my Corps
ers
:xperiment

THE REMR BULLETIN

Volume 14, Number 1

February 1997

News from the Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Development of a Geomembrane System for Underwater Repair of Concrete Structures

by James E. McDonald, U.S. Army Engineer Waterways Experiment Station

The U.S. Army Corps of Engineers operates and maintains a wide variety of hydraulic structures, including mass-concrete gravity dams, rock-fill dams with concrete facings, and roller-compacted concrete dams. Concrete appurtenances associated with such dams include intake towers, outlet works, and stilling basins. Located at over 600 project sites throughout the United States, these structures are subjected to a wide spectrum of environmental conditions. Also, the advanced ages of these structures, more than 40 percent of which are over 50 years old, increase the potential for concrete deterioration.

Many of these structures exhibit concrete cracking, which allows water intrusion into or through the structure. Water leakage through hydraulic structures can also result from poor concrete consolidation during construction, improperly prepared lift or construction joints, and water-stop failures. When leakage rates through cracked or deteriorated concrete and defective joints become unacceptable, repairs are made. Conventional repair methods generally consist of localized sealing of cracks and defective joints by cementitious and chemical grouting, epoxy injection, or surface treatments. Even though localized sealing of leaking cracks and defective joints with conventional methods has been successful in some applications, in many cases some type of overall repair is still required after a few years. Consequently, the potential for geomembranes in such repairs was evaluated as part of the Corps' Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program.

Various configurations of geomembranes have been used as impervious synthetic barriers in dams for more than 30 years. Generally, membranes are placed either within an embankment or rock-fill dam as part of the impervious core or at the upstream face of embankment, rock-fill, and concrete gravity dams. In recent years, geomembranes have been increasingly used for seepage control in a variety of civil engineering structures, including canals, reservoirs, storage basins, dams, and tunnels. Geomembranes have also been used successfully to resurface the upstream face of a number of old concrete and masonry dams, particularly in Europe.

A review of geomembrane applications (McDonald 1993) indicated that the success of these systems in arresting concrete deterioration and controlling leakage in dams, canals, reservoirs, and tunnels and the demonstrated durability of these materials are such that these systems are considered competitive with other repair alternatives. With a few exceptions, geomembrane installations to date have been accomplished in a dry environment by dewatering the structure on which the geomembrane is to be installed. Dewatering, however, can be extremely expensive and in many cases may not be possible because of project constraints. A durable geomembrane system that could be installed underwater to minimize or eliminate water intrusion and leakage would be an economical alternative for repair of a variety of hydraulic structures. Consequently, research was initiated to develop a procedure for underwater installation of geomembrane repair systems.

A two-phase contract to develop the system was awarded to Oceaneering International, Upper Marlboro, MD, and CARPI/USA, McMurray, PA, based on their respective expertise in underwater construction and geomembrane systems for dam rehabilitation. In Phase I, a conceptual design for the underwater repair system was developed based on research, material testing, and detailed evaluation of individual components and procedures. The constructibility of the design was demonstrated in Phase II through successful underwater installation of the system on a simulated concrete structure.

Conceptual Design

The objective of this phase of the study was to perform research, material testing, and evaluation of individual components and techniques required to facilitate successful underwater installation of membranes and to develop a procedure for underwater installation on the upstream face of a dam. Work in this phase included developing design criteria, surveying available materials, conducting material testing, and evaluating materials and assembly techniques. Material testing was conducted, when applicable, in accordance with standardized tests. However, other even more valuable information was collected with nonstandardized tests, namely with multiaxial, large-scale tests or tests that were intended to simulate conditions likely to be encountered during actual installation. Testing was conducted on drainage materials, membrane materials, anchorage profiles, gaskets, anchor bolts, and surface repair compounds.

Various types and thicknesses of geomembranes were tested to determine their conformability, burst resistance, and puncture resistance in the presence of a very rough substrate (Figure 1). Samples of membrane were placed in a pressure vessel that was sealed and pressurized to a maximum pressure of approximately 150 psi (1 MPa). Samples of membrane that did not rupture during pressurization were subjected to the maximum pressure for 24 hr. The specimens were then removed from the pressure chamber and inspected. A sample of reinforced polyvinyl chloride (PVC) after testing is shown in Figure 2. Obviously, the membrane conformed to the very irregular substrate without puncturing.

The mechanical fastening system that secures and seals the membrane system to the surface of the structure also received considerable attention in the design phase. The stainless steel profiles must be flexible enough to conform to the substrate, yet stiff enough to ensure continuous compression of the gasket without an excessive number of anchor bolts. The performance of both chemically grouted and mechanical anchors installed under submerged conditions was evaluated. A profile and gasket conformability test is shown in Figure 3. In this test, a 1-in. (25-mm) -thick, open-cell neoprene gasket is being compressed by a 1/4-in. (6-mm) -thick stainless-steel profile with anchor bolts on 12-in. (305-mm) centers.

