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The REMR Bulletin

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

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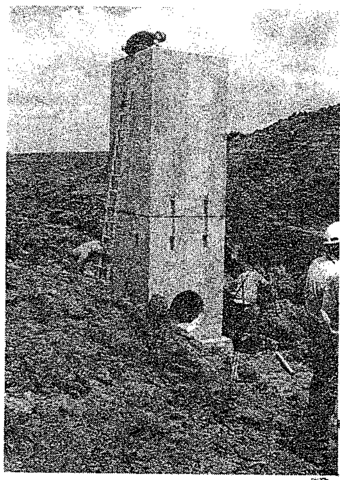

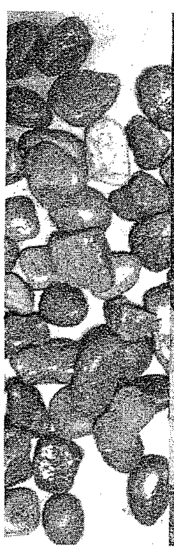

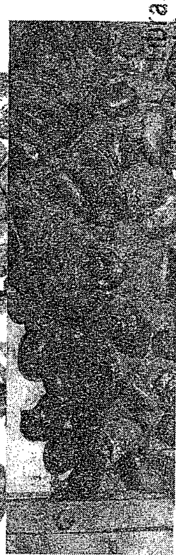

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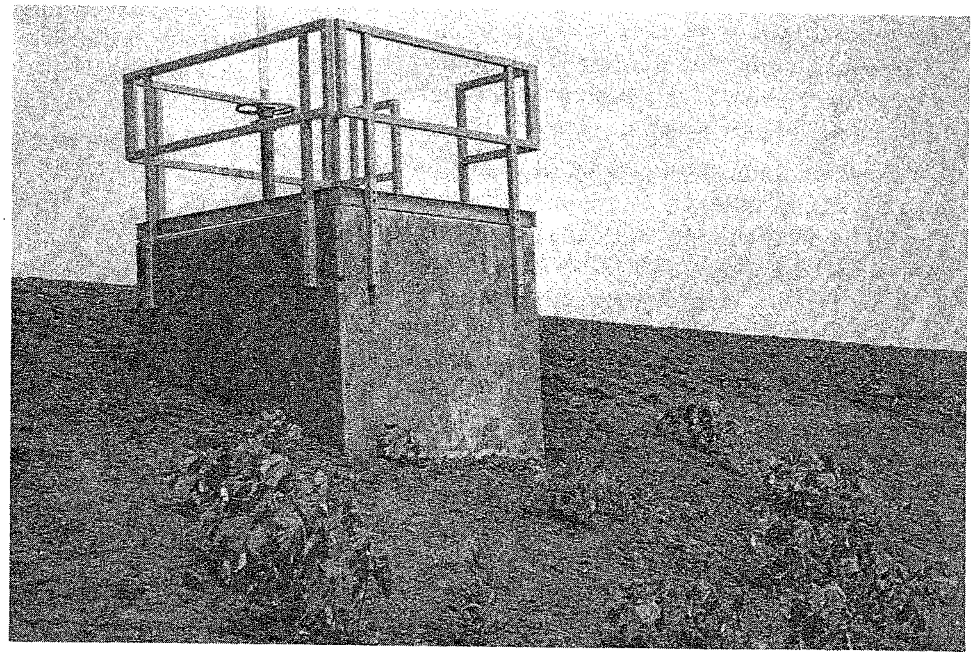


Gatewell at Chouteau Island

Innovative Products and Procedures Used on Chouteau Island Levee Relocation

by
Tamara L. Atchley
US Army Engineer District, St. Louis

Several gravity drainage structures exist within the flood control system of the St. Louis District. Most of these structures consist of bituminous-coated, corrugated metal pipe (CMP) (for drainage through the levee) and a concrete gatewell (a manhole which houses a sluice gate). During periods of low flow, the gate remains open to allow gravity drainage of the protected side of the levee. During flood conditions, the gate is closed to prevent backwater flow through the pipe and into the protected area.

Most of these structures have been in service for many years, and now some of them are experiencing problems. Problems with drainage pipes stem from loss of soil support around the pipes and separation of pipe joints. Pipe failure is the eventual result. Another problem is the routine maintenance required by the concrete gatewells in these structures. After being built by the Corps, the structures have become property of the local levee district. Because the local district has limited funding, it needs structures that are as maintenance free as possible.

HIGH DENSITY POLYETHYLENE PIPE

Chouteau Island separates the Mississippi River and the Chain of Rocks Canal just upstream of Lock and Dam No. 27 at Granite City, Illinois. During the October 1986 flood of the Mississippi River, it became apparent that it would be necessary to relocate a portion of the existing levee on Chouteau Island. The new section of levee required a gravity drainage structure with a concrete gatewell and a relatively short gravity drainage pipe.

The levee relocation project allowed the District to gain experience with new procedures and products. The products included high-density polyethylene (HDPE) pipe and fiber-reinforced plastic (FRP). HDPE is corrosion resistant and allows more effective joint connections than the CMP. The FRP products are highly resistant to corrosion and eliminate the need for periodic maintenance, such as the painting required even for galvanized metal. A sluice gate made from a synthetic material was also selected for use in the structure.

Problems associated with CMP failures in the St. Louis District resulted from the separation of the pipe joints. If joints are not watertight, seepage may erode the soil surrounding the pipe and leave a void. Even if a joint is adequate initially, seepage through the soil may carry away the material around the pipe, causing the joint to open and compounding the problem. Also, excessive settlement of the supporting soils and inadequately compacted backfill can contribute to joint failures. In any case, because CMP is flexible, it depends on the surrounding soil for its stability; once the soil support is lost, the pipe has a tendency to collapse.

