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Countermeasures to Protect Bridge Piers from Scour

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I. Introduction

Scour causes 60% of bridge failures in the United States. National studies by the Federal Highway Administration (FHWA) of bridge failures caused by floods have shown the threat to bridge foundations is approximately equally distributed between scour at bridge piers and scour at bridge abutments. This paper presents the methods and results of NCHRP Project 24-07(2), "Countermeasures to Protect Bridge Piers from Scour." [1]

Approximately 83% of the 583,000 bridges in the National Bridge Inventory (NBI) are built over waterways. To cite just one example of the magnitude of the threat to bridges over water, in the 1994 flooding from a single storm (tropical storm Alberto) in Georgia, more than 500 state and locally owned bridges experienced damage attributed to scour. Thirty-one state owned bridges experienced scour depths ranging from 4 to 6 meters. Those 31 bridges had to be replaced.

Based on technical advisories and guidance from FHWA, most bridge owners have implemented comprehensive programs, inspections, and operational procedures to make their bridges less vulnerable to damage or failure from scour. New bridges are designed to resist damage from scour, while existing bridges are inspected regularly and evaluated to determine if a present or potential condition exists that may render the bridge vulnerable to damage during a future flood. When such a condition is found to exist, the bridge is rated as scour critical, and further evaluations are made to determine the best way to address the problem. Where pier scour is a problem, installation of pier scour countermeasures can be considered as one option in a comprehensive Plan of Action to reduce the vulnerability of the bridge.

Countermeasures for scour and stream instability problems are measures incorporated into a highway-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream instability and bridge scour problems. While considerable research has been dedicated to development of countermeasures for scour and stream instability, many countermeasures have evolved through a trial and error process and lack definitive design guidance. In addition, some countermeasures have been applied successfully in one area, but have failed when installations were attempted under different geomorphic or hydraulic conditions. This is particularly true of pier scour countermeasures. In the mid-1990s, FHWA guidance to the state DOTs cautioned that pier scour countermeasures, such as riprap, may not provide adequate long-term protection, primarily because selection criteria, design guidelines, and specifications were not available.

By the late 1990s, progress had been made in developing selection, design, and installation guidelines for pier scour countermeasures. For example, the publication of the first edition of Hydraulic Engineering Circular HEC-23 in 1997 was a first step toward identifying, consolidating, and disseminating information on countermeasure guidance [2]. In addition, the first phase of NCHRP Project 24-07 provided the initial results of laboratory and field research to evaluate the performance of pier scour countermeasures and develop design and implementation guidance [3].

From the review of the literature, it is apparent that local scour at bridge piers is a potential safety hazard to the traveling public and is a major concern to transportation agencies. Bridge-pier scour is a dynamic phenomenon that varies with water depth, flow velocity, flow angle, pier shape and width, and other factors. If it is determined that scour at a bridge pier can adversely affect the stability of a bridge, scour countermeasures to protect the pier should be considered. Because of their critical role in ensuring bridge integrity, and their potentially high cost, it is important that the most appropriate countermeasures be selected, designed, and constructed.

The objectives of NCHRP Project 24-07(2) were to develop and recommend: (a) practical selection criteria for bridge pier scour countermeasures, (b) guidelines and specifications for design and construction, (c) guidelines for inspection, maintenance, and performance evaluation. The countermeasures considered included:

- Riprap
- Partially grouted riprap
- Articulating concrete blocks
- Gabion mattresses
- Grout-filled mats

In addition, issues related to riprap at skewed piers and mounded riprap were investigated.

II. Laboratory Testing

NCHRP Project 24-07(2) was initiated in April 2001 to refine the results of earlier work, test additional pier scour countermeasures, and develop selection criteria and detailed guidelines and specifications. Laboratory testing for an initial set of countermeasures (riprap, articulating concrete blocks, and indoor tests of partially grouted riprap) was conducted in the hydraulics laboratory at the Colorado State University (CSU) Engineering Research Center. Testing began in August 2003 and was completed in December 2004. Continuation funding for additional countermeasure testing (gabion mattresses, grout-filled mattresses, riprap at skewed piers, mounded riprap, and prototype-scale tests of geotextile bags and partially grouted riprap) was authorized in December 2004 and testing of these countermeasures was completed in December 2005.

A. Developing the Testing Program

Items identified as gaps in the current state of the practice were reviewed and a specific test, or series of tests, was designed to address each deficiency. Merits and deficiencies of each countermeasure were considered in developing the testing program, including:

- Selection criteria
- Design methods and guidelines
- Construction specifications and guidelines
- Maintenance and inspection guidelines
- Performance evaluation guidelines

Dominant-process design models that accurately reflect the mode of failure associated with the particular countermeasure were reviewed in order to size the armor elements for the laboratory conditions. Typically, these models included local hydraulic conditions characterized by a combination of velocity and shear stress. It was decided to size all countermeasure armor for a design velocity of 2 times the critical velocity of the 0.6 mm sand comprising the bed material, resulting in an approach velocity of 0.6 m/s for countermeasure design.