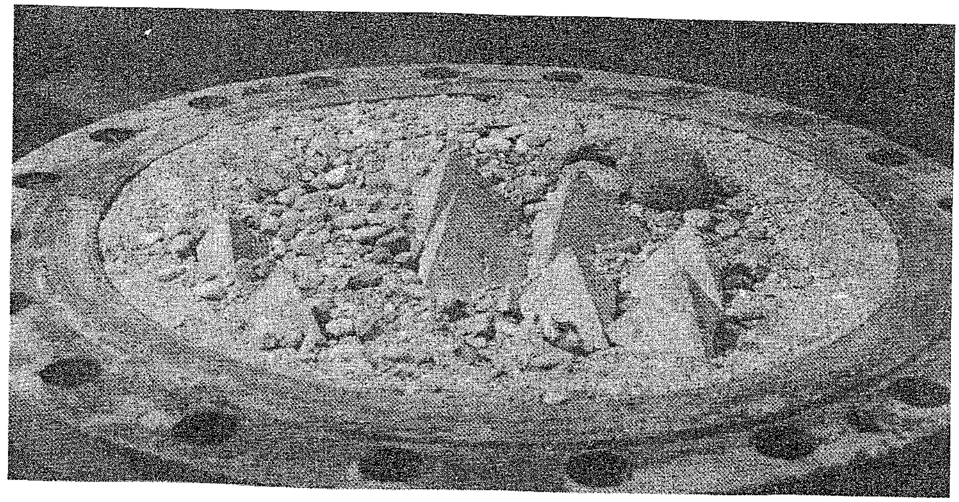


Figure 1. Simulated substrate in the puncture test

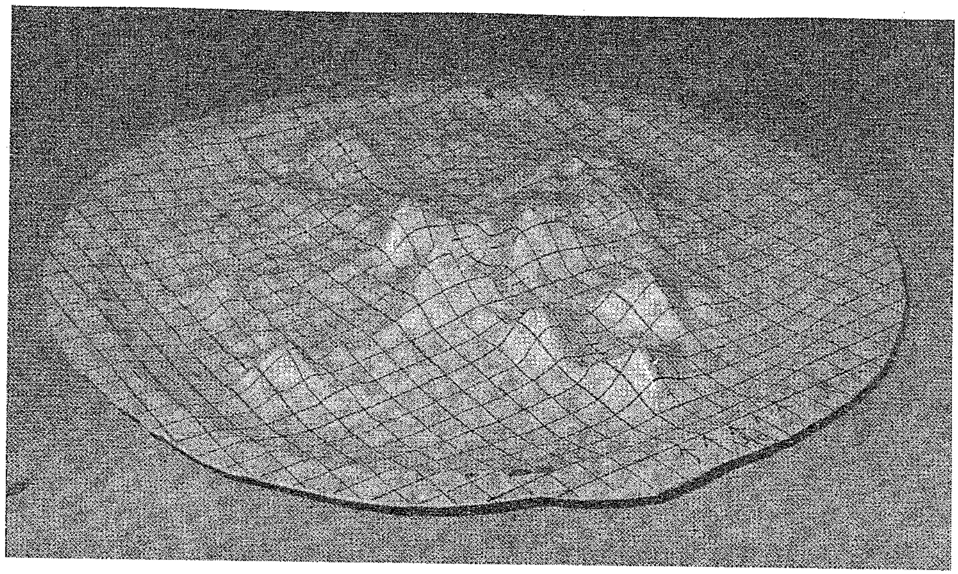


Figure 2. Condition of the reinforced PVC after a sustained load of 150 psi (1 MPa) for 24 hr

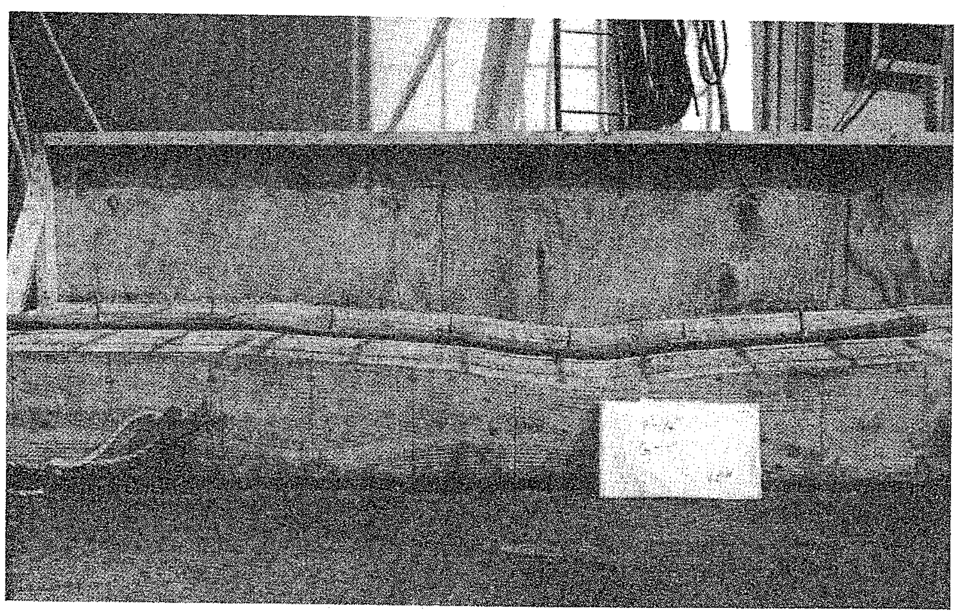


Figure 3. Profile and gasket conformability test

The geomembrane system designed for underwater installation on the upstream face of a dam consists of a high-density polyethylene (HDPE) geonet drainage layer, and a PVC geomembrane backed with geotextile reinforcement, anchored and sealed around the perimeter and along vertical splices (Figures 4 and 5). Development of the system is described in detail by Christensen et al. (1995).

A PVC geocomposite consisting of a geomembrane backed with nonwoven geotextile reinforcement was selected over the other available membrane materials because of its superior qualities with respect to constructibility, mechanical performance, durability, and prior use. HDPE geonet with preferential flow is a suitable drainage medium behind the membrane should a drained system be installed. The drained water can be discharged downstream through the structure or directly into the reservoir. Stainless-steel anchor bolts were selected to secure the perimeter profiles and vertical splice profiles to the concrete structure. Stainless-steel flat-bar profile sections with a minimum thickness of 1/4 in. (6 mm) were selected. Unless site-specific conditions dictate otherwise, the gasket should be open-cell neoprene, medium hardness, with a channel-shaped cross section.

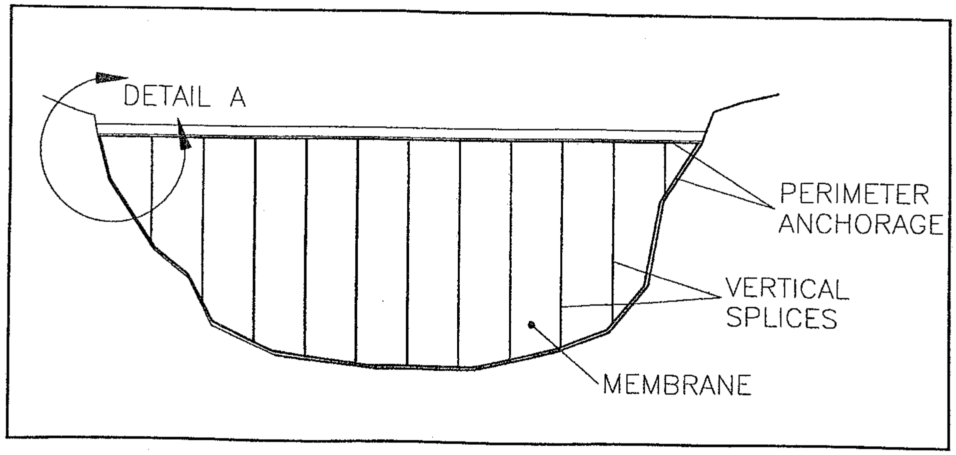


Figure 4. System general scheme

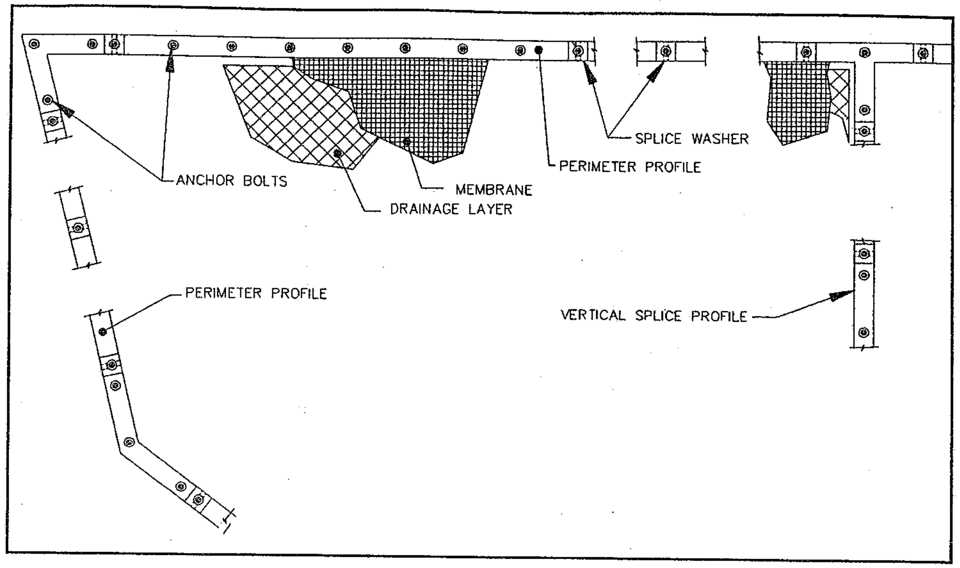


Figure 5. Assembly detail A from general scheme (Figure 4)

Constructibility Demonstration

The objective of this phase of the study was to demonstrate that the conceptual design could be practically installed underwater and that it provides a reliable barrier to moisture intrusion. The constructibility demonstration is described in detail by Marcy, Scuero, and Vaschetti (1996) and summarized in the following. The conceptual design and the constructibility demonstration are also summarized in a 9-min video report (REMR-CS-5).