The HDPE pipe selected by the contractor has a corrugated exterior that is beneficial for compressive strength. With proper installation, fill heights of more than 50 ft are possible. HDPE is highly resistant to chemical attack and corrosion, an important feature when runoff contains high concentrations of fertilizers and other agricultural chemicals. It is also resistant to UV radiation and abrasion. It is lighter than clay, concrete, or CMP and, therefore, easier to handle and install. It does not burn at temperatures below 700° F; grass-fire temperatures are generally below this limit.

Care was taken to ensure proper installation of the HDPE pipe. A minimum compaction of 95 percent of the maximum dry density as determined by the Standard Proctor Test was required for areas within 4 ft of the pipe. Full-time inspection and testing by an independent testing firm was also included in the project specifications. Pipe sections were joined with a split-ring coupling band with a neoprene gasket. The band, which is corrugated to match the pipe, fits over an equal number of corrugations on each side of the joint. Coating the areas around the joints with a mastic and wrapping them with geotextile should ensure soundness of the joint and prevent the infiltration of soil fines into the pipe, if a leak were to develop at a joint (Figure 1).

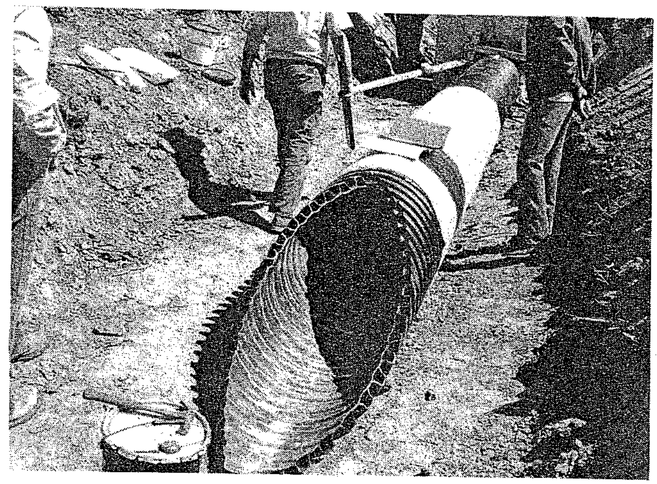


Figure 1. Joint coated with mastic and wrapped with geotextile to prevent infiltration of soil fines

PRECAST GATEWELL

The original design of the Chouteau Island gatewell called for a cast-in-place structure; however, the contractor submitted for approval a precast, two-section gatewell with the same dimensions and reinforcement as the original structure. The precast plan was approved.

The contractor used a concrete mixture with a high cement content and a water-reducing admixture. The mixture yielded a 6-day compressive strength of 4,651 psi and a 14-day strength of 5,235 psi for the base pour and a 7-day strength of 4,421 psi for the wall pour. Preassembling attachments at the precast yard, including the gate, ladders, guardrails, grating, and stoplog angles, provided an opportunity for problem correction before delivery. Also, the field team was able to have a "practice" installation (Figure 2).

The structure was separated into its two components for shipping. Attachments that spanned the

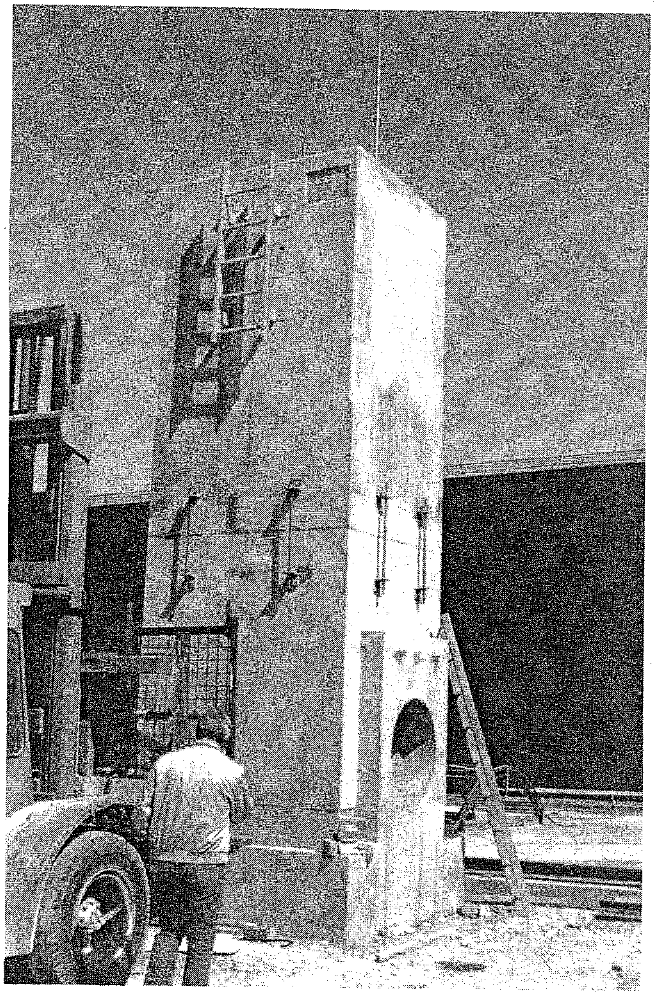


Figure 2. Two-section gatewell, preassembled at precast yard

joint in the gatewell and pieces that could be easily damaged during travel were removed for shipment. The actual installation of the gatewell presented no problems.