The laboratory tests were not designed to replicate any particular prototype scale conditions. For example, a $2V_{crit}$ run (using a 20-cm square pier) was not intended to represent specific scale ratio of a prototype pier or flow condition. The intent of the testing program was to provide valid comparisons of countermeasure types to each other and to the unprotected condition. In each case, the test countermeasure was "designed" to withstand the $2V_{crit}$ hydraulic condition. For example, the riprap size was selected such that particle dislodgement or entrainment was not anticipated during the $2V_{crit}$ run. This did not mean that the riprap (or any other countermeasure) would not fail due to other factors, such as settling, edge undermining, or winnowing of substrate material. Selected runs utilizing an approach velocity greater than $2V_{crit}$ were intended to take each system to failure by particle dislodgement.

Criteria for rating performance was consistent between countermeasures, but was not necessarily identical for all countermeasures. A countermeasure was considered to have failed if the countermeasure (or its component particles) was dislodged, lifted, or entrained. Relative performance was gauged by whether the countermeasure functioned as intended. Specifically, if settling along the countermeasure edge was expected, actual settlement was not considered poor performance. Maximum scour anywhere within the limits of the countermeasure or along the edge of the countermeasure was documented.

The testing program also addressed stability and performance issues associated with the extent of the countermeasure placement around the pier, and the termination details at the pier and around the periphery of the installation. Lastly, various filter types and extents were investigated by varying this aspect for selected test runs.

B. Testing Protocol – Indoor Flume

Testing conducted for Research Project 24-07(2) utilized the largest of the CSU laboratory's sediment recirculating flumes. The flume is 2.6 m wide by 1.4 m deep by 60 m long, and is capable of recirculating water and sediment over a range of slopes up to 2%. The maximum discharge in the flume is 2.8 cubic meters per second (cms).

A mobile data acquisition cart traverses the flume and provides flexibility in data collection. A wide variety of point gages and velocity probes can be mounted to the cart. The data acquisition cart can then be positioned to collect data at any given location in the flume. The cart also has the capacity to provide space and power for a personal computer for data collection.

Three piers were placed along the centerline of the testing flume. Square piers 20 cm long by 20 cm wide were used. Spacing between the piers was a minimum of 13 m to ensure the formation of uniform flow lines upstream of each pier. Sand with a d_{50} of approximately 0.6 mm was placed in the flume to a depth of approximately 45 cm. Figure 1 provides a schematic of the flume, data acquisition cart, pier layout, and ancillary components.

A matrix of flume tests was developed for the research program and approved by the NCHRP research panel prior to initiating the tests. Each clear water test consisted of a series of two discharges. Discharge rates were predetermined to correspond to flow velocities of V_{crit} and $2V_{crit}$ where V_{crit} is the calculated critical velocity of the 0.6 mm sediment size utilized throughout the testing program (0.3 m/s). The V_{crit} and $2V_{crit}$ runs were performed without sediment recirculation. Separate runs on selected countermeasure configurations were performed at velocities greater than $2V_{crit}$ with sediment recirculation, therefore, clear water, live bed, and sediment-deficient conditions were examined.

During the live bed runs, bed form type, length, and height were recorded. Flow duration was sufficient to ensure that bed forms migrated through the system. One baseline run was performed at velocities up to $3V_{crit}$ to determine the performance of standard, loose riprap under conditions where particle dislodgement or entrainment is anticipated.

Data collected during each test included pre-test surveys, approach flow velocity, local pier velocity, flow depth and post-test surveys. In addition, non-professional photographic and video footage was recorded for each test. Water surface elevations were collected every 1.3 m along the flume, and local and approach flow velocities collected at each pier. Water surface elevations were determined by a point gage accurate to ± 1.5 mm. Velocities were collected with an 3-D acoustic doppler velocimeter, accurate to ± 2%. Approach velocities were collected at 20, 60, and 80% of the flow depth. Local velocity profile measurements were collected at each pier. Pre- and post- test surveys were conducted with a point gage and total station survey equipment. Survey resolution was sufficient to accurately map each scour hole and document system performance.

Prior to each test, the tailgate was closed and the flume slowly filled with water until the target flow depth of 30.5 cm was established. Flow was introduced very slowly to ensure no local scour occurred during start-up. During the slow filling process, air was allowed to escape from the sand bed. With the flume full of water, discharge was slowly increased to the target discharge, while simultaneously opening the tailgate until steady flow at the target depth of 30.5 cm relative to the initial bed surface was obtained. This process ensured a gradual acceleration of flow until the target velocity was achieved and maintained.