The demonstration required a test structure that simulated a concrete hydraulic structure in need of repair. In an effort to make the constructibility demonstration comprehensive, the test structure was designed and built with features that replicate possible situations which could complicate the underwater installation of the geomembrane system. These features included rough surfaces, complex corners, depressions and protrusions, a V-shaped notch representing a construction joint, and various holes simulating discrete leakage points. The concrete structure was designed and constructed in the configuration of an L-shaped wall as shown in Figure 6.

A vacuum manifold was incorporated into the wall. The manifold creates a suction behind the membrane to simulate different hydrostatic heads and to test the efficiency of the system. The manifold is connected to 1-1/2 in. (38-mm) holes in the concrete which simulate points of discrete leakage through the structure.

After a successful installation in the dry (Figure 7), the wall was lifted with a crane and lowered into the test tank to a depth of 20 ft (6.1 m). Multiple installations were performed underwater. The profiles were used as templates for the anchor-bolt holes. Holes were drilled with a hydraulic hammer drill, and the bolt holes were cleaned with water and a plastic brush. Three types of anchor bolts were installed: torque-set wedge bolts, chemical anchors which use a two-part epoxy, and chemical anchors which use two-part epoxy and a glass encapsulated resin cartridge. Underwater epoxy was applied to smooth the rough concrete at the perimeter. The geonet drainage layer was positioned and secured to the wall with small expansion anchors. The gasket was placed over the anchor bolts along the perimeter, and the membrane sections were rolled down the face of the wall. Bolt holes were punched in the membrane by tapping the membrane over the bolts with a hammer. A second gasket layer was placed between overlapping membrane sheets at the vertical splices and perimeter seal. The profiles were placed on the wall and the anchor bolts were torqued to 35 foot-pounds (1.4 joules).

After all of the bolts were tightened, water was evacuated from behind the membrane using a hydraulic ejector. The combined effort of the pressure depression behind the membrane and the water depth resulted in a hydrostatic head of approximately 40 ft (12.2 m) of water. Two weeks after the vacuum was shut off, the

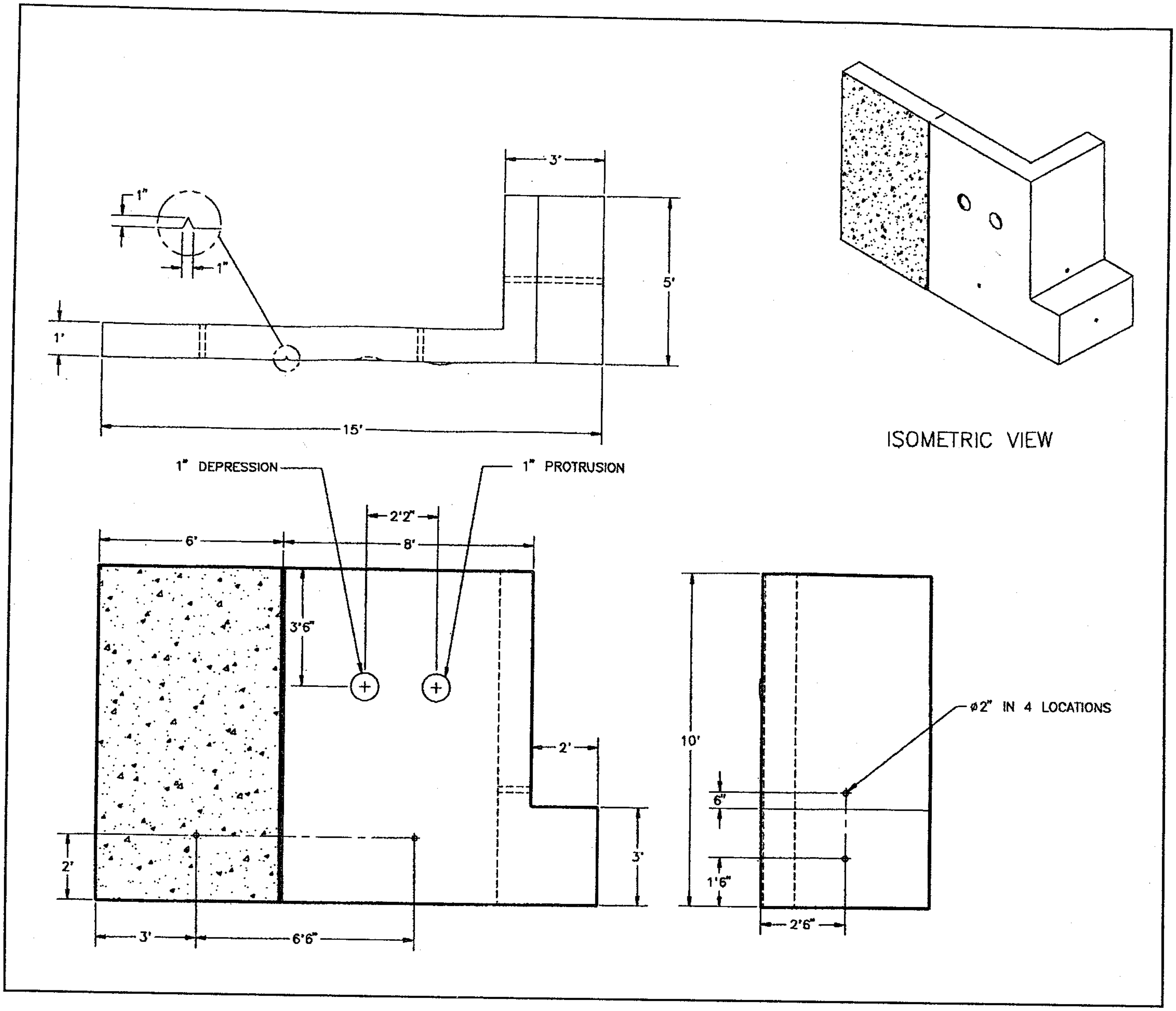


Figure 6. Test structure for constructibility demonstration

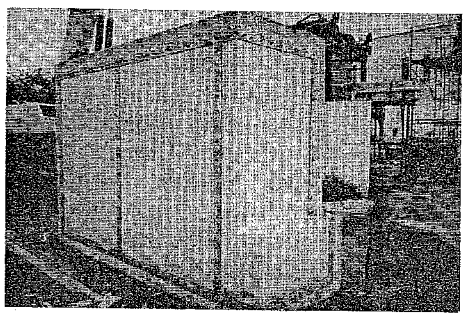


Figure 7. Completed dry installation

membrane remained tightly conformed to the wall (Figure 8), indicating that seepage through the repair system was extremely slow. During one of the underwater installations, five anchor bolts that used a combination of two-part epoxy and a glass-encapsulated resin cartridge were used. These five bolts loosened as the nuts were tightened. Failure was later attributed to the installation technique.

