C to 100° C). Increases in temperature may accelerate time-dependent effects such as creep and stress relaxation. Similarly, evaluation of composite systems at low temperature is essential since high strength and stiffness degradation rates under thermal cycling are observed in cold region structures. The increase in stiffness at low temperature is attributed to crystallization and instantaneous thermal stiffening, which is dependent upon polymer type and temperature. The decrease in temperature can lead to a possible increase in (a) modulus, (b) tensile and flexural strength, (c) fatigue strength and creep resistance, and (d) adhesive strength. Also, decrease in temperature can lead to a possible reduction in (a) elongation and deflection, (b) fracture toughness and impact strength, (c) compressive strength, and (d) thermal coefficient.

Chemical properties

Cured resins (matrix) hold the fibers together for shear transfer. Their chemical resistance against pH, strength, stiffness, and viscoelastic

properties are related to their chemical structure. The presence of moisture can lead to chemical changes, potentially affecting their properties.

FRP composites employed in marine applications are subjected to hydrothermal stresses and moisture-induced chemical and mechanical properties. Water penetrates FRPs through two processes: (a) diffusion through the resin and (b) flow through cracks or flaws. During diffusion, absorbed water is not in the liquid form but consists of molecules or groups of molecules that are linked together by hydrogen bonds to the polymer. The molecules that dissolve in the surface layer of the polymer migrate into the bulk of the material under a concentration gradient. Water penetration into cracks or other flaws occurs by capillary flow. Water also penetrates at the fiber-matrix interface. Moisture pickup leads to a loss of chemical energy, which is attributed to hydrolytic scission of ester groups. However, increased hydrostatic pressure reduces water uptake due to closing of micro cracks.

Mechanical properties

Longitudinal tensile strength estimation is based on the rule of mixture and assumes that once the fibers break the matrix is unable to sustain the load, and the composite fails. It is also assumed that all fibers have the same tensile strength with no debonding from resins.

Longitudinal compressive strength is about one-half the tensile strength. Compression failure is controlled by the buckling of individual fibers, denoted as microbuckling. The microbuckling of fibers is, in turn, controlled by the misalignment of the fibers, shear modulus, and shear strength of the composite. Fiber misalignment in an FRP composite is caused by the microcatenary that is inherent in the fibers coming off a spool during production.

Transverse tensile strength of an FRP (transverse to the fiber direction) is controlled by the strength of the matrix, the strength of the fiber-matrix interface strength, and the defects present in the matrix. Several classical models and empirical formulas are available to predict the transverse tensile strength with a suitable correction factor for the presence of voids. Agarwal and Broutman (1990) used a strength-of-materials approach to determine the transverse strength of an FRP composite by following the assumption that the transverse tensile strength is controlled by the ultimate

strength of the matrix and modified it with either a stress concentration factor or strain magnification factor. A more advanced way of calculating the reduction factor, similar to that of the stress concentration and strain magnification factors, is found from the three-dimensional state of stress in the composite. From this, a suitable failure criterion, such as the distortion energy criterion, can be employed to determine the reduction factor.

Transverse compressive strength can be deduced from classical transverse tensile strength equations. Equations by Nielson (1967) and Chamis (1984) can be adapted for transverse compressive strength by replacing the ultimate tensile strength of the matrix with the ultimate compressive strength. The value of the transverse compressive strength is generally higher than the values of the transverse tensile strength for the matrix. The same is true in the full-scale composites as well. Hence, a designer should carefully consider the fiber/fabric architectures and fiber orientations during design of structures such as miter gates. These gates are subjected to in-plane and out-of-plane loads that lead to force couplings and stress concentrations. Therefore, combined load effects have to be considered properly in a design.

Bending strength is defined as the value of stress per unit length at failure on the plate surface. As described by GangaRao et al. (2001), preliminary design of composite components is based on carpet plots of flexural strength. Due to the strain distribution existing in the cross section, the bending strength of FRPs is greater than the tensile strength. This type of behavior is seen in concrete. Typically, the bending-to-tensile strength ratio is about 1.5. This kind of relationship can be determined by experimental values of both the tensile and bending strengths independently. Some models predict the bending-to-tensile strength ratio for a unidirectional composite material, and a model based on the Weibull distribution was developed by Bullock (1974). Composites made of glass fiber/epoxy exhibited the bending/tensile strength ratios to be on the order of 1.30.

Inplane and interlaminar shear strengths are important for continuous FRPs of orthotropic materials whose physical and mechanical properties depend strongly on the fiber direction and stacking sequence of fibers/fabrics. Because composites are orthotropic, their stress/strain analysis requires knowledge of several elastic constants, which may not be readily available. Composites typically have low shear strength, and shear failure could be dominant under off-axis loading. Based on fiber/fabric

configuration, shear failures could occur within the plane of the composite (in-plane failure) or within the plane of thickness (interlaminar failure). To avoid catastrophic shear-related failures in composite structures such as miter gates, it is necessary to evaluate both in-plane and interlaminar shear failure criteria including proper knock-down factors in the design.

Several test methods are available to evaluate the in-plane and interlaminar shear properties. The shear test method should lead to a uniform shear stress distribution on the shear plane. Depending on the structural shape, in-plane shear strength can be measured by tests such as off-axis tension test (flat plate), Iosipescu test, or torsion test (thin-walled tubes) (Chamis and Sinclair 1977; Adams and Walrath 1987; Foley, Roylance and Houghton 1989). Interlaminar shear strength could be determined by a short-beam test, Iosipescu test, or the double-notch method (Kedward 1972; Walrath and Adams 1983; Dadras and McDowel 1990).

Creep and relaxation of composites occurs when a polymeric material is subjected to a constant load. The deformation continues to increase with time. This phenomenon of increasing strain under constant load is known as “creep.” Conversely, a constant strain imposed on a polymeric material induces stress; but then the stress decreases with increasing time under constant strain, a process known as “stress relaxation.” Both creep and stress relaxations are manifestations of the viscoelastic behavior of polymeric materials. Viscoelasticity arises because polymers are long-chain molecules, and, under stress, parts of a molecule or even an entire chain of molecules can rearrange and slide past each other. This is especially significant when the operating temperatures are above the polymer-glass transition temperature, Tg. However, rearrangement of molecules does take place in the glassy state (i.e., below Tg), albeit at a much slower rate. Furthermore, creep and stress relaxations are more pronounced in thermoplastics than in thermosets. The presence of fillers and reinforcements can further restrict creep. However, even in thermosets, one can observe chain rupture under large deformations.

Fatigue and fracture of FRP materials have been active areas of research during the past 20 years. Unlike homogeneous materials, FRP composites accumulate damage through crack propagation rather than by developing localized damage, and fracture does not always occur by propagation of a single macroscopic crack. The damage accumulation in these materials is microstructural, which includes fiber/matrix debonding, matrix cracking,

delamination, and fiber fracture. Fatigue damage mechanisms in unidirectional composites primarily depend on loading mode (e.g., tensile, compressive, bending, torsion, or combinations) and on the loading direction (i.e., parallel or inclined to the fiber direction). Typically, the damage mechanism in tensile fatigue is in three stages: (a) fiber breakage, (b) matrix cracking, and (c) interfacial shear failure. This is true for compression fatigue, except that there could be buckling failure of fiber.