Each run then proceeded for a duration of approximately 2.5 hours while velocity and water surface data was collected at each pier, and at designated locations between piers. For tests utilizing live-bed conditions with sediment feed, the duration was increased to 8 hours per run. After each test, the discharge was gradually decreased and the tailgate adjusted to ensure that no additional scour occurred during the drain-out period. Typically, the flume was allowed to drain out overnight, and the sand bed around each pier was mapped the next day. Figure 2 provides an example of the general testing procedure and results.

C. Riprap

Most of the early work on the stability of pier riprap is based on the size of the riprap stones and their ability to withstand high approach velocities and buoyant forces. Parola [4] noted that secondary currents induced by bridge piers cause high local boundary shear stresses, high local seepage gradients, and sediment diversion from the streambed surrounding the pier, and that the addition of riprap also changes the boundary stresses. His study recommended that the stone size should be determined for plane bed conditions, which were the most severe conditions found in model studies to that point.

However, a subsequent study of the causes of riprap failure at model bridge piers conducted by Chiew [5] under clear-water conditions with gradually increasing approach flow velocities defined three modes of failure:

1. Riprap shear failure – whereby the riprap stones cannot withstand the downflow and horseshoe vortex associated with the pier scour mechanism.
2. Winnowing failure – whereby the underlying finer bed material is removed through voids or interstices in the riprap layer.
3. Edge failure – whereby instability at the edge of the coarse riprap layer initiates a scour hole beginning at the perimeter and working inward that ultimately destabilizes the entire layer.

Lim and Chiew [6] conducted experiments to evaluate the stability of pier riprap under live-bed conditions with migrating bed forms. Subsequent research [3, 7, 8, 9, 10, 11, 12] indicated that bed-form undermining is the controlling failure mechanism at bridge piers on rivers where mobile bed forms are present during high flows, especially sand bed rivers. Figure 3 shows typical photographs of riprap tests under clear-water and live-bed conditions in the CSU indoor flume.

D. Partially-Grouted Riprap

Current practice in the United States discourages the use of grouted riprap, primarily because the voids within the riprap are often nearly completely filled with grout in most cases, which creates rigidity and impermeability that often leads to failure. Guidelines on the construction of grouted riprap in the United States are associated almost entirely with riprap bed and bank protection (e.g., [13]). Total grouting converts a flexible revetment material like a riprap layer into a rigid mass and reduces the permeability of the layer. This may cause the entire riprap layer to fail as a result of either undermining or uplift and thus negates the natural benefit caused by raveling of loose riprap into the scour hole or trough of migrating bed forms.

Partially grouted riprap provides a more suitable alternative to total grouting because it alleviates the concerns and problems associated with complete filling of the voids with grout. Partial grouting increases the stability of the riprap unit without sacrificing flexibility and allows for the use of smaller rock and thinner riprap layers in areas where the required stone size for loose riprap is expensive or unavailable.

A target for grout placement is that the voids within a riprap layer should contain about 75% grout in the upper third of the layer, 50% grout in the middle third, and 25% grout in the lower third [2]. No grout should penetrate deep enough to come into contact with any underlying filter. Construction methods must be closely monitored to ensure that the appropriate voids and surface openings are provided. Contractors in Germany have developed techniques and special equipment to achieve the desired grout coverage and the right grout penetration.

Heibaum [14] indicates that grouting has proven its long-term stability and ability to keep costs low, and notes that laboratory tests at Braunshweig University in Germany proved that partially grouted riprap is stable up to a flow velocity of 8 m/s. Since the riprap is dumped or placed as needed and only then is the layer grouted, a close contact to structural elements such as bridge piers can be achieved. Figure 4 shows typical photographs of the partial grout tests in the indoor flume.

E. Articulating Concrete Blocks

Articulating concrete block systems (ACBs) can be used to provide a flexible armor layer as a pier scour countermeasure. These systems consist of preformed units which interlock, are held together by steel rods or cables, are bonded to a geotextile or filter fabric, or abut together to form a continuous blanket or mat. Data sheets for a number of the more common proprietary ACB revetment systems can be found in [16], while [3] provides a brief review of the limited studies conducted on the use of ACBs for pier scour protection.

There is limited experience with the use of articulating block systems as a scour counter-measure for bridge piers alone. More frequently, these systems have been used for bank revetments and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. For this reason, guidelines for placing articulating block systems along banks and in channels are well documented (e.g., [15]), but there are few published guidelines on the installation of these systems around bridge piers.

Where ACB systems have been installed as a countermeasure for scour at bridge piers, cable-tied concrete block mats are used most often. There are two failure mechanisms for ACBs: (1) overturning and rollup of the leading edge of the mat where it is not adequately anchored or toed in, and (2) uplift at the center of the mat where the leading edge is adequately anchored. In the absence of a filter or geotextile, winnowing can still occur and can result in subsidence of all or a portion of the ACB mat. Studies conducted on the effectiveness of ACBs as a countermeasure have determined that the use of a filter fabric or geotextile is essential to the overall effectiveness and stability of the ACB system unless the substrate is coarse enough to prevent winnowing through the open cells of the blocks.