GATE

Both a cast-iron gate and an alternative synthetic gate were specified for this project. The contractor chose to furnish a synthetic gate with a steel matrix encapsulated in a high-density polymer. The steel frame is epoxy coated; seals, wedges and side guides are of high-molecular-weight polyethylene, medium-density polyolefin. Because these materials are considerably lighter than cast-iron, there is less friction between the gate leaf and seal. Also, the gate can be raised and lowered with a smaller operator (physical size and horsepower), is more maneuverable and easier to install, and requires fewer personnel for installation than for a cast-iron gate.

ATTACHMENTS

Usually, metal attachments are used for gatewell structures. Because of their exposure to the outside environment, these metal attachments require periodic inspection and routine maintenance to ensure their structural soundness. To provide a structure as maintenance free as possible, attachments chosen for the Chouteau Island gatewell are FRP products.

A proprietary combination of fiberglass reinforcements and a thermosetting polyester resin system was used for the ladders, stoplog angles, and guardrails. This material is corrosion resistant, nonconductive (thermally and electrically), nonmagnetic, strong, and lightweight. Structural shapes are formed by pulling fiberglass rovings and reinforcement through a curing and forming die to create the final shape. Color is throughout the material so painting is not needed. It is necessary, however, to seal cut edges or holes to maintain the integrity of the material.

The FRP products were shipped preassembled and ready for installation, thus eliminating the need for highly skilled labor or special equipment.

FRP ladders and guardrails used in the project conformed to OSHA standards. Stainless steel standoff clips hold the ladders to the gatewell and a pin and keyway arrangement prevents rotation of the rungs. Sections that formed the guardrail were joined by epoxy bonding the ends of the tubing to an internal joint coupling piece and by riveting each side of the joint through the tubing and joint coupler with nylon rivets.

Stoplog angles were installed to accommodate stoplogs in the event the gate is inoperable during flood conditions. FRP angles guided the lower stoplogs during installation and now hold the upper stoplogs in place. Stainless steel anchors hold the angles to the gatewell.

A 1-1/2- by 1-1/2-in. mesh pattern was selected for the grating so that it could be installed in either direction. Because this grating is more flexible than metal grating, design was governed by deflection criteria. The result is a very high factor of safety against failure loads. The grating also has a skid-resistant grit surface. It is supported on three sides by an FRP, one-piece combination angle and anchor embedded in the top of the gatewell walls (Figure 3).