The system was tested to determine the effect of the defective bolts. As the ejector evacuated water behind the membrane, the membrane conformed tightly against the wall. With the ejector shut off, the membrane remained tightly conformed for approximately 2 hr. With the suction reapplied, divers were able to locate a small leak near the defective bolts by injecting dye into the water near the bolts. The defective bolts were removed, and replacement bolts were installed underwater. When the nuts were tightened, an efficient seal was achieved. This installation demonstrated that the system is repairable as well as constructible.

Results of the underwater installation dealt with two basic issues:

- Installation constructibility.
- Sealing efficiency of the system.

From the standpoint of installation feasibility, the underwater test demonstrated that ease of installation depended on the roughness of the substrate and the geometry of the structure. In rough areas, detailed procedures were required to ensure good

Figure 8. Membrane tightly conformed to the substrate

perimeter sealing, while on fairly smooth surfaces, installation of all components was easily accomplished. Experience in the dry had already shown this, but environmental conditions underwater amplified the problems associated with difficult features. This test mirrored experience in dry installations and showed that additional care is required to ensure good perimeter sealing when installations are performed in the more challenging underwater environment.

The research team believed that particular geometries of the structures, such as the complex corners, should be treated with a prefabricated sheet. Such scenarios will have to be addressed for each installation. Structures with complex shapes, such as intake towers, may require prefabricated membrane pieces to reduce installation time. Protrusions and depressions may constitute a design issue if they are very sharp. Experience in the dry, however, has proved that such irregularities can be adequately addressed with additional transition layers of nonwoven, needle-punched geotextiles.

Testing the system revealed that seepage through the repaired area was very slow. Even where five adjacent anchor bolts failed, leakage was slow enough to make detection of the leak difficult to notice even when dye was injected at the point of leakage. Although the leakage rate was not measured, the research team believed that it was slow enough to be negligible with respect to the requirements of most concrete hydraulic structures. The use of a drained system helped to locate and rectify the leak.

Conclusions

The successful underwater installation of the membrane repair system demonstrated the feasibility of the system. Although results of the demonstration were more qualitative than quantitative, it is evident that the system is constructible and will perform acceptably when designed and installed correctly.

Compared to dewatering of a structure for repair, a geomembrane system that can be installed underwater minimizes the impact of the repair on project operations such as hydropower generation, and recreation. Also, the underwater repair system eliminates the potentially adverse environmental impacts associated with dewatering of many structures.

Future Work

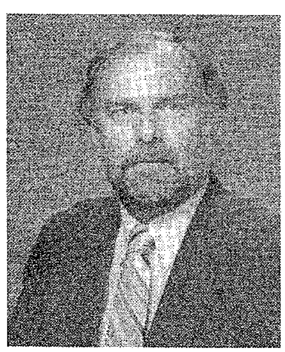
Pending availibiity of funding, current plans are to demonstrate the constructibility of the underwater repair system on a prototype structure. Candidate structures or appurtenances are being solicited. Anyone with a potential application for a repair of this type should contact Jim McDonald at (601) 634-3230.

References

Christensen, J. Chris, Marcy, Matthew M., Scuero, Alberto M., and Vaschetti, Gabriella L. (1995). "A Conceptual Design for Underwater Installation of Geomembrane Systems on Concrete Hydraulic Structures," Technical Report REMR-CS-50, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Marcy, Matthew M., Scuero, Alberto M., and Vaschetti, Gabriella L. (1996). "A Constructibility Demonstration of Geomembrane Systems Installed Underwater on Concrete Hydraulic Structures," Technical Report REMR-CS-51, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald, James E. (1993). "Geomembranes for repair of concrete hydraulic structures," *The REMR Bulletin*, 10(4), 1-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.



James E. McDonald *is a research civil engineer in the Concrete and Materials Division, Structures Laboratory, WES. He is the Problem Area Leader for the Concrete and Steel Structures portion of the REMR Research Program and is also the Principal Investigator for three REMR work units, including 32636, "New Concepts in Maintenance and Repair of Concrete Structures," McDonald has been involved with various aspects of concrete research for 35 years and has authored more than 100 technical reports and articles relating to concrete and other construction materials. He received his bachelor's and master's degrees in civil engineering from Mississippi State University. He is a registered Professional Engineer in the State of Mississippi.*

Repair of Failed Waterstops: An Evaluation of Products and Techniques

by Roy L. Campbell, Sr., U.S. Army Engineer Waterways Experiment Station

A survey of the condition of Civil Works structures to identify research needs (McDonald and Campbell 1985) revealed that one of the deficiencies frequently reported in periodic inspections was leakage at the monolith joints. Subsequently, a REMR technical report (McDonald 1986) compiled case histories documenting repairs to stop leakage at joints where waterstops had failed. Of the materials and techniques used to make these repairs, some appeared to be successful, while many others had failed. Most materials had been used in prototype repairs with limited or no laboratory evaluation because of a lack of appropriate test methods and equipment.

In response to the need for a means of testing and evaluating potential products and techniques to be used in the repair of waterstop failures, the secondary waterstop test apparatus (Figure 1) was designed and constructed at the U.S. Army Engineer Waterways Experiment Station (WES). The apparatus simulated a vertical joint in a mass-concrete structure by butting together the 1.2-m by 1.2-m (4-ft by 4-ft) faces of two 0.6-m- (2-ft-) wide concrete blocks. To accommodate grout-injection, grout-plug, and joint-sealant repair techniques, the joint model included injection ports, a vertical 152-mm- (6-in.-) diameter borehole at the midlength of the joint, and a simulated field-sawn slot (approximately 19-mm (3/4-in.) wide by 38-mm (1-1/2-in.) deep) at the upstream edge of the joint, respectively.