Durability

In lieu of traditional steel reinforced concrete and steel structures with corrosion problems, glass and carbon FRP composites in the form of bars, fabrics, laminates, plates, and shapes are being increasingly employed in structural applications. FRPs undergo changes in their thermo-mechanical properties with time. These changes are a function of temperature variations, humidity, freeze-thaw variations, pH variations, alternate wetting and drying, exposure to moisture in seawater applications, sustained stresses, and others.

Fiber and resin durability under aging

The degree of damage/deterioration depends on various factors such as the type and volume of fibers and resins, fiber sizing chemistry, severity of the external environmental agents such as pH and temperature values, cure conditions during manufacturing, and quality control issues. Design guidelines and material selection criteria that consider the effects of mechanical and environmental loads must provide confidence in terms of 50-100 year service life of structural materials and systems. The fibers in FRP composites are the main load-carrying elements. The polymer matrix (cured resin) protects the fibers from damage, ensures good alignment of fibers, and allows in-plane force transfer between fibers. Fibers are selected based on the strength, stiffness, and durability requirement for specific applications. Resins are selected based on the function (e.g., wet lay-up vs. factory manufactured) of FRP composite systems. Fiber types typically used in the construction industry are carbon and glass, with thermoset epoxy, vinyl ester, polyester, and urethane resins.

Moisture and temperature response of composites

Moisture uptake in composites leads to matrix softening due to hydrolysis; reduction in matrix dominant properties of a composite, such as shear

strength of composites; lowering of glass transition temperature; and reduction in composite strength and stiffness. Mechanical property reduction is accentuated in the presence of stress and temperature. GangaRao et al. (2001) extensively discuss relevant mathematical models, the effect of aging on degradation mechanisms, and design factors related to moisture and temperature effects.

Effect of acid/salt/alkaline solutions

Acid, salt, and alkaline reactions of FRP composites leading to aging are a major durability factor to be considered in design. Some reports indicate that acids (e.g., hydrochloric, sulfuric, phosphoric, nitric) are more detrimental to carbon FRP than alkaline solutions are. Similarly, depending on cement content and additives, concrete environment can be highly alkaline (~pH=12.8) and may lead to a combination of alkali-silica reaction resulting in reduction of glass FRP composite strength, stiffness, and toughness, resulting in fiber embrittlement. CFC-WVU (Ajjarapu, Faza, and Ganga Rao 1994) suggested that the rate of degradation of the composite materials under harsh environmental data conditions is of an exponential form, where time (t) < 450 days. From the experimental data, it was concluded that the maximum strength reduction was 50 percent in 450 days. However, beyond 450 days, the strength did not change considerably.

Durability of glass

Silica forms a major part of glass composition, and it is the silica network that gets attacked during exposure to various environmental agents.

Alkaline attack on glass is described by two theories: (a) etching and (b) hydroxilation and dissolution leading to notching. Etching is produced by an alkali attack. As the silica network is attacked, other components of the glass are released. Hydroxilation and dissolution are caused by chemical hydroxilation of silica in the glass. Hydroxilation is associated with dissolution and is characterized by leaching of calcium from the glass. The leached calcium, when combined with water, deposits a calcium hydroxide compound on the surface of the glass and drastically reduces the rate of reaction. Following hydroxilation and dissolution, etching (notching) is caused by the formation of calcium hydroxide crystals on the glass surface as found by X-ray diffraction analysis.

Acid attack leads to a leaching process in which hydrogen or hydronium ions exchange for alkali and other positive mobile ions in the glass. The remaining glass network, mainly silica, retains its integrity. It may become hydrated if the network is relatively unstable, or it may become denser and more stable than the original glass. Acid reacts more slowly with glass than does alkali does.

Neutral pH solution attack from water, salt, etc. occurs on glass similar to that produced by acids. Also, neutral or acidic solution attack on glass may in turn become alkali attack. Alkalies removed during acidic or neutral pH solution attack reenter the solution surrounding the bulk glass and proceed to cause etching.

Sizings and surface treatments of fiber/fabric reinforcements play a vital role in composite performance. Glass fibers used as reinforcement in various FRP composite products are surface-treated with sizing agents to lubricate and protect the fibers surface, modify the fiber surface such that it is more easily compatible with and wettable by resins, and improve bonding between resins and the fiber surface. Mechanical properties and durability are strongly dependent on the interface between fibers and resin. Surface treatment produces additional bonding sites on the fiber surface, while sizing enhances fiber processability with a protective coating on the fiber surface and can provide a coupling agent for the fiber/resin bond. The sizings that work well on glass fibers with vinyl esters are designed to dissolve quickly into the resin, thus freeing up the silane coupling agents for reaction with the resin and providing good bonds.

Nanoclays consist of nanoparticles that have at least one dimension at the nanometer size. When the nanoclay is mixed with resin and exfoliated, it possesses properties ideal for reducing the moisture/chemical diffusion rates. However, selection of appropriate nanoclay involves a number of variables including morphology, cation exchange capacity, aspect ratio, charge density, degree of purity, ability to exfoliate, and others.

Creep rupture

Creep (stress) rupture, also called static fatigue, refers to the tensile fracture of a material subjected to constant sustained high stress levels during the service life of a structural element when the material reaches its strain limit. Time required to rupture under creep loads (endurance time) decreases with the increasing ratio of the sustained tensile stress to the

short-term strength of the FRP. Carbon fibers exhibit better creep characteristics compared to glass fibers. Creep behavior is significantly dependent on creep properties of the resin. Resin creep behavior will be significant if the FRP sheet/laminate is bi-directional. Many codes typically use a conservative factor to account for creep.

Fatigue

The mechanical durability of composites depends on several factors, including constituent materials (fibers and resins), manufacturing methods, freeze-thaw fluctuations with and without external load fluctuations, and others. The failure mechanisms which include thermal and mechanical fatigue are very complex in composites under harsh environments and also under variable loading with time, which include thermal and mechanical fatigue. The interaction of failure mechanisms of the constituents can lead to faster deterioration of composites than the constituents themselves. Proper accounting for thermo-mechanical fatigue response is important for the durability and safety of a structure. When a material is cyclically loaded in the presence of freeze-thaw effects and pH variations, the term "environmental fatigue" is generally used. Fatigue performance is influenced by (a) constituent material properties and their volume percents, (b) fabric architecture, (c) fiber/matrix interface bonding, and (d) type of induced load. Discernable deterioration from weathering on FRP involves exposure of fibers, surface microcracking, and weight loss. GangaRao and Vijay (2010) concluded that three years of weathering had a small (~10 to 20 percent) loss effect on the strength of glass-epoxy composites, while substantial degradation was noted under long-term exposure.

Ultraviolet effects

The creep behavior of a variety of polymeric materials under ultraviolet (UV) exposure was investigated by Regel et al. (1967). Specimens under sustained loads were irradiated with UV radiation from a lamp. During the short time period that the UV radiation was turned on, the creep strain increased sharply. The creep rate returned to the starting value without radiation. However, similar procedures using the infrared (IR) part of the spectrum did not yield similar results. Regel et al. (1967) attributed bond breakage as the intrinsic mechanism of degradation. Their experiments involving stress-relaxation indicated that UV also increased the rate in stress-relaxation. Thus, structures exposed to UV rays can exhibit higher strains under self-weight.