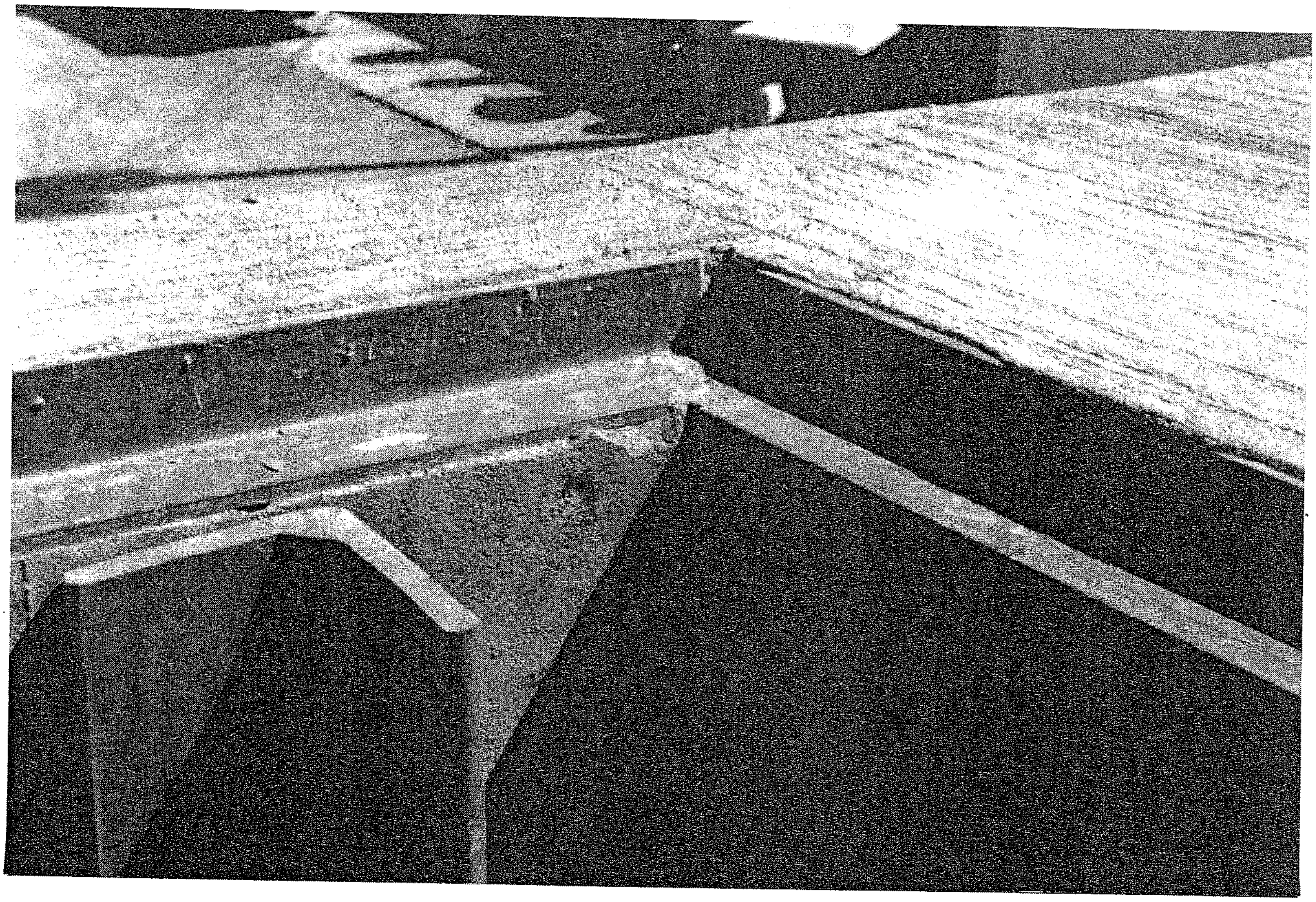


Figure 3. Top of gatewell showing fiberglass curb angle and stoplog guides

RECOMMENDATIONS AND CONCLUSIONS

The Chouteau Island Levee relocation was completed in the spring of 1988. It may take years before the performance of the new products used for the project can be evaluated. However, at this time, the present and potential benefits suggest that use of these products may play an important role in future district construction.

For further information, contact Tamara Atchley at the St. Louis District at (314) 263-5708.

Tamara Atchley is a structural engineer in the Structural Design Section of the St. Louis District. She received a B.S. in civil engineering from the University of Illinois and an M.S. in civil engineering from Southern Illinois University at Edwardsville. She was the structural engineer for the Chouteau Island Levee Relocation project. At present, she is the Project Manager in St. Louis for Reach 4 of the Arizona Canal Diversion Channel, a flood control project in Phoenix.

Cutoff Wall Contruction to Upgrade Mud Mountain Dam

by
K. D. Graybeal

INTRODUCTION

Mud Mountain Dam is located on the White River 5 miles upstream and southeast of Enumclaw, Washington. The drainage area above the dam is 400 square miles. A single-purpose flood control project constructed in 1941, it is operated by the Seattle District, US Army Corps of Engineers, to alleviate flooding in the lower Puyallup Valley. Water storage serves only for flood control and the reservoir normally fluctuates at relatively low levels. The highest reservoir elevation up to 1988 is 1,150 ft, which is 100 ft below the top of the dam. The dam is a 400-ft- high earth and rockfill structure with a crest length of 700 ft. The dam top is at elevation 1,250 ft. There is a concrete lined, chute-type uncontrolled spillway with a crest elevation of 1,215 ft. Two tunnels discharge normal flows. The general configuration of the project is shown in Figure 1.

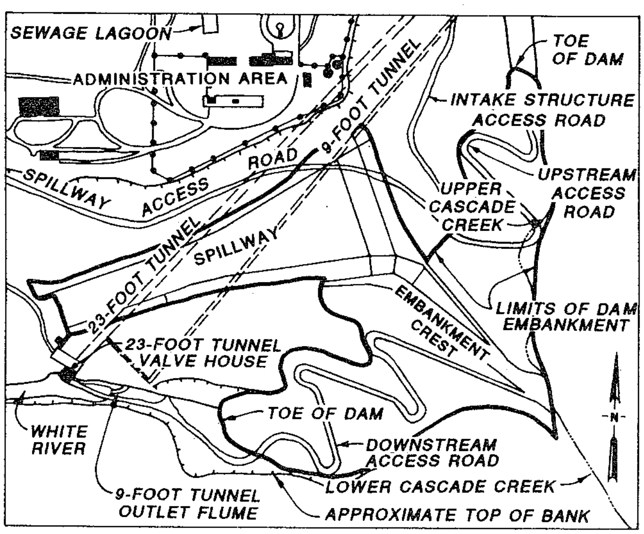


Figure 1. Mud Mountain Dam existing project

SITE CONDITIONS

The existing earth and rockfill dam extends across a narrow canyon with steep volcanic-type rock walls more than 200 ft high on both sides. The rock is jointed and faulted, moderately hard to hard andesite volcanic agglomerate bedrock consisting of angular blocks of andesite within a welded ash and tuff matrix. Unconfined compressive strengths range to more then 20,000 psi. Inclusions of weak sediment material occur in the bedrock of the canyon floor.

Above the rock canyon, the dam foundation is a complex assemblage of mudflow and glacial deposits, known as the Mud Mountain Complex (MMC). The MMC is an assemblage of relatively competent mudflows generally composed of boulders, cobbles, gravels, sands, silts, and hard, highly plastic clays.

DESCRIPTION OF DAM EMBANKMENT

The dam was constructed in 1941 with a central impervious core and with symmetrical upstream and downstream transition zones and rock shells (Figure 2).

The core consists of a mixture of sand, gravel, and glacial till, placed in lifts and compacted by sheep's foot rollers. Construction information and later testing suggest that the core borrow area was initially gravelly but became progressively more sandy as construction progressed. Thus, the upper portion of the core, above approximate elevation 1,050 ft, is predominately a silty sand with variable gravel content. Before placement of embankment materials, rock overhangs were scaled back on the canyon walls and cavities were filled with dental concrete. Some grouting of cavitites behind dental concrete was done; however, no grout curtain was constructed and there is no record of any "slush grouting" or other treatment of the rock surface.

Transition material is crushed diorite rock (4 in. minus), placed in lifts and compacted by tracked dozer. The transition zone does not provide adequate filter protection for a significant portion of the core material.

Rockfill shell material is quarry run rock, placed by dumping and sluicing with water.