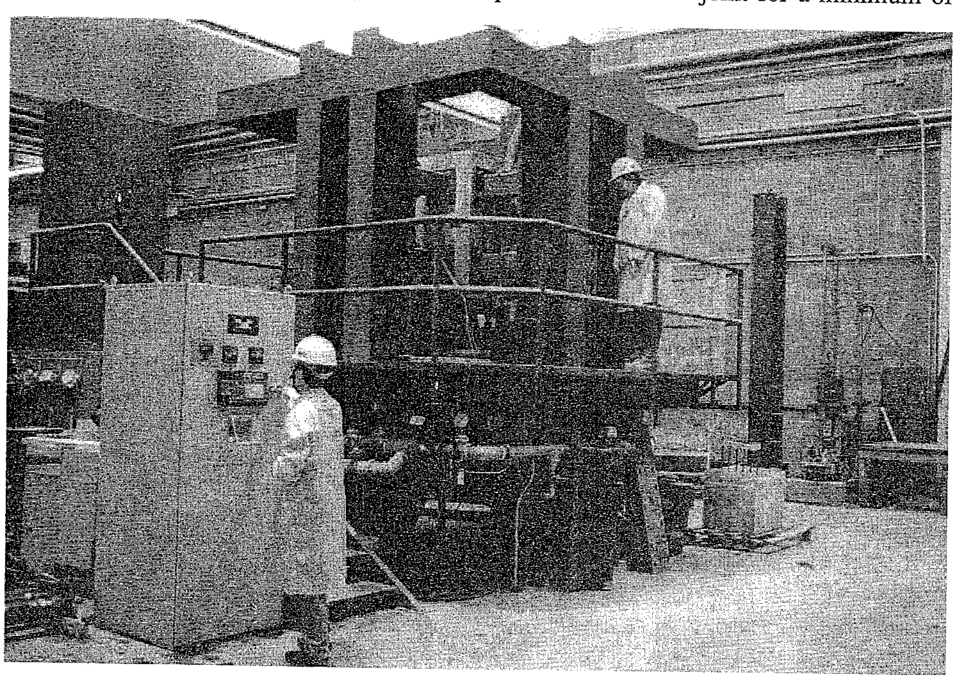


Figure 1. Secondary waterstop test apparatus

Repair Applications

Joint openings were preset for repair at either 1.6 mm (1/16 in.) or 12.7 mm (1/2 in.), except for prefabricated expansion joint repairs where a saw was used to create a 38.1-mm (1-1/2-in.) opening along the downstream vertical edge of the joint. For underwater applications, water was ponded within the joint for a minimum of 24 hr by either sealing the repair perimeter with polyurethane-soaked oakum or by enclosing the joint within a Plexiglas form.

Expansion joint

For prefabricated expansion joint repairs, an approximately 38-mm- (1-1/2-in.-) wide, 76-mm- (3-in.-) deep slot was formed by making a cut on each side of the downstream edge of the joint with a diamond-blade wall saw and breaking off the remaining cantilevered concrete sections between the cuts and joint. The remaining sawn surfaces were then sandblasted.

The neoprene expansion joint profile was cut slightly longer than the length of the vertical edge of the joint. The installation of an air valve and end caps made the profile an inflatable tube. The profile was inflated to an approximately 0.069-MPa (10-psi) pressure. Twice, the bond surfaces were roughened, and a neoprene conditioner was applied. The sawn concrete surfaces were twice wiped with cotton rags saturated with a concrete conditioner. The profile was deflated and an adhesive applied to the bond surfaces of the profile and concrete. The profile was then installed in the joint and reinflated. The adhesive was allowed to cure under pressure for a minimum of 24 hr.

Grout plug

The sleeve used to form the grout plug consisted of a plastic outer liner and fabric inner liner. A polyurethane grout was used to saturate the fabric inner liner by sealing and pulling a vacuum at one end of the sleeve while adding grout to the other end. The sleeve was placed in the open end of a pipe-shaped pressure vessel. The trailing end of the sleeve was opened, cuffed back, and secured with metal bands to the open end of the vessel. The pressure vessel was then placed in a pipe and suspended above the water-filled borehole at the midpoint of the joint. A combination of water and air pressure pushed the sleeve out of the pressure vessel into the borehole and at the same time inverted the sleeve. The applied pressure also forced the sleeve against the borehole face and the Plexiglas form sealing the bottom of the borehole. After installation, the grout sleeve was cut off just above the top of the joint, and an elastomeric filler grout was placed in the interior of the sleeve.

Injection grout

For chemical injection grout repairs, the outer perimeter of the joint between the upstream edge of the borehole and the upstream edge of the joint was sealed with polyurethane-soaked oakum, and the area within the perimeter was filled with water for a minimum of 24 hr. Polyurethane grout was then injected via a hand pump into the enclosed area through preformed ports in the concrete blocks. Water and entrapped gas within the perimeter were evacuated through a faucet-controlled port located in the top edge of the joint.

Joint Sealant

For repairs with a joint sealant, the simulated field-sawn slot in the upstream vertical edge of the joint was filled with sealant. For two of the repairs, the slot was partially filled with joint sealant, and the remainder was filled with a rigid epoxy top coat. The joint sealants were applied to dry concrete for some of the repairs and underwater for others. Metal saddles were installed at the ends of repairs when the joint opening was 12.7 mm (1/2 in.) to prevent the end of the sealant from being pushed into the joint by the water head.

Test Procedure

The upstream, top, and bottom edges of the joint were sealed by means of steel plates with rubber gaskets. Polyurethane strips were added at the gap between adjoining plates to complete the sealing of the applied-hydrostatic-head system. To reduce the potential for leakage around the ends of the repair, polyurethane strips were used to build up the ends of easily compressed joint sealant, grout plug, and prefabricated expansion joint repairs. The gaskets and strips formed compression seals as the steel plates were fastened to the concrete blocks with anchor bolts. The downstream edge of the joint was left open, and a catch trough was attached to direct water flow from the joint into a collection basin where flow rates were measured.

During testing, the water head was incrementally increased from 0 m (0 ft) to approximately 70 m (230 ft) at an approximate rate of 7 m (23 ft) every 3 min. The joint was then dewatered, and the plates were loosened. The one moveable concrete block (Figure 2) was repositioned to create a joint movement of 1.3 mm (0.05 in.). Tests were repeated for joint movements of 1.3, 2.5, 5.1, 7.6, and 10.2 mm (0.05, 0.10, 0.20, 0.30, and 0.40 in.), or until the repair failed.

Repair Performances

The performances of repairs are summarized in Tables 1 and 2 and in Figure 3.

Expansion joint

In test 22, a prefabricated W-type expansion joint with the V-shaped edges of its profile pointing downstream performed better than any of the other repairs made at a joint opening of 12.7 mm (1/2 in.) or greater. The profile was installed in a 38.1-mm (1-1/2-in.) opening along the downstream edge or joint (Figure 3). Testing of the repair was stopped because of failure of the confinement vessel seals at a joint movement of 7.6 mm (0.30 in.) and a water head of 55.8 m (183 ft).

Earlier, M- and W-type joint profiles had been installed and tested with the V-shaped edges of the profiles pointing upstream. Their performances (tests 20 and 21, respectively) were less than expected.