Temperature, freeze-thaw, and mechanical load cycling combined effects

The combined effects of temperature and dynamic loading on composites are difficult to find in the literature. An investigation was conducted by Kellogg (2005) on FRP composite effects under moisture, low temperature, and load rate. Specifically, impact toughness of a pultruded glass-FRP composite was evaluated on a parallel-to-the-fiber notched specimen. The results revealed that the load rate has the greatest influence on fracture sensitivity for a notched specimen, whereas the rate of increase of loading results in increased mean notch toughness values for wide ranges of moisture content or temperature. It was also determined that as temperatures were reduced below -25° C, the mean notch toughness increased for all test groups, which can be attributed to stiffening effects of polymer composites at low temperatures.

The greatest concern with temperature effects on composites is the laminate debonding under freeze/thaw cycling or fatiguing because of moisture expansion upon freezing. Freeze-thaw in the presence of salt can also result in accelerated degradation due to the formation and expansion of salt deposits in addition to the effects of moisture-induced swelling and drying. Using time-temperature-stress superposition principles, polymer composite material properties such as time-dependent stress at one temperature can be used to find those properties under another temperature, with certain restrictions and calibrations.

Reduction (knock-down) factors

Applying durability factors leading to reduction (knock-down) in the strength and stiffness of FRP structures is an essential element in the design of various FRP composites used as internal and external reinforcement for new and rehabilitated structures. Some of these factors were developed by professional societies and reported in their specifications or guide-specifications. These knock-down factors have to be incorporated into the design of any FRP structures.

Micromechanics, modeling, and test methods

Micromechanics approach

Micromechanical analysis of composite materials takes into consideration the geometry of the microstructure and the properties of the constituents (fiber and matrix). Representative volume element (RVE) is the smallest

portion of the composite material that contains all of the properties of the material. Because the composite is a heterogeneous material, stresses and strains are non-uniform over the RVE. However, to simplify the calculations, the volume occupied by RVE can be replaced by an equivalent homogenous material when looking at a larger scale than the RVE dimensions.

Rule of Mixtures

In isotropic materials, the uniform property relationship between Young's modulus, shear modulus, and Poisson's ratio is well known from classic mechanics of materials. Different properties of composites in different directions can be explained clearly by the Rule of Mixtures model. Further steps are necessary to find the constitutive stiffness equations of a laminate as a stack of layers in different directions and to find the equivalent apparent properties for design purposes. Using the stiffness matrix of a whole laminate, one can predict the strains and stresses in the laminate and, consequently, the strains and stresses in each layer. Such descriptions and corresponding equations for linear and curvilinear elements are available from several sources.

Numerical/Finite Element models

Macro-scale analysis (laminate level). Some current finite element (FE) software packages analyze a composite structure by computing the stiffness matrix for a laminate with a given stacking order and using it as an input for analyses. This kind of analysis is adequate to calculate the deformation response for buckling or dynamic loads and even to analyze for strain distribution throughout the laminate thickness under certain load types. However, void content, inadequate cure, fiber misalignment, etc. cannot be accounted for by the stiffness matrix computations.

Meso-scale analysis (ply level). For calculations of strains and stresses at each ply, some FE packages accept the input of the stacking order of a given laminate along with all of the parameters for each ply, including material elastic properties, thickness, and direction of each ply with respect to the general laminate coordinate system. The software calculates the stiffness matrix of the whole laminate and, eventually, strain and stress components at each ply for a given geometry and loading.

Standard test methods for FRP bars and laminates

ACI Committee 440, ASTM, CEN (European Committee for Standardization), CSA (Canadian Standards Association), and JSCE (Japan Society for Civil Engineering) provide guidelines on different test methods. These guidelines provide details about sample preparation, specimen conditions, test procedures, and calculations. Some of these are listed in Tables 1 and 2. These tables do not provide a complete list, and related aspects and specifications are listed in Appendix C of GangaRao and Vijay (2010).

Table 1. Available standard test methods for FRP laminates used as strengthening or repair materials (adopted from ACI 440.3 R-04).

Property	Test method	Property	Test method
Direct tension pull-off	ASTM-D4551 ACI 440-L.1	Tensile strength and modulus	ASTM-D3039 ACI 440-L.2
Lap shear strength	ASTM-D3165 ASTM-D3528 ACI 440-L.3	Bond strength	ASTM-D4551 ASTM-DC882

Table 2. Available standard test methods for FRP bars used for reinforcing or prestressing concrete (adopted from ACI 440.3 R-04).

Property	Test method	Property	Test method
Cross-sectional area	ACI 440 –B.1	Longitudal tensile strength and modulus	ASTM-D3916 ACI 440-B.2
Bond properties	ASTM-A944 ACI 440-B.3	Shear Strength	ASTM-D5379 ASTM-D3846 ASTM-D2344 ASTM-D4475 ACI 440-B.4
Bent bar capacity	ACI-B.5	Durability properties	ACI 440-B.6
Fatigue properties	ASTM-D3479 ACI 440-B.7	Creep properties	ASTM D-2990 ACI 440-B.8
Relaxation properties	ASTM-D2990 ASTM-E328 ACI 440-B.9	Anchorage properties	ACI 440-B.10
Tensile properties of deflected bars	ACI 440-B.11	Effect of corner radius on strength	ACI 440-B.12
Flexural properties	ASTM-D790 ASTM-D4476	Coefficient of thermal expansion	ASTM-E831 ASTM-D696
Glass transition temperature	ASTM-E1356 ASTM-E1640 ASTM-D648 ASTM-E2092	Volume fraction	ASTM-D3171 ASTM-D2584

Fabrication and field implementation

As a first step towards fabricating FRP structures, all of the required components are manufactured with or without inserts to design configure-tions and dimensions that conform to required specifications and tolerances with due consideration to QA and QC. Some of these fabrications may require a high degree of customization, including hybridization with the use of other materials. The next step involves trimming/cutting the components to required dimensions and drilling the necessary holes/openings so that the parts can be assembled in the manufacturing plant or onsite. This will be followed be joining schemes that could be a combination of mechanical fastening (bolts and rivets) or adhesive bonding. Following joining / embedment of parts, the necessary final finish is provided in the form of sealing open holes or exposed fabrics to avoid moisture ingress or providing protective UV coatings or paints. Transportation and field handling are much easier for these fabricated parts than for heavy materials made of steel and concrete. Fabrication steps may also involve hand lay-up.

FRP bonded joints

Joints are essential in FRP structures due to fabrication considerations, since complex structures come in different pieces that require different attachments. In addition, design constraints due to the low transverse modulus of FRP create major issues in terms of joining mechanisms.

Adhesively bonded joints

Strength and stiffness of adhesively bonded FRP joints are affected by several parameters.

- Bonded joints are controlled by two critical stress patterns: (a) shear stress due to unequal strains of the adherents and (b) peel stress induced at the free edge of the lap joint under eccentric loads.
- Stress concentrations at the joints are affected by thickness of the adherents, the overlap length of the bond, the thickness of the adhesive, and the stacking order of the laminate. Residual stresses during curing are considered a function of the adhesive thickness.
- Joint effectiveness is affected by the adherent's thickness, the fiber volume fraction, joint geometry, the effective adhesive length, and the fiber direction.

- Stiffness of bonded joints depends on the adhesive properties, adhesive thickness, stress distribution, and the inter-laminar shear modulus of the adherents.
- Joints may fail due to the adherent's failure, which may result from peel stress, adhesive thickness failure, or failure due to creep.

Additional details on a design approach for lap joints in composites and other types of connections are presented in GangaRao and Palkamsetty (2001).