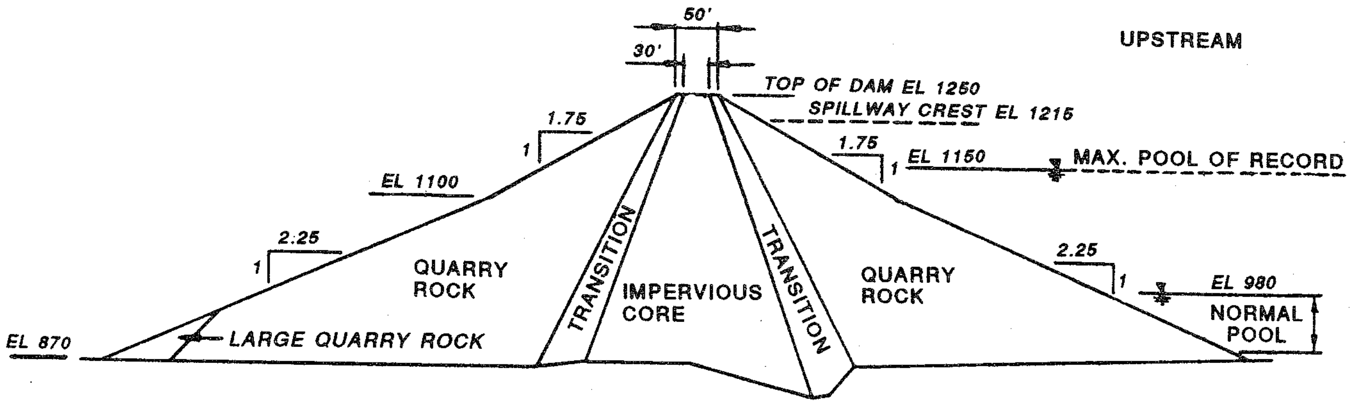


Figure 2. Mud Mountain Dam embankment section

PROBLEM WITH DAM CORE

Seepage through the dam was not a recognized problem before 1985 and no seepage has ever been visible at the downstream toe area. However, the reservoir has never been filled to its maximum level, and normally fluctuates at low pool levels below elevation 980 ft. Examination of the record of a single piezometer in the dam core, in relation to the pool, during the period 1982 to 1983, showed potential core deterioration. This prompted additional geotechnical investigations.

GEOTECHNICAL INVESTIGATIONS

In 1985 and 1986, eighteen borings were drilled in the dam to search for possible problem zones within the core. Then vibrating wire piezometers and multiple-staged open-tube piezometers separated by grout or bentonite seals were installed in the perforated drill casings.

Cable-tool methods were generally used for the dam core drilling to minimize potential for hydraulic fracturing of the core.

Geotechnical investigation of the dam core showed the following:

- There are many zones throughout the core where loose or soft materials were indicated by ease of drilling, ease of sampler driving, and by advance of the drill casing under its own weight during cable-tool drilling. This condition is most pronounced in the uppermost, more sandy portion of the core, which has had minimal exposure to the pool.
- Near the base of the dam where the core material is more gravelly, and within the range of normal reservoir fluctuations, the core contains many zones composed of a clean gravel matrix, with few fines. This condition is most pronounced adjacent to the steep canyon walls. These zones take on water when encountered by drilling during low reservoir levels, and produce water into the hole during higher reservoir levels. Drive samples taken from these zones resemble core materials from which the fines have been removed by washing.

PIEZOMETER OBSERVATIONS

Piezometer Inflow Test: To gain more information on the suspected core defects, water was introducted into selected piezometers. Tests were generally conducted with nominal head (about 50 ft) to minimize danger of hydrofracturing. A series of tests and observations revealed hydraulic communication through apparent core defects at various levels and locations throughout the core of the dam. Because of the observed separation of water levels in piezometer stages and the observed general downward gradient between stages, it was conclusively shown that during the tests, piezometer response represented flow through the core. Rapidity and magnitude of piezometer responses suggest that hydraulic communication occurs through cracks or similar features within the core, which is consistent with information derived and interpreted from the exploration program.

Piezometer Response to Pool: Piezometer response during normal pool fluctuations is abnormally large and rapid, particularly in those piezometers located next to the steep rock canyon walls. For example, the bottom stage of a piezometer located adjacent to the left canyon wall in the downstream third of the core showed a rapid 107-ft response to a reservoir drawdown of 132 ft in June 1986.

INTERPRETATION AND CONCLUSIONS

Interpretation of all data leads to the conclusion that settlement and arching of the dam core within the narrow, steep rock canyon has produced loose zones, and/or cracks in the core. The lower, more gravelly portion of the core is subjected to flushing action from repeated pool fluctuations leading to removal of fines from discrete zones, leaving a gravel matrix. Some fines may also be lost due to piping into the underlying jointed bedrock. Because of the sandy nature of the upper core, the lack of filter protection, the presence of defects in the core, and the continuing core deterioration, there is a major concern for the stability and safety of the dam, if subjected to a major flood event resulting in a very high pool. Even though the probability of such a major flood event is very small, remedial measures are needed to protect against continuing deterioration and eventual failure.

After evaluation of various alternatives, the decision was made to construct a concrete cutoff wall, keyed a minimum of 15 ft into the rock and into the MMC material. At its deepest point, the cutoff wall will extend to elevation 843, more than 400 ft below the top of the dam.

PRESENT STATUS

In July 1988, a contract was awarded to Soletanche, Inc. for construction of a cutoff wall in Mud Mountain Dam at an aproximate cost of $20 million. Excavation for the cutoff wall will be accomplished by the slurry trench method using a "hydrofraise" rock mill-type excavator. Procedures and equipment will be similar to those used by Soletanche for the recently completed cutoff wall at Navajo Dam. The generalized cutoff wall configuration is shown in Figure 3. The wall will be constructed of primary elements 8 to 22 ft long, spaced approximately 8 ft apart. Secondary elements will then be constructed between the primary elements. Wall thickness will be 40 in. in the deep (Type II) portion and 32 in. in the shallower (Type I) portions. Besides the verticality controls of the hydrofraise, assurance of continuity between elements in the deep (Type II) portion of the wall will be accomplished with color coded concrete in primary elements. Also, by core sampling of concrete in both adjacent primary elements from secondary element excavations, using specialized drilling equipment developed by Soletanche, will add to the control.