Although cables can prevent the loss of individual blocks or group of blocks from the matrix, the Factor of Safety design method presented in [15] does not attribute any additional stability benefit to cables. For the cable to provide a restraining force, tension must be mobilized in the cable, thus implying that blocks have already begun to uplift and therefore loss of intimate contact, which constitutes failure, has already occurred. Tests of ACB systems in the indoor flume were conducted using no cables; the critical shear stress for the blocks was determined in a smaller flume prior to placement around the test piers. Figure 5 shows typical photographs of ACB testing in the indoor flume.

F. Gabion Mattresses

Gabion mattresses are containers constructed of wire mesh or welded wire and filled with loose stones or other similar material. The stones used to fill the containers can be either angular rock or large cobbles, although angular rock is preferred due to higher degree of interlock between stones.

Gabions have been used for streambank protection for more than 100 years in Europe and have gained increasing popularity in the United States, especially in the desert Southwest. Like riprap, they are porous, being composed of loose rock, and are not susceptible to uplift forces. They can be stacked to form a wall or joined together to form a large mattress. If the configuration is undermined or becomes unstable, the inherent flexibility of the wire mesh allows them to deform to the bed or bank. In addition, the use of a wire mesh allows for the use of relatively small stones, which can yield the same amount of protection as much larger particles in a loose configuration.

It may seem intuitive that gabions should be effective as pier scour countermeasures, especially if they are installed with an underlying filter or geotextile and a seal to the pier is provided. However, rivers carrying coarse bedload can abrade the wire comprising the baskets. The passage of large bed forms can cause the wire mesh to break under tension during deformation of the gabion, allowing the fill stones to be lost. The gabion mattress may pull away from the pier face if there is significant edge settlement associated with winnowing or the passage of bed forms. These factors appear to have contributed to the failure of the gabions used to counter pier scour at the Whakatane River Bridge on State Highway 30 in New Zealand [10]. Anchoring the gabion to the bed with long steel rods may alleviate some of these problems. Figure 6 shows photographs of typical tests of gabion mattresses conducted in the indoor flume.

G. Grout-Filled Mats

The grout-filled mat is a single, continuous layer of strong synthetic fabric sewn into a series of bags or compartments that are connected internally by ducts. The compartments are then filled with a concrete grout that, when set, forms a mat comprised of a grid of connected pillow-shaped units. While the individual blocks may articulate within the mat and the mat remains structurally sound, the general design approach is to consider the mat as a rigid monolithic layer. In some cases, the mat may be strengthened with cables installed similar to those used in articulating concrete blocks. Filter points or weep holes allow for pressure relief across the mat.

Grout-filled mat systems can range from very smooth, uniform surface conditions approaching cast-in-place concrete in terms of surface roughness, to extremely irregular surfaces exhibiting substantial projections into the flow, resulting in boundary roughness approaching that of moderate size rock riprap. Because this type of armor is fairly specialized, comprehensive technical information on specific mat types and configurations is available from a number of manufacturers.

The primary failure mechanisms for grout-filled mats consist of rolling, undercutting, and scouring at gaps [16]. Rolling, the most severe form of failure, is related to uplift forces created by flow over the mat. This allows the mat at midsection to be "lifted up" slightly and then pushed loose by the force of the current or allows the edges of the mat to be rolled back. Undercutting is a gradual process arising from local scour at the mat edges and from the main horseshoe vortex. Scouring at the gaps between the mat and the pier wall allows the horseshoe vortex to generate a scour hole beneath the front edge or side sections of the mat.

The research to date on the use of grout-filled mats as a bridge scour countermeasure found that placement is extremely important for successful performance and effectiveness. Properly placed grout mats extending 1.5 to 2 pier widths were found to provide significant protection to bridge piers. Recommendations in [16] are that grout mats should be placed at bed level and that toeing in the edges of the mat may increase stability with regard to potential rolling failure and undercutting, especially under live-bed conditions. Further recommendations [17] are that anchors should be used to protect the leading edge against uplift forces when the mat is placed on the surface of a loose, erodible channel bed. References [18] and [19] stress the importance of a tight seal around the pier-mat interface to inhibit scour and undermining beneath the mat. Mat selection and sizing criteria based on analysis of sliding stability are provided in [2]. Figures 7 and 8 show installation and testing of materials used to simulate rigid and flexible grout-filled mats, respectively.

H. Outdoor Tests: Partially Grouted Riprap

Two tests were conducted to quantify performance of partially grouted riprap at essentially prototype scale. Installation of the countermeasure was performed at a rectangular pier measuring 0.45 m wide by 1.4 m long. A pre-formed scour hole around the pier was partially filled with sand-filled geotextile containers to serve as the filter beneath the armor stone. The containers were constructed using a nonwoven needlepunched fabric. Each container measured 1.2 m x 0.5 m x 0.1 m and was filled with 91 kg of sand.