Out-of-plane joints

Connections between two structural FRP members represent a zone of potential weakness, particularly when the two members are orthogonal to each other. In such cases, load transfer takes place in an out-of-plane mode causing stress concentrations. Two types of joints are mainly used in marine structures: (a) top-hat stiffened single skin and (b) sandwich configurations.

Special attention should be paid to the design of joints, since buckling is a major issue in laminates subjected to compressive stresses where the need for out-of-plane joints arises. Unlike conventional materials, unidirectional FRP composite joints are weak in transverse modulus.

Fabrication and installation

FRP fabrication is performed by several manufacturing methods. Methods suitable for field fabrication include hand lay-up, with modification to the scale of operation. Other methods such as filament winding could be used for large scale fabrication of parts such as cylindrical chimney liners. Some of these methods are perfected through experience (trial and error).

Fabrication accounts for structural supports and connections, site and environmental conditions, and accessibility. For example, plastic pipes, such as those used as a part of marine applications, can be field fabricated on ground, and temperature variations can cause a dimensional change in excess of 1 percent due to high thermal expansion and contraction. FRP pipes can be utilized for both underground burials and aboveground installations. FRP can be fabricated both on the ground and on a pipe platform (supporting bridge). Stiffnesses of FRP pipes differ by an order of magnitude.

Fabrication of FRP pipes can be accomplished with light tools. Deflection criteria are important for aboveground pipes, and minimizing radial stresses due to soil loads is important for buried pipes. FRP pipes can be strengthened as required in the longitudinal and radial (hoop) directions to withstand stresses and high fatigue stresses caused by varying load cycles. Large FRP cylinders used as chimney liners are typically either fabricated in manufacturing plants set up close to the construction location or transported to the field.

Fabrication aspects specific to the implementation of FRP hydraulic gates, including their hybridization (use of non-FRP inserts and steel frames), joining methods, transportation, field-handling, field-installation, and durability of areas subjected to wet-dry conditions, require additional work.

Inspection

Nondestructive techniques that can be used for FRP inspection and as QA/QC tools include (a) visual inspection, (b) tapping, (c) acoustic emission, (d) thermography, (e) ultrasound, and (f) x-ray radiography.

Visual inspection

Visual inspection is the most important and simplest technique. The inspector should consider the overall visual appearance of the entire structure for general appearance, noting the presence of discoloration that may be the result of improper wet-out or overheating. Improper curing and even accidental resin substitutions can be detected by large-scale changes in color. Large-scale debonding of a subsurface ply may be visible as lighter/darker areas. The inspector should look for local defects (ASTM 2006).

Tapping

By tapping the structural surface, an inspector can detect changes in the emitted sound. This technique can be employed for detecting many common FRP composite imperfections by combining with visual observations. This approach provides immediate clues for further inspection with methods that are more accurate. Modern tap hammers are instrumented with electronic output devices that provide quantitative recordable responses that can be correlated to delaminations in the structure.

Acoustic emission

Acoustic emission testing identifies and locates active defects in laminates by detecting minute acoustic impulses that are generated as a defect propagates under load. A major advantage of this procedure is its ability to monitor an entire part of a structure. The system consists of a specialized mixer and amplifier that feed sensor data to a signal analyzer. Most current systems can monitor 20 transducers simultaneously and analyze the input signal for arrival time, amplitude, and duration. This information can be organized and displayed graphically.

Infrared thermography

Non-destructive testing (NDT) techniques such as infrared thermography were developed as tools for detecting the in-situ condition of FRP bridge decks (shapes) and subsurface defects. The presence of subsurface defects such as debonds and delaminations formed during initial construction and in-service can adversely affect the structural integrity and service performance of FRP bridge decks. Infrared imaging allows a fast, non-contact overview of the structure, provides a global assessment of potential defects such as large inner-surface thickness irregularities and voids in the laminate, and does not require expensive equipment.

Ultrasonic

Digital ultrasonic thickness measurement instruments are available to measure the thickness of FRP parts quickly and accurately without access to both sides of the laminate. This technology depends on the fact that ultrasonic waves penetrate materials at different speeds depending on their density and type. In case of cracks or voids, ultrasonic devices can pick up the arrival times of the reflected waves and estimate the locations of voids. They also can be used for thickness measurements.

X-ray radiography

The X-ray technique used on FRP composites is relatively low power but produces high-resolution results, allowing differentiation between the corrosion barrier and the structural wall. Gauge blocks in the image plane are required for valid thickness measurements. X-ray differentiation techniques are commonly used for the micro-level study of aged FRP structures.

Rehabilitation and repair of FRP composite structures

FRP composites were used to repair structural elements because of their high strength-to-weight ratio. In addition, the emergence of resins that can cure underwater and allow FRP to be bonded to wet concrete made it possible to extend the application of FRP for emergency repair to substructure elements.

Sen and Mullins (2007) reported applications of FRP wraps to repair submerged bridge piles. At Friendship Trails Bridge connecting Tampa and St. Petersburg, Florida, 77 percent of the 254 piers needed repair, indicating a very aggressive environment. Here were used the pre-impregnated Aquawrap® Repair system with water activated urethane resin (http://www.airlog.com/FACS/FACS%20Aquawrap%20Splash%20Zone.htm) and the Tyfo® SEH-51A Repair system with Tyfo® SW-1 underwater epoxy (http://www.fyfeco.com/resources/abstracts/WaterfrontBrochure041609.pdf). Pullout tests were performed two years after the wrapping was completed to evaluate the FRP-concrete bond. Results from the bond tests showed that the wet lay-up system performed better in the partially wet and submerged regions. The water-activated pre-preg system performed better in the dry regions. (Pre-pregs consist of high-quality fabrics impregnated with curable resins.)

Rehabilitations of concrete bridge, timber bridge and viaduct structures were performed by CFC-WVU using glass or carbon fabric FRP in the last decade. These successful repair measures were implemented at a fraction of replacement costs, thus resulting in significant cost savings.

Material selection

Depending on the application (underwater or dry repair), materials could be selected for wet lay-up or open air application and use pre-preg kits. The most commonly used types of fibers in civil structures are glass and carbon. They can be selected as uni- or bi-directional woven fabric. The most commonly used resin systems are polyesters, vinyl ester, epoxy, and phenolics. Vinyl esters provide higher strength than polyesters, while epoxy resin is considered to be a high performance resin.

Access and repair equipment

The major equipment required for the repair operation is whatever can provide access to the location of the repair. A bridge might need special scaffolding, a crane, or a truck-mounted bucket platform for access under a bridge deck or for the sides of the bridge. For underwater repairs, boats and divers will probably be needed. Repair equipment is usually simple, consisting of items such as pressure washer machines, sanders, grinders, mixers, and lay-up rollers.

Repair procedures for concrete substrate and evaluations

Generally, repairs consist of the following six steps, and additional details can be found in respective repair guidelines and specifications provided in Appendix C of GangaRao and Vijay (2010). Post-repair evaluation can be accomplished immediately by visual inspection to make certain that there are no apparent voids that need to be fixed. Using any of the non-destructive testing methods assures that the cured product meets the design requirements. The general steps used while repairing concrete or timber structures with FRP composites include the following.

1. Clean the repair location using pressure washing or suitable equipment.
2. Sand the surface of the structure at the repair location to remove damaged surfaces and sharp angles.
3. Grind the surface to provide roughness, and vacuum and dry the surface for better bonding of the fabric to the concrete substrate.
4. Apply primer coat.
5. Apply resin coat and then fabric/fiber layers alternately to build the required thickness for strength and stiffness, if necessary.
6. Apply protective topcoat.