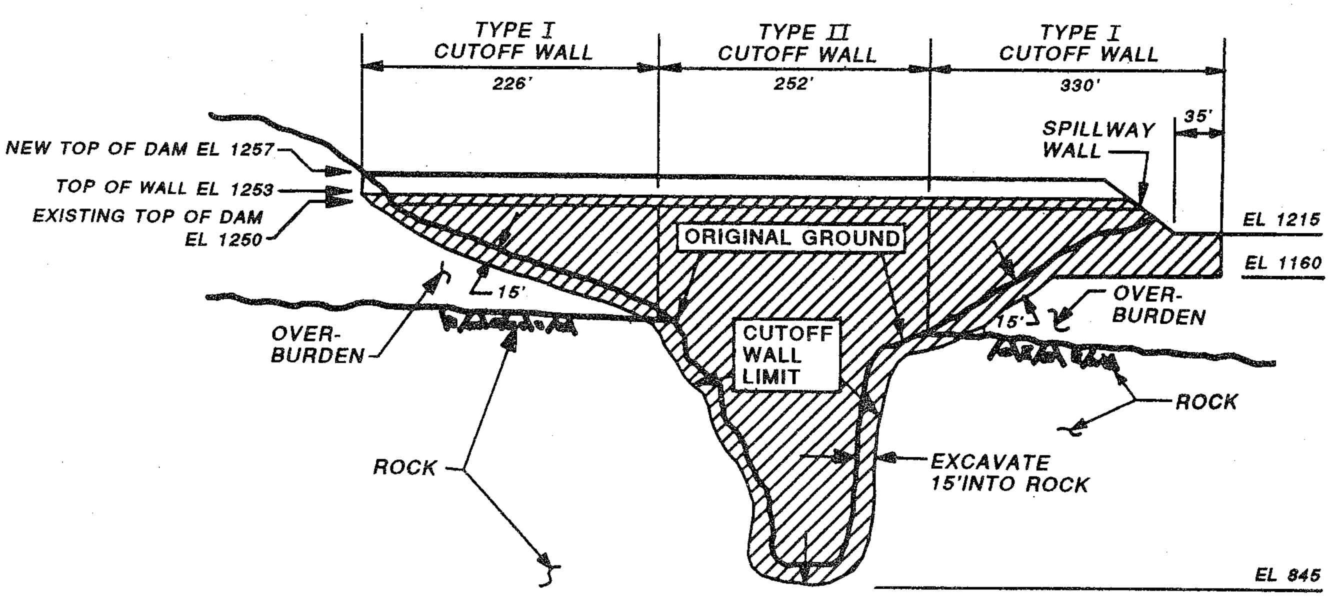
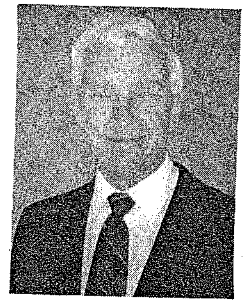


Figure 3. Cutoff wall cross section, looking downstream

The contract included excavating a 6-ft-diameter access shaft from the top of the dam to a 150- to 200-ft depth, to inspect and sample dam core materials, permitting a better understanding of the observed problems. Shaft excavation, at 180-ft depth, is now completed. It revealed three zones of small water-filled transverse cracks at respective depth of about 50, 100 and 140 ft.

The contractor has completed site development work and started construction of a short test section of the cutoff wall on May 16. Project completion is expected by early 1990.



Ken Graybeal is a geotechnical engineer, chief of the Soils Section in the Seattle District. He received his B.S. degree in civil engineering from the University of Washington and has done graduate work at Harvard University. He is a registered professional engineer in Washington State and has more than 27 years of experience in geotechnical engineering.

A Review of "Flume Investigation of a Composite, Erosion-Resistant Material"

by
Jerry Lee Anderson, Memphis State University
Robert F. Athow, Jr., US Army Engineer Waterways Experiment Station

A research investigation concerning the use of an erosion-resistant material to minimize the amount of waterflow-induced scour of noncohesive sediment was conducted by Dr. Andrew P. Salkied of Sediment Dynamics International, Ltd. Dr. Salkied documented his research in a report titled "Flume Investigation of a Composite, Erosion-Resistant Material."*

The research demonstrated, during a laboratory flume investigation, the effectiveness of a scour-inhibiting material in reducing the erosion rate of noncohesive material. The erosion resistance of the inhibitor was determined by a comparison of the waterflow-induced gravel loss rates from a sediment test section. Tests were conducted with three different grades of gravel with and without the use of the scour inhibitor. In all cases, the gravel loss (erosion) rate was less than 1 percent of the loss rate for the "clean" gravel when tested under similar velocity conditions. Data were collected to determine the "upper limit" of performance for the gravel and scour inhibitor combination. Data on the effect of using a less cohesive "clay" material were also compared.

A scour inhibitor has a wide range of potential applications for situations in which waterflow-induced scour of noncohesive sediments occurs. Scour may occur: at dikes and wing dams; with submarine cable and pipeline protection; and in unstable river and estuary channels.

BACKGROUND

Scour damage to training dikes and other structures emplaced in rivers and estuaries is a serious problem. Much of the damage is caused by erosion of substrate sediments. Because of the highly erodible characteristics of sand, scour zones occur most frequently in sandy substrates. Traditional methods, such as riprap, may not provide adequate protection because scour of the erodible substrate can continue between individual pieces of the riprap.