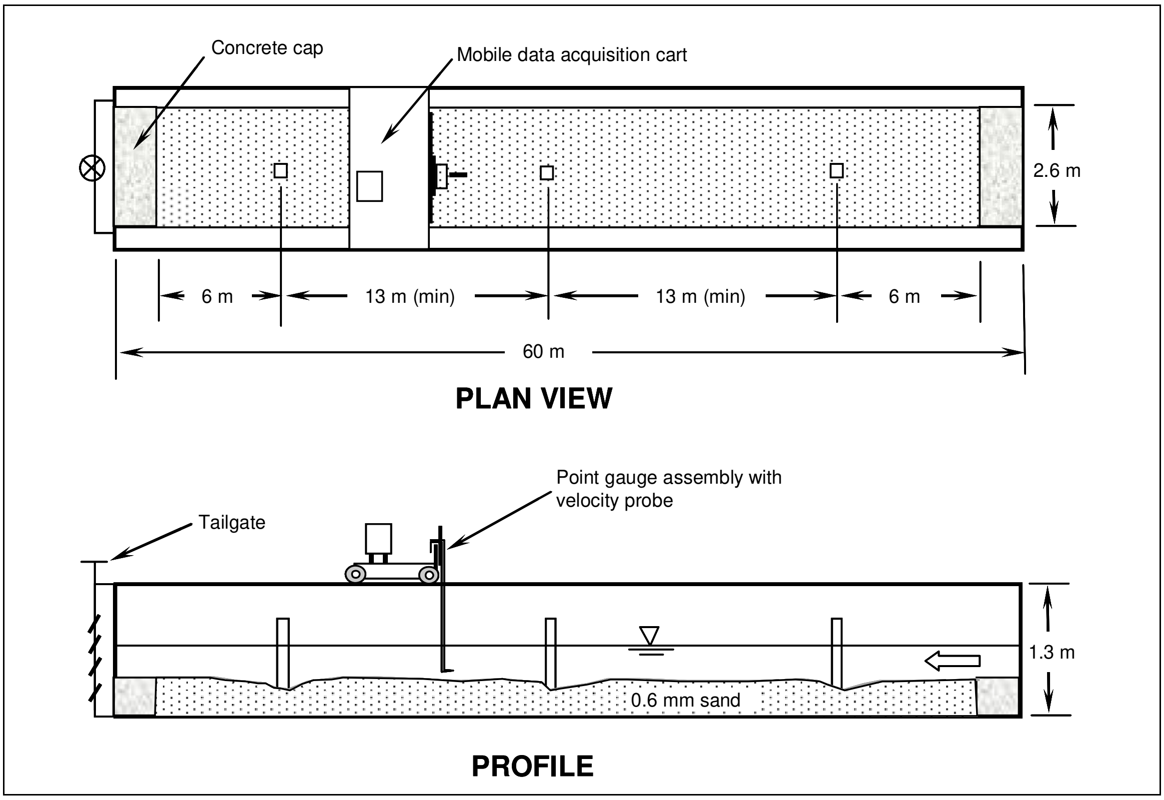
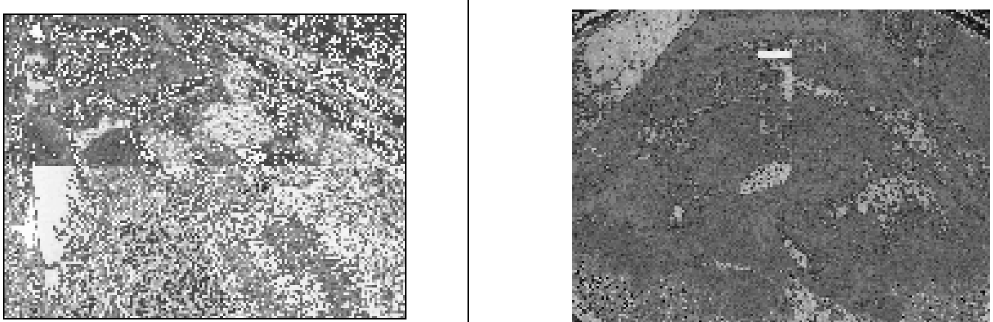


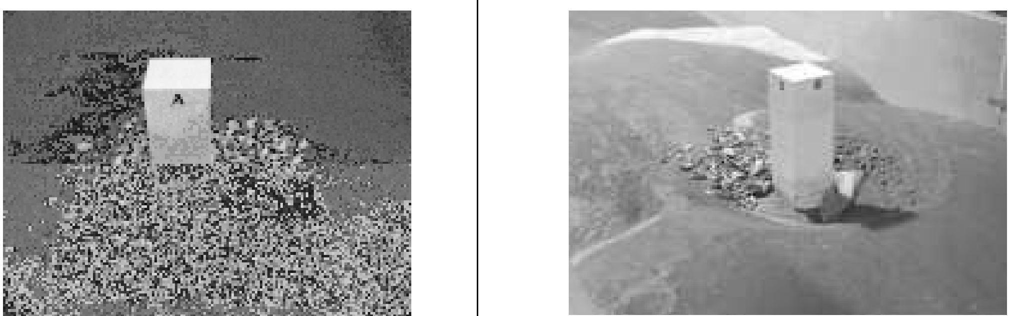
Figure 1. Schematic diagram of indoor flume showing data acquisition.



a. Flow is slowly introduced into flume.

b. Scour at unprotected pier after test at $2V_{crit}$ test. Note dune field.