Following repair/rehabilitation procedures, representative samples should be taken for laboratory testing for compliance with governing codes. It should be noted that repair of FRP composites with other FRP composites is not very prevalent and should be properly evaluated with respect to compatibility of repair materials.

Worldwide FRP hydraulic (miter) gate implementation and related components

Application of hydraulic gates worldwide is discussed in detail by Permanent International Association of Navigation Congresses (PIANC) Reports 105 and 106 (2009), including innovations to those lock gates. Miter gates of interest from the perspective of FRP composite application typically consist of two gate leaves that form a three-hinged arch when the gates are in the closed position. The three-hinged arch reactions are transmitted into the supporting piers by hinges or quoin posts at the support and by spot bearing blocks at the miter ends of the horizontal girders. Supporting structures and their foundations are designed to minimize deflections at the gate hinges or quoin posts under hydrostatic pressure so that the gate can function properly. Joint seal assemblies are provided for water tightness.

Japanese/Asian experience

In Japan, 438 FRP gates were installed between 1961 and 2002, with more than 80 percent installed before 1990. However, construction has declined in recent years (Tomiyama and Nishizaki 2006). Different styles of FRP gates were adopted including (a) slide gate, (b) flap gate, (c) roller gate, (d) swing gate, (e) miter gate, and (f) angle chute. Almost 90 percent of adopted FRP door bodies are compact, with the gate area smaller than 4.0 sq m.

Field studies reported by Tomiyama and Nishizaki (2006) were conducted on 53 in-service FRP gates. Results of visual inspection of all investigated FRP gates showed no major deterioration except for slight discoloration and water stain. Over 30 years, they were maintained only lightly. Tomiyama and Nishizaki (2006) concluded that “FRP gates seem to be more durable that steel gates after an equivalent period of time.”

Tomiyama and Nishizaki (2006) tested the strength of FRP sluice gates as part of an agricultural waterway system after the gates had served in the field for more than 35 years. Samples were cut from the gates, and test results were compared with test data on newly fabricated FRP samples with the same laminate composition as the gates being studied. Results show that FRP is suitable for water gates on a long-term basis. However, it is important to note that the FRP gate design requires several considerations to avoid potential in-service problems.

U.S. experience

Chowdhury and Hall (1998) reported the test results of full-scale hydraulic wickets at the Olmsted Wicket Dam, Smithland Facility, Kentucky. Tested prototypes included four traditional steel and one hybrid-FRP composite wicket. The composite gate was designed to be interchangeable with the other steel gates, and the supporting devices were independent from the gates. The hybrid composite gate consisted of a welded steel frame support with composite face and supporting I-beams. Geometry, strength, and stiffness requirements of composite gates were designed to match the steel gates.

Performance comparisons were conducted for the hybrid-FRP and steel composite wickets. Conclusions included the following.

- No major signs of distress were noted in either composite or steel gates immediately after installation.
- After 400 cycles of underwater operations (under highly abrasive and corrosive conditions) simulating 25 years of real operational cycles, long-term performance of the hybrid-composite and steel gates were visually observed and compared. The composite gate seemed unaffected with the minor exception of edge peel-off on both gate leaves, whereas the steel gates showed extensive corrosion.
- The composite gates needed less maintenance than steel gates.
- The flow experiments indicated that a lighter composite gate has a higher vibrational response than a steel gate of identical flow-induced hydrodynamic load history, and an agreement of the mode shapes suggested that the dynamic motion of both wickets is expected to withstand a similar pattern for low frequency dynamic load (Chowdhury and Hall 1998).
- This study recommended elimination of free edges in the design of composite gates in damage-prone areas.

European and other experience

PIANC Working Group (WG) Report 105 (2009) lists several glass FRP applications as alternative materials in marine structure construction. These applications utilized FRP structural shapes, FRP non-metallic reinforcements, structural strengthening materials such as FRP wraps, and hybrid structural elements consisting of conventional and composite materials. Some of these FRP applications in the United States include

(a) polymeric fender piles at a Navy pier in San Diego, California, and at a marine terminal in New Orleans, Louisiana, (b) recreational piers, (c) sheet pile at Masonboro Harbor, North Carolina, and sheet pile walls in Florida, (d) a thermoplastic guide wall and deck floating pontoon at the Port Allen navigation lock in Louisiana, (e) tongue and groove vertical plank walls, (f) fiberglass gratings and a thermoplastic lumber boardwalk at a marina, (g) piles with an FRP shell and a concrete core, (h) utility poles and cross beams, (i) FRP I-beams installed on an underwater timber pier, (j) FRP strips for concrete pier strengthening, and (k) composite hybrid decks. Also, international examples of alternate composite materials usage were presented, which included (a) wood-plastic composite members exposed to marine environment in Japan, (b) a glass FRP walkway structure in Italy, (c) glass FRP reinforcement for concrete in Saudi Arabia, (d) FRP reinforcement in Nakeel's Palm Cover canal project in Dubai, and (e) FRP reinforcement in the deck area for the Ben Schoeman Quay Expansion in Cape Town, South Africa.

PIANC WG Report 106 (2009) consists of a hardcopy and a DVD report that includes a review of 56 lock projects, a revision to the PIANC 1986 lock report, a dictionary on locks and waterways, and a worldwide list of locks. This report includes additional information and references regarding (a) effects of salt-water intrusion, (b) construction process modeling, (c) hydraulic aspects, (d) gates and valves, (e) lock equipment, and (f) lubricants and biological oils. This report also includes references to various other technical guidelines developed by different countries. This miter document reports the benefits of application of FRP composites for miter gates and other lock gates. Advantages of composites for miter gates as identified by PIANC WG Report 106 include (a) non-corrosion, (b) good resistance to aging in damp environments, (c) non-requirement of finishing paint, (d) reduced maintenance/transportation and gate fitting costs, and (e) less purchasing and maintenance costs of heavy equipment/machinery.

Many of the marine construction projects reviewed in PIANC WG Report 106 are suitable candidates for implementation with FRP composites. Some of the innovative concepts described in this report related to gates include (a) folded plate for gates (Germany), (b) a reversed miter gate (Netherlands, UK), (c) suspended miter gates (Netherlands), (d) rotary segment gates with horizontal axis (Germany), (e) vertical-axis sector gates (Germany, Finland, Japan), (f) composite lock gates (France), (g) gate linings and seals (Netherlands), (h) corrosion protection measures, (i)

self-propelled floating lock gates, and (j) rolling gates with integrated filling/emptying system. The review also includes a description of the study of a vertical-lift arch gate constructed of composite materials (France). PIANC WG Report 106 also stated that the "Spieringsluis" in the Netherlands was designed with a high-strength synthetic composite material to reduce maintenance costs of wooden or steel gates.

Some examples of uses of FRP composites for lock systems with Tenmat T814 composite consisting of phenolic resin and polyester fibers and polyethylene (http://www.tenmat.com/Content/T814) include (a) synthetic composite bushings to replace polyamide bearings that deteriorated faster than originally specified in the Orange Locks complex, Amsterdam, (b) heel bushing of Tenmat T814 composite for a shaft with a 316L stainless steel cap for a lock on an aqueduct in Enkhuizen, and (c) "soft" low-friction heel cap of polyethylene on an old shaft with a new 316L stainless steel cap on a high, narrow, and light (timber) lock gate with low hinge loads on the Wilhelmina Canal in Tilburg (PIANC Workshop 2009).