* Salkield, A. P. 1987 (Jun). "Flume Investigation of a Composite Erosion-Resistant Material," Sediment Dynamics International, Ltd., Moline, Illinois.

The scour inhibitor, made from a mixture of gravel and clay, is erosion-resistant and pliable when set. When the scour inhibitor is laid along the margins of a rigid structure located on an erodible sediment substrate, it provides scour resistance by covering the erodible sediment in the region of increased flow velocity. It also provides a comfortable margin between the erosion-resistant material and the erodible sediment where the velocities are much lower (Figure 1).

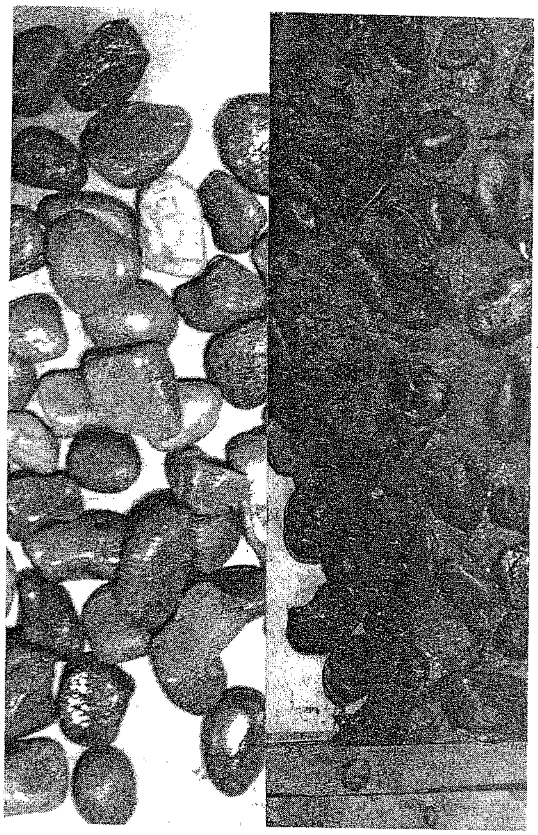


Figure 1. Typical sample of 1.5- to 2.5-in. grade "clean" gravel and of scour inhibitor, a clay matrix with 1.5- to 2.5-in. grade gravel

TECHNICAL OBJECTIVE

The technical objective of the flume investigation was to evaluate the erosion resistance of the scour inhibitor in comparison to the stability of clean gravel of the same size as that used to make the scour inhibitor. Three sizes of gravel were tested, with and without the clay matrix, to determine:

- The threshold of movement for graded gravel of three different sizes (0.5 to 0.75 in.; 0.75 to 1.5 in.; 1.5 to 2.5 in.)
- The transport rate of the gravel after initial movement had commenced.
- The threshold of movement (loss of gravel) for three samples of composite scour inhibitor.
- The rate of gravel loss for the three samples of scour inhibitor once erosion had been initiated.

The armoring and erosion behavior of the scour inhibitor was to be documented to include observation data and photographs.

EXPERIMENTAL PROCEDURE AND RESULTS

The investigation was conducted in accordance with questions posed by the technical objectives. Tests were conducted on the "clean" gravel to determine an initial "threshold" velocity and a velocity in which "continuous motion" of the gravel was observed. Erosion rates, lb/hr/ft, were experimentally determined for each grade of gravel for these two conditions. Results of tests conducted on the "clean" gravel are presented in Table 1.

Table 1

Comparison of "Clean" Gravel Loss Rates

Grade of Gravel, in.	Gravel Loss lb/hr/ft	Exit Velocity fps	
0.50 to 0.75	0.90	4.93	Threshold Velocity
0.75 to 1.50	1.10	4.78	
1.50 to 2.50	3.31	5.83	
0.50 to 0.75	13.10	5.84	Continuous Motion Velocity
0.75 to 1.50	12.64	6.29	
1.50 to 2.50	19.10	7.50	

In the scour inhibitor investigation, gravel loss could not be directly observed because the dispersed clay particles clouded the recirculated flume water. Although the initial flow velocity was approximately the same for the larger grade of gravel as in the "clean" gravel tests, during the scour inhibitor investigation of the two smaller grades of gravel, the velocities were not increased until the erosion rates had reached a minimum. This procedure was used until excessive quantities of the gravel were eroding. The same procedure was used with the 1.5- to 2.5-in. gravel; however, the flume was operated to its maximum flow after a stabilized gravel loss rate was occurring at a similar velocity for the "continuous motion" of the "clean" gravel. These results are presented in Table 2.

Table 2

Comparison of Gravel Loss Rates for Gravel with Scour Inhibitor

Grade of Gravel, in.	Gravel Loss lb/hr/ft	Exit Velocity ft/sec	
0.50 to 0.75	0.107	5.83	Continuous
0.75 to 1.50	0.081	6.29	Motion
1.50 to 2.50	0.179	8.45	Velocity

The erosion-loss rate of the 1.5- to 2.5-in. gravel mixed with the scour inhibitor was compared to a similar study on a "diamictite" gravel with a scour inhibitor made from dredge matrix material containing quantities of silt and sand.* The loss rate for "diamictite" gravel (composed of 3 parts gravel to 1 part clay by volume) was 1.3 lb/hr/ft at a velocity of 7.38 fps for threshold motion. The gravel loss rate for the scour inhibitor made from the diamictite material showed a much higher gravel loss rate than for the pure clay matrix of Salkield's investigation. The gravel loss rate for the scour inhibitor had an average of 4.87 lb/hr/ft at a velocity of 8.53 fps while for Salkield's investigation, the gravel loss rate at the same velocity was 0.179 lb/hr/ft. These figures indicate the loss rate from the dredge material matrix was more than 27 times that of the matrix made with the pure clay.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this investigation was to test the behavior of a scour inhibitor made from gravel and a "pure" clay and thus provide information on the upper limit of performance. Three grades of gravel were investigated. The gravel loss rate from the scour inhibitor was less than 1 percent of the loss rate from the "clean" gravel of the same grade. This performance was observed at flow velocities of up to 13 percent above the velocity that produced "continuous motion" transport in the "clean" gravel.