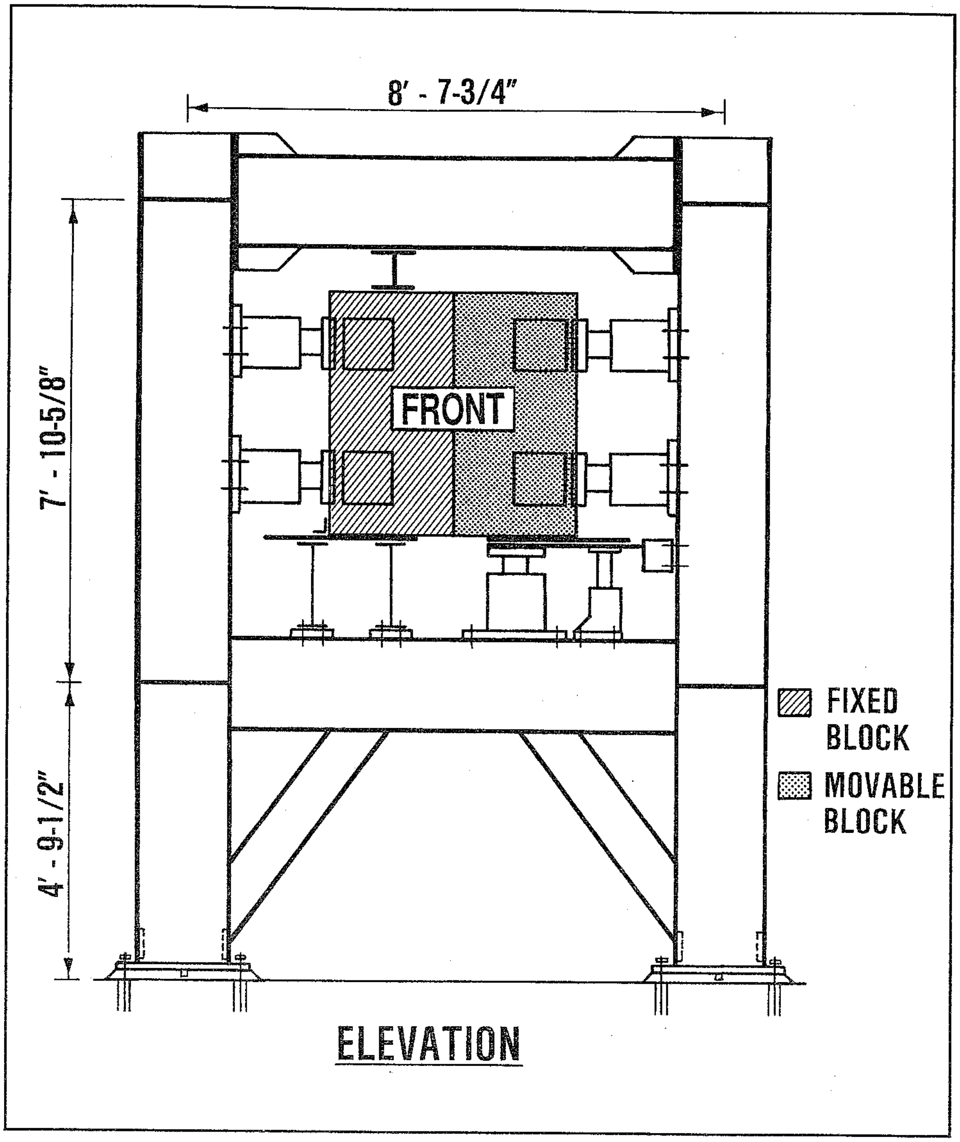


Figure 2. Simulated joint movement system (secondary waterstop test apparatus)

Table 1. Performance of Chemical-Grouts								
		Application		Initial Seepage		Repair Failure		
Repair Material	Test No.	Joint Opening mm (in.)	Injection Pressure MPa (psi)	Joint Movement mm (in.)	Water Head m (ft)	Joint Movement mm (in.)	Water Head m (ft)	Description
Hydrophobic Polyurethane Injection Grout	1	1.6 (1/16)	≤ 0.14 (20)	0	11.9 (39)	0	53.3 (175)	Grout debonded
	2	1.6 (1/16)	≥ 0.69 (100)	2.5 (0.10)	1.5 (5)	2.5 (0.10)	18.0 (59)	Grout debonded
Hydrophilic Polyurethane Injection Grout	3	1.6 (1/16)	≥ 0.69 (100)	1.3 (0.05)	63.7 (209)	2.5 (0.10)	60.4 (198)	Grout debonded
Hydrophobic Polyurethane Injection Grout	4	1.6 (1/16)	≥ 0.69 (100)	2.5 (0.10)	1.5 (5)	2.5 (0.10)	8.5 (28)	Grout debonded
Hydrophilic Polyurethane Injection Grout	5	1.6 (1/16)	≥ 0.69 (100)	1.3 (0.05)	64.9 (213)	5.1 (0.20)	58.8 (193)	Grout debonded
	6	12.7 (1/2)	≥ 0.69 (100)	1.3 (0.05)	24.1 (79)	2.5 (0.10)	30.5 (100)	Grout debonded
Hydrophilic Polyurethane Injection Grout	7	12.7 (1/2)	≥ 0.69 (100)	2.5 (0.10)	33.2 (109)	2.5 (0.10)	48.5 (159)	Grout debonded
Hydrophobic Polyurethane Injection Grout	—	12.7 (1/2)	≥ 0.69 (100)	—	—	—	—	Grout set before joint could be filled
Hydrophilic Polyurethane Injection Grout	8	12.7 (1/2)	≥ 0.69 (100)	1.3 (0.05)	22.6 (74)	1.3 (0.05)	34.1 (112)	Grout debonded
Hydrophilic Polyurethane Grout Plug	9	1.6 (1/16)	~ 0.048 (7)*	0	0.3 (1)	2.5 (0.10)	57.3 (188)	Grout sleeve debonded
Hydrophilic Polyurethane Injection Grout and Urethane Joint Sealant	10	1.6 (1/16)	≥ 0.69 (100)**	7.6 (0.30)	38.1 (125)	10.2 (0.4)	52.7 (173)	Grout debonded and sealant ruptured

*Pressure applied within the sleeve to compress grout and sleeve against borehole face.
**Joint sealant applied to dry concrete surface.

The earlier M-type joint profile remained inflated during testing. The other profiles were deflated after the bond adhesive had cured.

Grout plug

During installation of the grout plug, the pressure within the core of the repair sleeve was intended to be approximately 0.103 MPa (15 psi); however, the pressure could be maintained only at approximately 0.048 MPa (7 psi). When the repair was dewatered in preparation for testing, it was observed that the polyurethane fabric was highly compressible and had a large cell structure. Water could be easily squeezed out of the polyurethane fabric by pressing with one's finger. When water was added to the top of the repair, the water would travel the length of the repair and exit as droplets out the bottom.

Initial seepage through the grout-plug repair (test 9) was observed at no joint movement and a water head of 0.3 m (1 ft). The repair failed at a joint movement of 2.5 mm (0.10 in.) and a cumulative water head of 197.5 m (648 ft).

Injection grout

The chemical grout products tested were all polyurethanes. Testing was initiated on a repair in which a grout had been injected into a joint opening of 1.6 mm (1/16 in.) at pressures of 0.138 MPa (20 psi) and less. The repair (test 1) performed poorly. The performances of subsequent repairs were greatly improved by increasing the injection pressures to 0.69 MPa (100 psi) and greater.