A schematic of the forces acting on miter gates during closing operations and of the importance of considering contact aspects of gates is described by Rigo and Ryszard (2010) and in PIANC Workshop (2009) presentations. Flat-shell action of the gate structure with in-plane forces in open position starts changing when the two gate leaves meet and the water head grows. At first, the top hinge releases followed by the bottom hinge, and then the gate passes its loads to the heel posts, wherein perpendicular loads imposed on the gate along with the in-plane loads change the gate's response from "flat shell" to "plate" behavior.

A review of these documents by PIANC, including implementation of FRP materials and structural systems for marine structures in the USA and in Europe, indicates the potential of FRP as a construction material for different types of lock gates and also for their repair. The design, development, and implementation work that are needed for miter gates include the following.

1. Design of FRP lock gates with a focus on specific types (e.g., miter gates) and applicability of the available design guidelines, specifications, and research experience,
2. Decision on (a) performance-based design parameters, including dimensions, allowable deformations, stress-strain levels, fatigue

parameters, energy absorption, and durability, (b) selection of materials (fiber/fabric architecture, resin, additives), (c) production methods including QA/QC, (d) assembly/fabrication issues at manufacturing facility/field location, (e) joining schemes and integration with other frameworks (e.g., hinges, support frame, etc.), (f) transportation, (g) field handling and site erection, (h) field performance monitoring, and (i) life-cycle cost analysis,

3. Miter gate production using (a) pultruded shapes and plates that can be assembled in the field or in the manufacturing plant, and/or (b) in-situ or manufacturing plant-based vacuum-assisted resin-injection molding process,
4. Limited laboratory and analytical evaluations specific to FRP structures, ensuring compliance with design limit states,
5. Field implementation and monitoring, and
6. Repair of FRP structural elements and evaluation of their durability in the laboratory and on actual structures in the field.

Conclusions

The feasibility of design, development, and implementation of FRP composite structural systems with a focus on civil and marine applications is provided by GangaRao and Vijay (2010). Discussions on revisions of individual chapters of the USACE Technical Letter "ETL 1110-2-548, Composite Materials for Civil Engineering Structures," (USACE 1997b) is provided throughout that report and also included in Appendix B of GangaRao and Vijay (2010).

The scope of GangaRao and Vijay (2010) is limited to brief presentations on FRP constituents, structural shapes, and systems, including their field implementation in bridges, buildings, marine structures, automobiles, aircrafts, and others. Short- and long-term properties and the influence of fiber orientation on strength, stiffness, and deformation of composite products are described under combined external and environmental (durability/aging) loads.

Necessary input is provided to this study wherein Japanese/European knowledge and practice are highlighted. To indicate the advancement in FRP applications in those countries, some of the marine structures with specific focus on hydraulic gates implemented in Japan and Europe are included. Within the scope of this work, composite and constituent material specifications and guidelines for the design, testing, fabrication,

non-destructive testing, inspection, and repair are provided and discussed. Additional work and elaboration on these aspects, including field performance, data analysis, and major revision to the technical letter ETL 1110-2-548 (1997a), are necessary.

7 Summary and Conclusions

Summary

John T. Myers Locks and Dam is located on the Ohio River at Mile 846.0, about 3.5 miles downstream from Uniontown, Kentucky. This is an important facility for providing navigation capability for the Ohio River, a major artery for commercial navigation in the United States.

The lock chambers at John T. Myers Locks and Dam are very susceptible to wall damage due to the large amount of traffic passing through these locks. The majority of the damage includes gouges and spalls from barge impacts and abrasion in the concrete adjacent to an armor strip. Many of the gouges are next to a vertical joint. In addition to the spalls and gouges, wall armor separation exists at several locations.

The majority of the armor damage occurs in the 1,200-ft lock chamber as opposed to the 600-ft chamber. The armor damage is a result of a combination of impact and abrasion by commercial barge traffic that typically uses the 1,200-ft chamber. Wall armor separation is vulnerable to "catching" protruding metal on barges. This is a special concern for barges that have protection themselves.

Extensive armor repairs are necessary at John T. Myers Locks and Dam due to the lack of vertical armor protection and the high volume of barge traffic. Innovative repair techniques must be researched and demonstrated to achieve repair methods that will not disrupt navigation traffic. Additionally, non-traditional materials for repair and rehabilitation of concrete structures, such as fiber reinforced polymer (FRP) composites, should be evaluated for specific application to inland hydraulic navigation locks and dams.

This MCNP monitoring study of expedient lock wall repair demonstrations at John T. Myers Locks and Dam consisted of the following five aspects.

Evaluation of John T. Myers wall armor system

The majority of the observed damage was located where straight-run wall armor was terminated. This was particularly true where the armor was

terminated near vertical monolith joints. Two such joints existed on the land-wall side of the 600-ft lock, one at the upper approach and a second at the lower approach. Thus, the opportunity existed to demonstrate two fundamentally different repair techniques at these two locations with essentially no disruption to river traffic. The first technique attached two concrete monoliths at the vertical joint so as to create a single unit. The second technique did not attach two concrete monoliths at the vertical joint but allowed them to remain as two separate units. In both cases, the steel plates acted as permanent forms and were backfilled with a rapid-hardening, high-early-strength, low-shrinkage concrete.

General classifications of observed damage were described and, where repairs were considered necessary or desirable, potential repair strategies were proposed. Detailed repair procedures were developed and prioritized as a joint effort between ERDC and CELRL. Factors that should be considered include the following.

- Impact on the users must be limited. Repairs should be accomplished without shutdown of the lock if at all possible.
- The smaller auxiliary chamber should be used to demonstrate and optimize repair techniques to minimize impact on the main chamber.
- All expedient repair methodologies should be considered because of the major economic impact of any interruption of normal operations. Any potential for the use of advanced materials, techniques, and equipment should be considered.
- Repair techniques should focus on restoring the system to its original capability.
- A primary advantage of polymer-cement materials is the enhanced bond between repair and existing substrate concrete.
- While there are a variety of rapid-hardening cementitious materials that may be appropriate, prefabricated stay-in-place steel forms have merit and should be evaluated.
- Use of rapid-hardening repair materials with compressive strengths in accordance with ASTM (2000) C928 should be considered to minimize downtime.

Repair of 600-ft chamber upper land-wall approach vertical joint

This first vertical-joint repair technique fit steel plates over the vertical joint separating monoliths L-35 and L-36. The plates were welded together at all

joints, and 60 concrete anchor bolts were spot-welded to the plates. This technique attached the two concrete monoliths together as one unit.

A team of CELRL and ERDC GSL personnel developed a repair technique in September 2006 that would replace old deteriorated concrete by installing steel plates over the new concrete using the steel plates as stay-in-place forms held by anchor bolts. It was believed the steel plates would provide more protection from barge impact at this particular vertical joint location where the lock chamber wall meets the approach wall on the upstream side of the chamber.

Replacement steel plates were 0.5-in. thick and cover 36 in. in width on the upstream monolith and 48 in. in width on the downstream monolith, for a total coverage of 7 ft in width. The steel plate could have been one piece factory-bent to fit the profile of the two monoliths or two pieces placed individually and welded at all seams. The total steel plate coverage would then have been 140 sq ft. The contractor chose to install four pieces of steel plate, each 5-ft-high (20 ft total height) and 7-ft-wide, and each factory-bent to fit the monoliths. Each individual steel plate covered 35 sq ft, and the total coverage for the 4 steel plates was the required 140 sq ft.