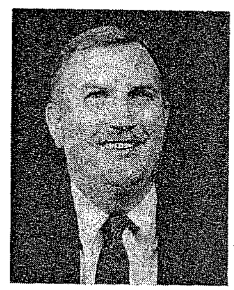
A comparison of the performance of the scour inhibitor made with a pure clay matrix and one made with "diamictite" matrix and a dredge material indicated that the gravel loss rate was about 27 times smaller for the clay matrix inhibitor. This result is probably a consequence of a loss of cohesion of the "diamictite" matrix caused by silt and sand in the dredge material.

Further investigations should include:

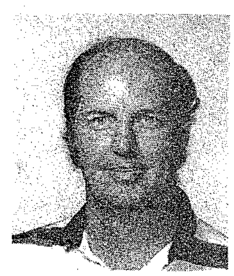
- Flume studies on the effect of known proportions of sand and silt in the clay matrix material.
- Large-scale handling, mixing, and laying trials in a field situation.
- Cost analysis comparison of the material for the scour inhibitor with conventional material.
- Comparative cost analysis of the techniques of scour prevention.

For further information, contact Robert Athow at (601) 634-2135.

Dr. Jerry L. Anderson is a registered professional engineer in the States of Tennessee and Missouri. Dr. Anderson has been on the faculty of the Department of Civil Engineering at Memphis State University for 17 years where he has a primary responsibility for the water resources program. Dr. Anderson's primary interest areas have been in urban drainage, floodplain modeling and investigation, flood control, erosion and sedimentation, and numerical modeling.



Robert Athow is a Research Hydraulics Engineer in the Estuarine Engineering Branch, Estuaries Division, Hydraulics Laboratory, WES. He received his B.C.E. from the University of Delaware and has done graduate work at Mississippi State and Texas A&M Universities. Presently, he serves as the program manager for the Improvement of Operations and Management Techniques Program.



*Parker, W. R. and Kirby, R. 1983. "Trials with the Sediment Science Scour Inhibitor: A Technique for Alleviating Scour, Modifying Underwater Topography and Providing Alternative Habitats," Unpublished.

Structural Displacement Monitoring Device to be Demonstrated

A demonstration of the Continuous Deformation Monitoring System (CDMS) will be held at Dworshak Dam in Ahsaka, Idaho, during the second week of August, 1989.

The CDMS is an automated monitoring system capable of measuring stuctural displacement over time using high-accuracy Global Positioning System surveying technology. This system will be installed at Dworshak Dam in early July and shall be monitoring portions of the structure for possible movement through the end of September.

There will be four separate demonstration dates beginning Aug. 8 and ending Aug. 11, 1989. Presentations are scheduled to start at 9 p. m. and end at noon.

Anyone wishing to attend should contact Mr. Carl Lanigan, U. S. Army Engineer Topographic Laboratories, CEETL-TL-SP, at AC 202 355-2752 or 2695.

REMR Symposium Registration Information Now Available

The REMR Technology Transfer Symposium will take place at the Holiday Inn - Saint Paul Center, 411 Minnesota Street, St. Paul, Minn., on Sept. 7 and 8, 1989.

As REMR programs are completed, innovative technology has emerged from the research. Much of this technology can be applied to many different areas of repair and rehabilitation. Large savings are now realized through the application of that technology. Widespread use will result in continued savings, according to William F. (Bill) McCleese, REMR Research Program manager.

McCleese said, "The program for the symposium is taking shape and I hope to see a lot of the chiefs from Districts and Divisions among the attendees."

The meetings will begin at 8:30 a. m. on both days. The agenda concludes at 5 p.m. on Thursday. The symposium ends at 12 noon on Friday.

Attendees may use the St. Paul Airport Limousine Service to reach the hotel. This shuttle charges $5.50 one-way per passenger and operates around the clock. It can be found outside, across from baggage carousel No. 6 on the lower level of the airport.

Single room rate at the Holiday Inn for the US Army Corps of Engineers is $45 plus tax per night. Individual reservations should be made by Aug. 6, by phoning AC 612 291-8800, or 1-800-HOLIDAY.

Request for Articles

Attention: Readers with experience in REMR activities

Subject: Request your articles, reports, photographs, news, and notices about your REMR activities

What to do: Send us a draft of your article. Furnish any illustrations you have (original glossy photographs and line drawings)

What we'll do: Publish your article under your byline. Provide you with editorial assistance if needed

Contact us: By writing—Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: CEWES-SC-A, PO Box 631, Vicksburg, MS 39181-0631

By calling—Bill McCleese, (601) 634-2512

COVER PHOTOS

Precast gatewell during installation.

Typical sample of 1.5- to 2.5-in. grade "clean" gravel and of scour inhibitor, a clay matrix with 1.5- to 2.5-in. grade gravel.

The REMR *Bulletin*

The REMR Bulletin is published in accordance with AR 310-2 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 US Government agencies, and the engineering community in general. Contributions 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, publication, or promotional purposes. 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 the Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: Bill McCleese (CEWES-SC-A), PO Box 631, Vicksburg, MS 39181-0631, or calling 601-634-2587.

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