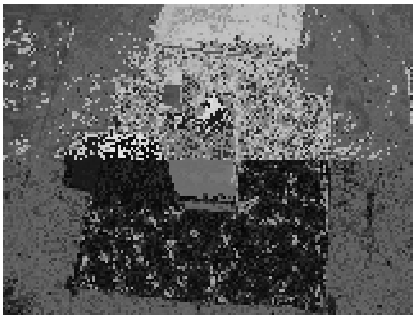
Figure 2. Indoor flume tests showing typical initial and final conditions.



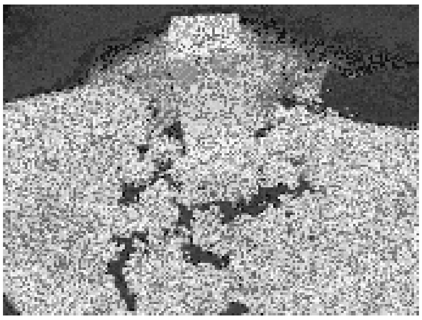
a. Riprap after $1V_{crit}$ test.

b. Riprap after $2V_{crit}$ test. Note particle displacement when areal extent is insufficient.

Figure 3. Indoor flume tests of standard riprap.

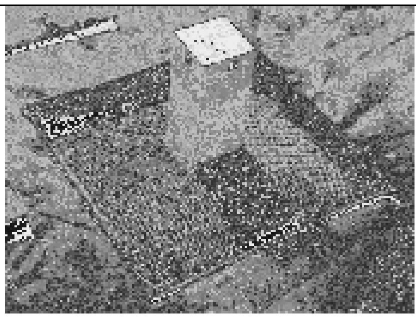


a. Placing partially-grouted riprap. Note spot-grouting technique.

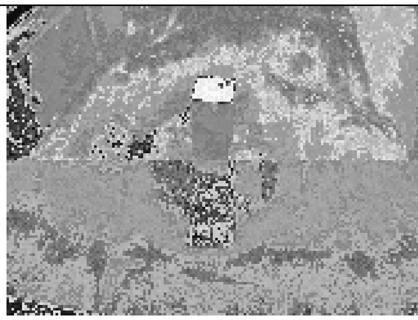


b. Partial grout after $2V_{crit}$ test.

Figure 4. Indoor flume tests of partially grouted riprap.

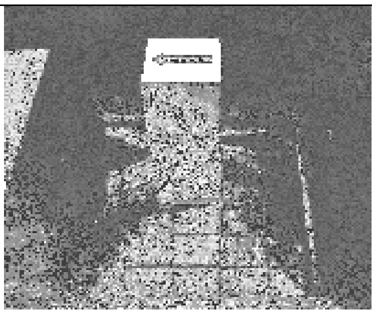


a. Placing articulating concrete blocks. Note toedown depth at perimeter of system.

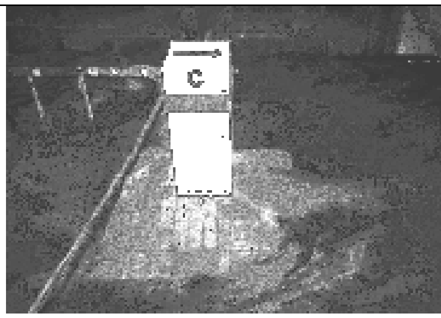


b. Articulating concrete blocks after 2 V_{crit} run. Note dune field and loss of blocks on the right side of pier.

Figure 5. Indoor flume tests of articulating concrete blocks.

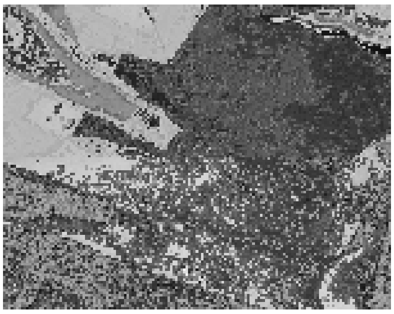


a. Placing gabion mattresses. Note toedown depth at perimeter of system.

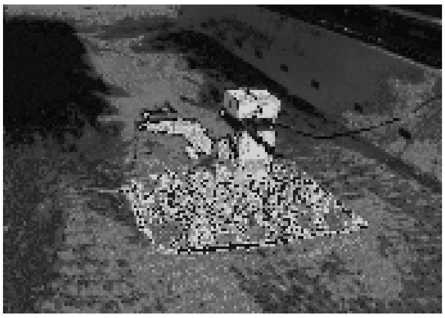


b. Gabion mattresses after 2 V_{crit} run. Note dune field and loss of blocks on the right side of pier.