Repairs made at a joint opening of 1.6 mm (1/16 in.) performed similarly in that initial seepage was generally observed at a cumulative water head near 140 m (460 ft). The repairs made with hydrophilic polyurethanes (tests 3 and 5) performed better than those made with hydrophobic polyurethanes (tests 2 and 4).

Repairs made at a joint opening of 12.7 mm (1/2 in.) also showed the tendency for better performance by the hydrophilic-polyurethane (tests 6 and 7) than performance by the hydrophobic-polyurethane (test 8).

One repair was made using a high-viscosity (3,200 cps) grout. The grout was so difficult to inject via hand pump that less than half of the joint was filled before the material set. The repair and test were discontinued. The viscosity of all other grouts was 1,200 cps or less.

Table 2. Performance of Joint-Sealant and Expansion-Joint Materials

Repair Material	Test No.	Application		Initial Seepage		Repair Failure		
		Joint Opening mm (in.)	Concrete Surface	Joint Movement mm (in.)	Water Head m (ft)	Joint Movement mm (in.)	Water Head m (ft)	Description
Polyurethane Joint Sealant	11	1.6 (1/16)	Dry	1.3 (0.05)	10.7 (35)	5.1 (0.20)	61.0 (200)	Sealant debonded
	12	1.6 (1/16)	Submerged	2.5 (0.10)	1.2 (4)	5.1 (0.20)	9.1 (30)	Sealant debonded
	13	12.7 (1/2)	Submerged	0	1.2 (4)	1.3 (0.05)	46.3 (152)	Sealant debonded
Silicone Joint Sealant	14	1.6 (1/16)	Dry	2.5 (0.10)	32.3 (106)	10.2 (0.40)	63.4 (208)	Sealant debonded
Polycarbonate Joint Sealant	15	1.6 (1/16)	Submerged	0	0.9 (3)	0	63.0 (223)	Sealant yielded
Polysulfide Joint Sealant	16	1.6 (1/16)	Dry	2.5 (0.10)	1.5 (5)	10.2 (0.40)	5.5 (18)	Sealant debonded
Urethane Joint Sealant	17	1.6 (1/16)	Dry	2.5 (0.10)	33.5 (110)	10.2 (0.40)	60.4 (198)	Sealant debonded
Polyurethane Joint Sealant and Rigid Epoxy Top Coat Composite	18	1.6 (1/16)	Submerged	1.3 (0.05)	24.7 (82)	5.1 (0.20)	45.1 (148)	Sealant and top coat debonded from concrete
	19	12.7 (1/2)	Submerged	0	1.5 (5)	1.3 (0.05)	0.6 (2)	Sealant and top coat debonded from concrete
M-Type Expansion Joint with V-Shaped Edge Pointed Upstream	20	38.1 (1-1/2)	Dry	0	33.8 (111)	1.3 (0.05)	59.1 (194)	Profile material split and debonded
W-Type Expansion Joint with V-Shaped Edge Pointed Downstream	21	38.1 (1-1/2)	Dry	0	1.2 (4)	0	53.6 (176)	Profile debonded
W-Type Expansion Joint with V-Shaped Edge Pointed Downstream	22	38.1 (1-1/2)	Dry	5.1 (0.20)	50.9 (167)	7.6 (0.30)*	55.8 (183)*	*Test stopped due to failed containment seal

Joint sealant

For repairs made at a joint opening of 1.6 mm (1/16 in.), joint sealants (tests 11, 12, 14, 16, 17, and 18) generally performed better than injection grouts (tests 2, 3, 4, and 5). Joint sealants applied at a joint opening of 12.7 mm (1/2 in.) performed very poorly (tests 13 and 19). Seepage through the wider repairs was initially observed at no joint movement and water heads of 1.5 m (5 ft) and less.

It was noted that joint sealants applied to dry concrete surfaces (tests 11, 14, 16, and 17) performed better than those applied underwater (tests 12, 15, 18, and 19).

A repair made with a joint sealant that remained pliable after set performed very poorly (test 15). After curing underwater for 7 days, the product was observed to be nearly as pliable as it was during its application. The surface of the sealant was no longer adhesive; however, the material just beneath the surface appeared to be uncured sealant and was adhesive.

Repairs made with a rigid epoxy top coat over a joint sealant (tests 18 and 19) generally performed poorer than repairs made with only the joint sealant (tests 12 and 13, respectively).

Injection grout and joint sealant

A repair using both a polyurethane injection grout and a joint sealant (test 10) performed better than any of the other repairs made at a joint opening of 1.6 mm (1/16 in.). Results of the test showed no leakage up to a joint movement of 7.6 mm (0.30 in.) and a water head of 38.1 m (125 ft). The magnitude of the leakage was not measurable for water heads up to