Saw cuts by a diamond blade saw were made along the template lines, and a hand-held impact concrete breaker was used to dislodge concrete within the sawed area being repaired. Internal bolt-and-nut anchors were next installed to hold the rapid-set, high-early-strength concrete in place. The cavity void was then spread-filled with rapid-set, high-early-strength concrete grout material meeting ASTM (2000) C928, Type R3 (or equivalent) to form a base for the backside of the plate.

Next, 7/8-in.-diam. steel plate anchor bolt holes were drilled into sound concrete on approximately 2-ft centers, staggered off the internal bolt-and-nut anchors previously installed to hold the rapid setting concrete. The steel plate was attached with anchor rods. All steel plate anchor rods were fully welded to the steel plate. All horizontal joints between the steel plates were welded full length and full depth, and the steel plates were fully welded to the wall armor rub strips that were cut to butt-weld to the steel plates. All welds were ground flush, and the steel plates were painted to conform to the lock structure appearance.

This repair technique effectively connected two concrete monoliths as one unit with a continuous steel plate that was firmly anchored into the sound concrete of both individual monoliths.

Repair of 600-ft chamber lower land-wall approach vertical joint

Subsequent to the repair of the 600-ft chamber upper land-wall approach vertical joint (September 2006), a weld was found to have cracked due to elemental movement between the two monoliths caused by uneven settlement or thermal expansion and contraction. Hence, the demonstration in September 2007 of a fundamentally different repair technique devised for the 600-ft chamber lower land-wall approach vertical joint was well-founded from the standpoints of both an unexpected consequence as well as an alternative repair methodology.

The lock wall design at the lower vertical joint is significantly different from the upper lock wall design. Here, the wall armor rub strips extend all the way to the top of the wall on monolith L-2 (the main lock chamber wall), but terminate about 13.5 ft below the lock wall on monolith L-1 (the guide wall entrance into the main lock chamber).

The second vertical joint repair technique required 12 16-in.-high by 12-in.-wide by 0.75-in.-thick steel plates for monolith L-1, and 17 16-in.-high by 12-in.-wide by 0.75-in.-thick steel plates for monolith L-2. The wall armor plate strips required recesses cut on the top and bottom to accommodate the steel plates that were then welded together horizontally at each joint for each independent monolith. Each steel plate was attached to the wall by spot-welding to four concrete anchor bolts. The vertical steel joints between monoliths L-1 and L-2 were not to be welded together. Thus, the two concrete monoliths were not attached but remained two separate units.

The supplemental repair wall armor plates were located in the spaces between the existing horizontal straight-run wall armor. The contractor created an accommodating recess by cutting 2-in.-high by 12.5-in.-wide notches in the existing horizontal straight-run wall armor (which were positioned with approximately 12.25 in. of space between them) and saw-cut and removed the concrete to a depth of approximately 2 in., or to sound concrete.

The supplemental wall armor plates were placed onto anchor bolts utilizing threaded nuts on each side of the wall armor plate and adjusted to maintain the top surface of the supplemental wall armor plates to be flush with the top surface of the existing horizontal straight-run wall armor at the same plane as the edge of the cut-out notches.

The contractor welded full length and full depth along the top and bottom of the horizontal length of the supplemental wall armor plates, including the 2-in. vertical seams at the ends of the cut-out notches. The outer anchor rod nuts were then removed, and the anchor rods were cut and welded solid to the armor plate and ground flush with the face of the armor plate. The contractor then spread/pumped a grout mixture meeting ASTM (2000) C928, Type R3 specifications to fill the void and form a base for the backside of the armor plate.

The top 13.5 ft of monolith L-1 on the downstream side of the vertical joint undergoing repair did not have wall armor rub strips. Hence, the repair steel plate for this area was not 16-in.-high by 12-in.-wide as had been those placed below. This section of repair steel plate required dimensions of 13.5-ft-high by 12-in.-wide. The anchor bolts embedded into the concrete monolith were spaced appropriately for this dimension, as were the holes drilled into the plate.

Repair of 1,200-ft chamber upper-river approach wall

The area of concrete damage selected by the Louisville District for repair demonstration monitoring was located in lock monolith R-73, an upper-river approach wall to the 1,200-ft chamber. The surface area of the damage was about 1.5-ft by 6-ft with an apparent maximum depth of almost 2 ft. The contractor was directed to install repairs to the damaged concrete by using rapid-setting, high-early-strength concrete applied with the form-and-pour method. This is a technique that requires filling a cavity between the formwork and the prepared substrate with a repair material and consolidating that material by vibration or rodding.

Mechanical anchors between repair and existing concrete were required. Actual numbers and locations of anchors were determined in the field depending on size and shape of the repair cavity after concrete removal. Anchors had a minimum cover of 2 in. over the headed ends. Either bonded or expansion anchors were installed in holes drilled into the concrete substrate.

Placement openings or chutes were required to place the repair material and allow for insertion of a vibrator for internal consolidation of the repair material. Chutes were constructed to allow overfilling of the repair cavity prior to consolidation. This was necessary to ensure that the upper edges of the repair cavity were completely filled after consolidation. Forms were designed to permit rapid removal without hammering or prying against the repair.

Based on tests performed at ERDC, a repair material that satisfied the criteria necessary for a successful repair at John T. Myers Lock was developed. These criteria included a mixture with good workability, a 1-hr (minimum) initial time of set, and a minimum 3-hr compressive strength of 3,000 psi. Elevated temperatures similar to anticipated ambient conditions at the project site were emphasized during the lab investigation.

No. 5 grade 60 reinforcing steel installed in drilled holes was used as anchors. Lengths of individual anchors varied depending on location and orientation within the repair. Typically, anchors had an embedment depth of 6 in. and a minimum cover of 3 in. Anchors were embedded in a medium viscosity epoxy. A total of 18 anchors were installed.

The formwork was removed approximately 2.5 hr after completion of placement. The repair cavity was completely filled, and the exposed surface was uniform and sound. The excess material in the vicinity of the chutes was removed. Overall, the exposed surface of the repair had a generally uniform appearance, and the wall repair demonstration was deemed to be fully satisfactory.

Fiber reinforced polymer (FRP) composites for inland hydraulic structures

Because it is prudent to remain abreast of the latest developments in materials technology that might be applicable to repair and rehabilitate marine structures, West Virginia University was commissioned to conduct a feasibility review of FRP. That study dealt with the possibility of design, development, and implementation of FRP composite structural systems that are of interest to USACE, focusing on civil and marine applications. Constituents, short- and long-term properties, and influences of fiber orientation on strength, stiffness, and deformation of composite products were described under combined external and environmental loads. That study focused primarily on glass polymer composites (fiberglass).

Glass is one of the most common types of fibers used as reinforcements in composites. Fibers can be long (continuous), short, chopped, milled, or in the form of elongated single crystals. Continuous fibers come in the form of untwisted bundles as strands or twisted bundles as yarns, and also as a collection of parallel continuous strands (roving).

Resins are the polymer binders that hold the fibers together and transfer the loads between the fibers, in addition to protecting them from environmental factors and carrying shear loads. Thermoset resins (e.g., polyester and epoxy) transform into matrix binders after curing through an irreversible chemical reaction. By heating, thermoplastic resins are softened from solid state before processing (making a composite) without chemical reactions. Thermoplastics return to solid state (matrix) once processing is completed.