Figure 6. Indoor flume tests of gabion mattresses.

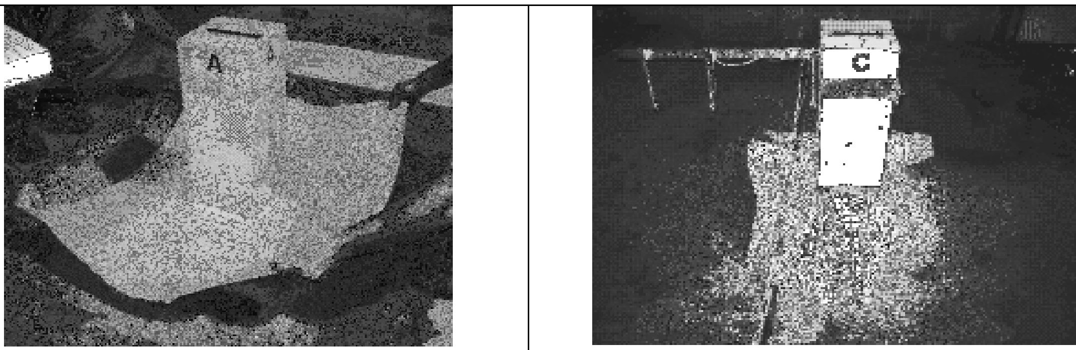


a. Placing rigid grout-filled mat.



b. Rigid grout-filled mat after $2V_{crit}$ test. Note undermining around periphery of mat.

Figure 7. Indoor flume tests of rigid grout-filled mats.



a. Placing flexible grout-filled mat. b. Flexible grout-filled mat after $2V_{crit}$ test.

Figure 8. Indoor flume tests of flexible grout filled mat.

The geocontainers were dumped into flowing water (0.5 m/s velocity and 0.5 m depth) using a standard backhoe bucket equipped with a grapple. Durable sandstone rock with a d_{50} of 15 cm was dumped onto the geocontainers using the same piece of equipment.

The first test examined potential constructability and environmental issues associated with underwater application of grout (Figure 9). Because of concerns voiced by permitting agencies involved with in-stream construction, water quality was monitored with a series of In-Situ Troll 9000 Profilers placed in stream at seven locations. The Troll 9000 Profilers continually recorded measurements of pH, conductivity, turbidity, and temperature. Baseline conditions were established prior to initiation of the grout placement 3.6 m upstream of the pier along the centerline of the flume. Figure 10 provides an illustration of pH measurements from the continuous monitors.

The second test was conducted to examine the stability performance of partially grouted riprap (right side of pier) compared to standard (loose) riprap of the same size and gradation (left side of pier). After 2 hours of flow with an approach velocity of 1.95 m/s, the loose riprap had scoured adjacent to the pier, whereas the partially grouted riprap remained intact. Figure 11 shows the installation after completion of the test.

III. Results

NCHRP Project 24-07(2) resulted in the development of a unified set of guidelines, specifications, and procedures that can be accepted by the State DOTs in the U.S. for the design, installation, and inspection of a variety of armoring-type countermeasures to protect bridge piers from scour.

To guide the practitioner in developing appropriate designs and ensuring successful installation of pier scour countermeasures, a countermeasure selection method was developed. Five factors comprise the model, and a numerical Selection Index (SI) is calculated. A higher value of SI indicates that the countermeasure type is more appropriate from both suitability and economic perspectives. The five factors are:

S1: Bed material size and transport
S2: Severity of debris or ice loading
S3: Constructability constraints
S4: Inspection and maintenance requirements
LCC: Life cycle costs

The Selection Index is calculated as:

SI = (S1xS2xS3xS4)/LCC

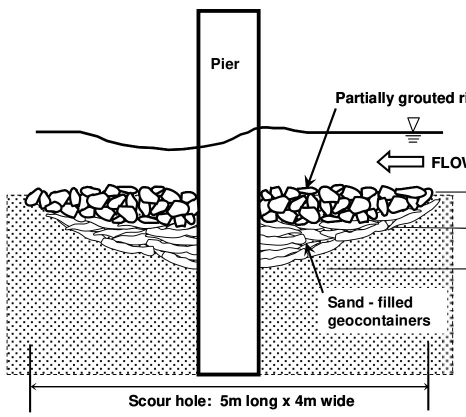
The findings and recommendations of the study are being combined into guidelines for each countermeasure type as stand-alone appendices. These guidelines are presented in a standard three-part format using the Federal Highway Administration's Hydraulic Engineering Circular (HEC) 23 [2] as a guide. Each guideline includes:

- Part 1 – Design and Specification
- Part 2 – Construction
- Part 3 – Inspection, Maintenance, and Performance Evaluation