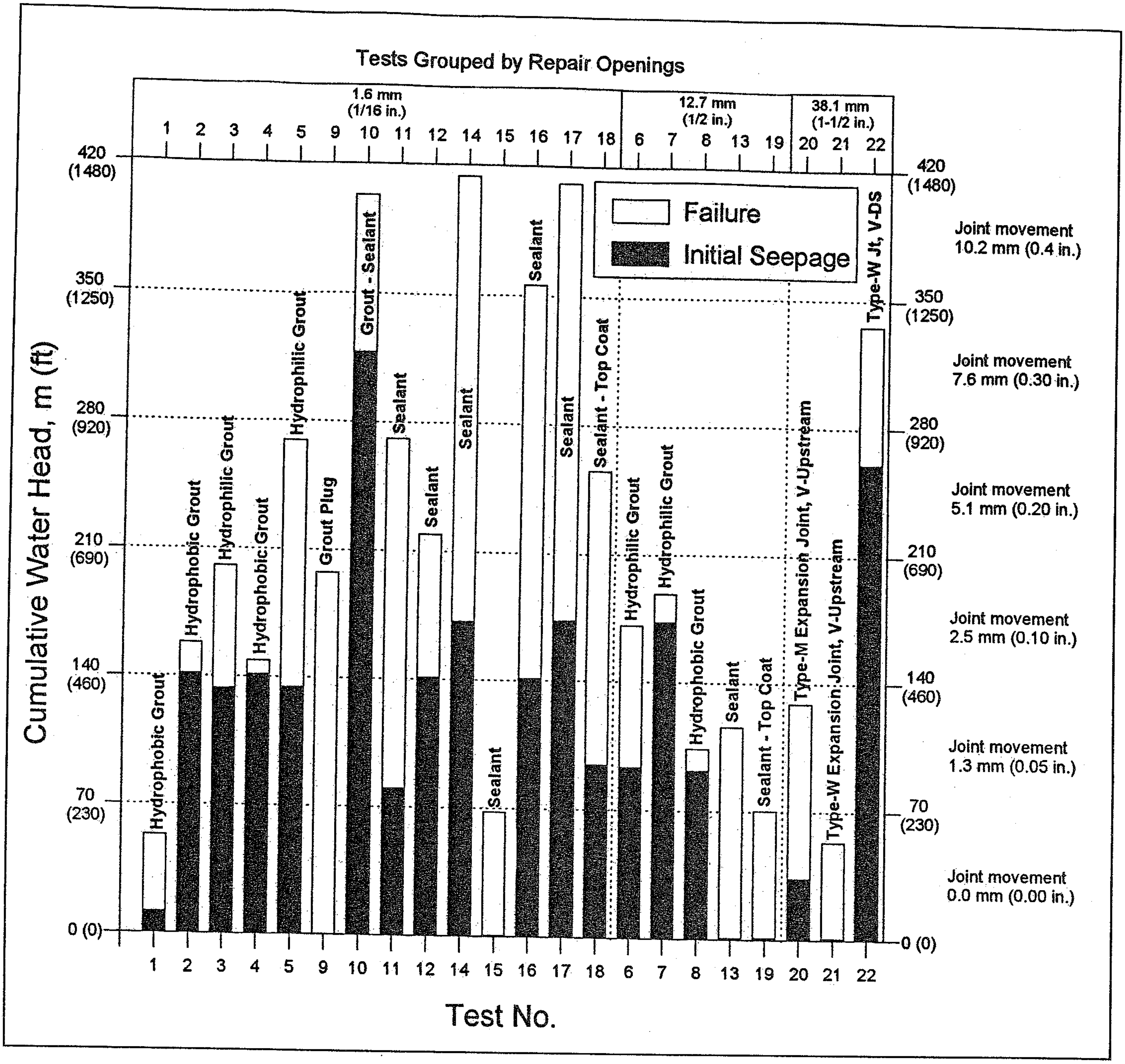


Figure 3. Cumulative sum of water heads for joint movements at times of initial seepage and failure of the repair (Tables 1 and 2 for repair details)

70.1 m (230 ft). The repair failed at a joint opening of 10.2 mm (0.40 in.) and a water head of 52.7 m (173 ft).

Conclusions

Expansion joint

Prefabricated expansion joints are generally expected to perform well for repair widths up to 38.1 mm (1-1/2 in.) where maximum joint movements are 5.1 mm (0.20 in.) and less. The expansion-joint profile must be installed with the V-shaped edges of its cross-sectional area pointed in the direction of flow (downstream). Special techniques will have to be developed to prevent premature leakage from occurring around the ends of the expansion-joint repairs.

Grout plug

The chemical grout-plug repair technique could not be properly tested as the test apparatus was not set up to provide confining pressures within the sleeve greater than the applied water head. For the repair technique to be successful in a field application, the pressure head within the sleeve must be greater than that of the water head being plugged and must be maintained throughout the life of the repair; otherwise, the sleeve will likely debond and leak.

Injection grout

Chemical injection grouts are generally expected to perform well for repair widths up to 12.7 mm (1/2 in.) where maximum joint movements are 1.3 mm (0.05 in.) and less. Overall, grouts that are hydrophilic polyurethanes are expected to perform better than those that are hydrophobic polyurethanes. Optimum repair performance can be expected when grouts are injected at or near the maximum pressures allowed by the repair conditions.

Joint sealant

Joint-sealant repairs to the upstream face of joints are generally expected to perform well in reducing leakages through joints that are 1.6 mm (1/16 in.) and less in width where maximum joint movements are 2.5 mm (0.1 in.) and less.

Joint-sealant products that are recommended for application to wet concrete surfaces will likely perform better when applied to a dry surface. Extremely pliable sealants are not applicable for repairing leaks in hydraulic structures because they yield under small water heads until failure of the repair occurs. The use of an exterior rigid coating over a joint sealant does not appear to increase the repair performance and, therefore, is not an applicable means of repair. Special techniques will have to be employed to prevent premature leakage from occurring around the ends of sealant repairs.

Injection grout and joint sealant

Grout-sealant repairs are generally expected to perform well in reducing leakages through joints that are 1.6 mm (1/16 in.) and less in width where maximum joint movements are 7.6 mm (0.3 in.) and less.

Recommendations

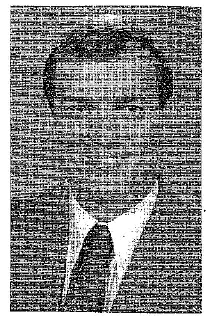
Because the tests conducted were of a short duration, additional study is needed to determine the effects of creep and aging on repair performances. Supplementary testing in the form of bond tests should be considered to further evaluate the concluded differences in performances of hydrophilic- and hydrophobic-polyurethane injection grouts.

For additional information, contact Roy Campbell, Sr., at (601) 634-2814, or e-mail to Campber@ex1.wes.army.mil.

References

McDonald, J.E. (1986). "Repair of Waterstop Failures: Case Histories" Technical Report REMR-CS-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald, J.E., and Campbell, R.L., Sr. (1985). "The Condition of Corps of Engineers Civil Works Concrete Structures," Technical Report REMR-CS-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.



Roy L. Campbell, Sr., is a research civil engineer in the Concrete and Materials Division, Structures Laboratory, WES. He received his B.S. degree in civil engineering from Mississippi State University. Campbell is the database manager for the REMR Repair Products and Bibliography Databases.

Annual REMR-II Field Review Group Meeting Scheduled for June

The ninth REMR-II Field Review Group (FRG) Meeting will be held in the Washington, D.C. area on June 3, 1997. During this meeting, the FRG members will review the progress of ongoing work units. The meeting will be open to the public as well as to Corps personnel involved in the repair, rehabilitation, maintenance, and rehabilitation of the Nations' infrastructure. Representatives from Corps Districts and Divisions are encouraged to attend. For additional information, contact Lee Byrne by calling (601) 634-2587 or by e-mailing to byrnel@mail.wes.army.mil.

REPAIR·EVALUATION·MAINTENANCE·REHABILITATION

REMR

RESEARCH PROGRAM

Featured in this Issue

The REMR Bulletin is published in accordance with AR 25-30 as one of the information exchange functions of the Corps of Engineers. It is primarily intended to be a forum whereby information on repair, evaluation, maintenance, and rehabilitation work done or managed by Corps field offices can be rapidly and widely disseminated to other Corps offices, other U.S. Government agencies, and the engineering community in general. Contribution of articles, news, reviews, notices, and other pertinent types of information are solicited from all sources and will be considered for publication so long as they are relevant to REMR activities. Special consideration will be given to reports of Corps field experience in repair and maintenance of civil works projects. In considering the application of technology described herein, the reader should note that the purpose of *The REMR Bulletin* is information exchange and not the promulgation of Corps policy; thus guidance on recommended practice in any given area should be sought through appropriate channels or in other documents. The contents of this bulletin are not to be used for advertising, or promotional purposes, nor are they to be published without proper credits. Any copyright material released to and used in *The REMR Bulletin* retains its copyright protection, and cannot be reproduced without permission of copyright holder. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. *The REMR Bulletin* will be issued on an irregular basis as dictated by the quantity and importance of information available for dissemination. Communications are welcomed and should be made by writing U.S. Army Engineer Waterways Experiment Station, ATTN: Lee Byrne (CEWES-SC-A), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199, or calling (601) 634-2587; e-mail: byrnel@mail.wes.army.mil.

Robert W. Whalin

ROBERT W. WHALIN, PhD, PE
Director

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

OFFICIAL BUSINESS
CEWES-SC-A