FRP composites are ideally suited as a quick and effective structural repair tool because of their light weight, high strength, and corrosion resistance. The availability of resins that cure underwater extended the wrap application to substructure elements such as partially submerged piles that are damaged. Also, FRP composites were used in offshore platforms where corrosion in the presence of seawater is a major concern. Some of the current FRP applications include (a) buoys and floats, (b) strengthening of primary steel structures, (c) helicopter landing decks, and (d) walls and floors to provide protection against blast and fire.

Miter gates of interest from the perspective of FRP composite application typically consist of two gate leaves that form a three-hinged arch when the gates are in the closed position. The three-hinged arch reactions are transferred into the supporting piers by hinges or quoin posts at the support, and by spot bearing blocks at the miter ends of the horizontal girders. Supporting structures and their foundations are designed to minimize deflections at the gate hinges or quoin posts under hydrostatic pressure so that the gate can function properly.

Full-scale hydraulic wickets were tested at the Olmsted Wicket Dam, Smithland Facility, Kentucky. Tested prototypes included four traditional steel and one hybrid-FRP composite wicket. The composite gate was designed to be interchangeable with the other steel gates, and the supporting devices were independent from the gates. The hybrid composite gate consisted of a welded steel frame support with composite face and

supporting I-beams. Geometry, strength, and stiffness requirements of composite gates were designed to match the steel gates.

Conclusions

At John T. Myers Locks and Dam, the majority of lock wall damage was located where straight-run wall armor was terminated. This was particularly true where the armor was terminated at a vertical monolith joint. The relatively small concrete section between the end of the armor and the monolith joint was inadequate to resist the impact loads transmitted through the armor. This resulted in localized crushing and damage to the concrete. One solution was to saw-cut around and chisel out the damaged wall material down to sound concrete, install bolt-and-nut anchors to hold new concrete in place, and refill the cavity with glass fiber reinforced polymer grout mortar or other impact resistant material.

A second type of damage to the wall occurred where straight-run wall armor intersected vertical corner armor for floating mooring bits, ladders, etc.

A third area of great concern existed at vertical joints where two monoliths meet at an angle. Damage occurred both in and above the armored zone. The most severe damage occurred outside the armored zone, and impact appeared to be the primary cause of this damage. A solution was to saw-cut around and chisel out the damaged material down to solid concrete, install anchors and vertical wall armor steel plates on both sides of the joint, and grout the void with non-shrinking grout.

A fourth type of wall damage occurred where wall armor terminated within a concrete monolith, and was generally localized near the end of the individual armor sections. Impact forces appeared to have caused this problem.

The fifth type of wall armor damage at John T. Myers Locks and Dam occurred due to loss of concrete between armor strips on the bullnose. Here however, the consequences of a catastrophic event such as a barge sinking are not as dire as they would be inside the lock chamber.

To minimize adverse impacts to the shipping industry, any repairs to a multiple lock system such as John T. Myers Locks and Dam should be performed during a scheduled outage or when river traffic can be diverted to a secondary lock. Use of rapid-setting, high-early-strength repair

materials with compressive strengths in accordance with ASTM (2000) C928 should be utilized to minimize downtime. Also, fiberglass reinforcement should be included to improve toughness and impact resistance.

Two distinctly different repair techniques were utilized at the 600-ft lock chamber. At the upper land-wall approach vertical joint, the damaged material was saw-cut around and chiseled down to sound concrete for a distance of 3 ft on the upstream side of the joint and 4 ft on the downstream side of the joint. The armor strips were cut at these locations. Steel armor plates were placed over these voids on either side of the vertical joint and were held at the proper location by anchor bolts with nuts on each side of the plates. Rapid-setting, high-early-strength grout was pumped into the void behind the plates. Here, the steel plates became permanent forms for the new concrete grout. The steel plates were then full-welded all around and full-welded to the ends of the anchor strips. Then, the outside nuts were removed from the steel plates, and the bolts were cut and smoothed flush with the plates. Finally, the anchor bolts were welded to the steel plates in accordance with the manufacturer's recommendations. This effectively joined two distinctive monoliths into one unit.

An unintended consequence of joining these two monoliths was that these monoliths experienced uneven settlement or thermal expansion / contraction. This elemental movement resulted in at least one weld cracking. Hence, the demonstration of a fundamentally different repair technique was devised for the lower land-wall vertical joint to eliminate the possibility of weld failures. The two monoliths should not have been welded together with steel plates to form one unit.

At the lower land-wall approach vertical joint, the armor strips were not cut to reduce their length. Here, the concrete was saw-cut and chiseled for a distance of 1 ft on either side of the vertical joint. The contractor created an accommodating recess in the armor strips by cutting a 2-in.-high by 12.5-in.-wide notch in the existing horizontal run wall armor. Then, 16-in.-high by 12-in.-wide steel plates were spot welded to the armor strips in the notches that had been previously cut. Here again, the steel plates became permanent forms for the new concrete grout. The void behind the plates was then filled with rapid-setting, high-early-strength grout, and all horizontal and vertical joints between the steel plates and the armor strips were full-welded all around. The steel plates on each side of the vertical joint were

not welded to each other; hence, the two monoliths were not attached as a single unit. The unintended consequences of weld breakage experienced at the upper land-wall vertical joint, where the two monoliths had been joined to become one monolith, were precluded.

A section of the 1,200-ft chamber upper-river approach wall was selected for repair demonstration. Here, the form-and-pour technique was applied after the damaged section was saw-cut around and the damaged concrete chisel-removed to form a void for new concrete. Anchor bolts consisting of reinforcing steel rods were placed into holes drilled at angles and stabilized with epoxy glue to hold the new concrete. At least 2 in. of new concrete covered all bolts. A rapid-set grout developed by the ERDC concrete laboratory was poured into the cavity. The formwork was removed 2.5 hr after placement. The repair cavity was completely filled, and the exposed surface was uniform and sound.

The feasibility of using fiber reinforced polymer (FRP) composites for inland hydraulic structure application was investigated. FRPs are referred to as the materials of the 21st century because of their many advantages. Extensive usage on civil infrastructure is inhibited because of lack of data on long-term field performance. Japanese and European experience is more extensive than U.S., particularly with hydraulic and miter gates. FRP use in inland navigation structures is very promising, since these materials have better durability and cost effectiveness than conventional materials at appropriate applications.

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14. ABSTRACT

The lock chambers at John T. Myers Locks and Dam are very susceptible to wall damage. The majority of the damage includes gouges and spalls in the concrete adjacent to a lock wall armor strip. Many of the damaged regions are next to a vertical joint. The majority of the damage occurs in the 1,200-ft lock chamber as opposed to the 600-ft chamber. Innovative expedient repair methods and techniques were evaluated and demonstrated that will not disrupt river navigation traffic. Non-traditional materials for repair and rehabilitation of concrete structures such as fiber reinforced polymer (FRP) composites were evaluated for specific application to inland hydraulic navigation locks and dams. This Monitoring Completed Navigation Projects (MCNP) expedient lock wall repair demonstration consisted of five aspects: (1) Evaluation of John T. Myers Locks and Dam wall armor system, (2) Monitoring repair of 600-ft chamber upper land-wall approach vertical joint, (3) Monitoring repair of 600-ft chamber lower land-wall approach vertical joint, (4) Monitoring repair of 1,200-ft chamber upper-river approach wall, and (5) Feasibility of fiber reinforced polymer (FRP) composites for inland hydraulic structure application.

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