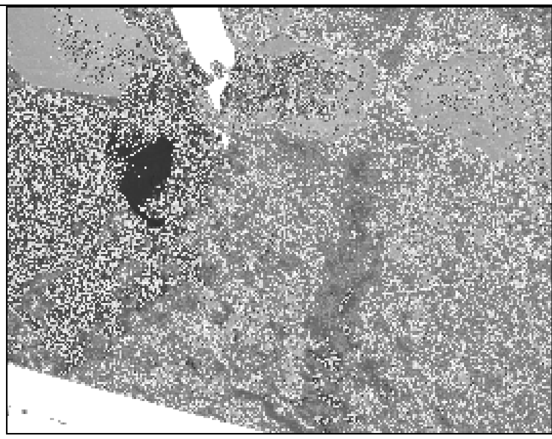
Individual guidelines are currently being finalized and will contain specific recommendations for the following items: Material requirements, extent of system placement, thickness, filter requirements, transitions and termination details, and countermeasure sizing equations.

Acknowledgment

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration (FHWA), and was conducted through the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Research Council (NRC).



a. Schematic diagram of installation



b. Placing grout in flowing water.

Figure 9. Prototype-scale test of partially-grouted riprap in the outdoor flume.

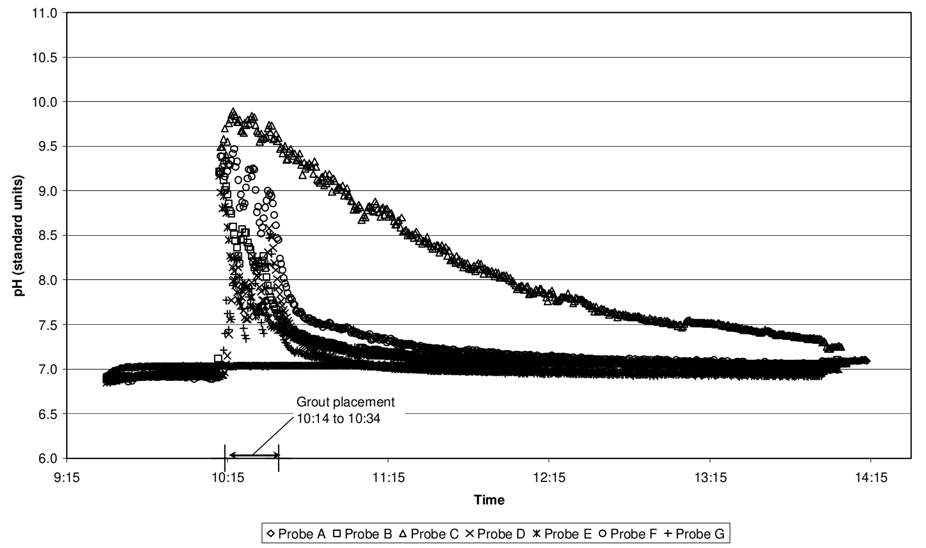
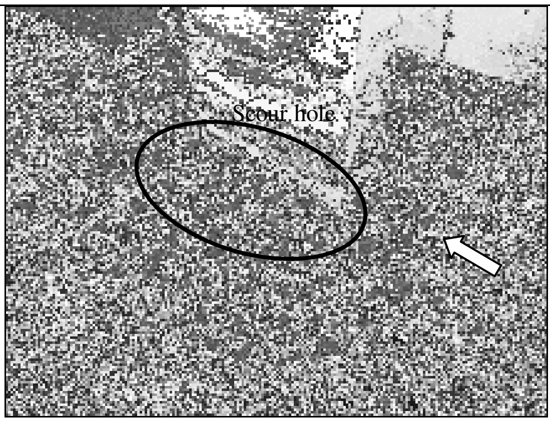
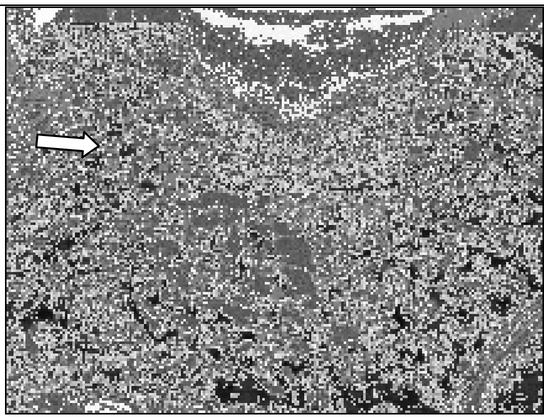


Figure 10. Graph of pH versus time during partial grout installation. Probe C (triangles) was located directly downstream of the pier at the edge of the grouting area.



a. Left side of pier showing displacement of loose rock riprap and scour adjacent to pier.



b. Right side of pier showing partially grouted riprap intact.

Figure 11. Comparison of standard riprap vs. partially grouted riprap.

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