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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 593

**Countermeasures to Protect
Bridge Piers from Scour**

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Fort Collins, CO

Subject Areas

Planning and Administration • Bridges, Other Structures, and Hydraulics and Hydrology • Materials, Construction, and Maintenance

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in cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

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2007
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Crawford F. Jencks

Deputy Director, Cooperative Research Programs
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This report documents research on local scour at bridge piers resulting in the development and recommendation of a practical selection criteria for bridge-pier scour countermeasures, guidelines and specifications for design and construction of those countermeasures, and guidelines for their inspection, maintenance, and performance evaluation. Because of their critical role in ensuring bridge integrity and potentially high cost of these countermeasures, it is important that the most appropriate countermeasures be selected, designed, and constructed. The contents of this report are, therefore, of immediate interest to highway professionals responsible for planning, administrating, evaluating, designing, constructing, inspecting and maintaining bridges and other structures founded in erosive areas. The report is also of interest to those charged with specifying materials testing procedures and acceptable results, setting budget goals, and making policy.

Scour at bridges is a potential safety hazard to the traveling public. Because of the critical role of countermeasures in ensuring bridge integrity, as well as their potential high cost, scour countermeasures must be selected, designed, and constructed based on site conditions and other factors. NCHRP Project 24-07, completed in October 1998 by the University of Minnesota, was undertaken to research the performance of various countermeasures for pier protection.

Under NCHRP Project 24-07(2), Ayres Associates Inc. was contracted to extend the results and applicability of the earlier project by developing and recommending practical selection criteria for bridge pier scour countermeasures; guidelines and specifications for design and construction of the suitable countermeasures; and guidelines for inspection, maintenance, and performance evaluation of the countermeasures. The countermeasures addressed include riprap, partially grouted riprap, articulating concrete block systems, gabions, grout-filled mattresses, and geotextile sand containers (used as a filter).

Because some of the addressed countermeasures have had limited field application, the specifications contained within this report are based in part on laboratory testing using small- and prototype-scale models. These controlled experiments cannot duplicate all of the possible field conditions; consequently, frequent monitoring must be included in the countermeasure programs that use the new guidelines presented in this report.

The selection methodology defining the proper conditions for the use of each specific countermeasure is presented in Appendix B. The selection methodology is also available on the TRB website (http://www.trb.org/news/blurb_detail.asp?id=7998) as an interactive Microsoft, Excel spreadsheet. Appendixes C through G consist of stand-alone documents containing all the necessary information on each individual countermeasure. To develop the guidelines and recommendations for partially grouted riprap, three German documents were translated into English. These translations are available in the Reference Document on the TRB website.

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Two research team members from Germany supported the team on several tasks. Dr. M.H. Heibaum, a geotechnical engineer from the German Federal Waterways Engineering and Research Institute (*Bundesanstalt für Wasserbau*, or BAW), assisted in the translation of BAW “Code of Practice” documents and provided guidance on the design and installation of partially grouted riprap and sand-filled geotainers as implemented along Germany’s extensive inland waterway system. Mr. Justus Trentmann of Gewatech-Soil and Hydraulic Engineering (*Gewatech Grund- und Wasserbau GmbH & Co. KG*) of Osnabrück, Germany, provided his unique expertise with the formulation and installation of partially grouted riprap. Both hosted a field trip to various field sites in Germany for two research team members and a Federal Highway Administration representative, providing insights on partially grouted riprap technology, which were invaluable in the development of an appropriate laboratory test plan. Dr. Heibaum and Mr. Trentmann also participated in a prototype-scale installation in the United States, which demonstrated the adaptability of this technique as a pier scour protection countermeasure.

All laboratory testing was performed at the Colorado State University Engineering Research Center Hydraulics Laboratory under the direction of Dr. C. Thornton and Mr. M. Robeson. The assistance of Ms. L.G. Girard and Mr. D. Varyu, graduate students, is also acknowledged.

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S U M M A R Y

Countermeasures to Protect Bridge Piers From Scour

Overview

This research accomplished its basic objectives of developing guidelines and specifications for design and construction, and guidelines for inspection, maintenance, and performance evaluation for a range of pier scour countermeasures including riprap, partially grouted riprap, articulating concrete blocks, gabion mattresses, grout mattresses, and geotextile sand containers.

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, 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, constructed, and maintained.

In this study, existing design equations for sizing the armor component of the pier scour countermeasures of interest were used to develop a laboratory testing program. However, sizing the armor is only the first step in the comprehensive design, installation, inspection, and maintenance process required for a successful countermeasure. A countermeasure is an integrated system that includes the armor layer, filter, and termination details. Successful performance depends on the response of each component of the system to hydraulic and environmental stresses throughout its service life. In this context, filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also necessary.

To support the selection of an appropriate pier scour countermeasure for site-specific conditions, a countermeasure selection methodology was developed. It provides an assessment of the suitability of each of five specific countermeasure types based on a variety of factors involving river environment, construction considerations, maintenance, performance, and estimated life-cycle cost of each countermeasure.

Research Approach

This research was undertaken to extend the results and applicability of an earlier study (NCHRP Project 24-07), investigate additional countermeasure types, and develop detailed design guidelines. The research approach involved the following steps.

1. Transition from NCHRP 24-07 based on a review of the Users Guide and Final Report (both unpublished) from the University of Minnesota, St. Anthony Falls Laboratory

2. Integration of European technology identified during a 1998 TRB/FHWA scanning review by including two research team members from Germany
3. Completion of a literature review and evaluation of current practice for pier scour countermeasures
4. Field site visits to countermeasure installations in the United States
5. Field site visits to research facilities and scour countermeasure installations at numerous project sites in Germany—sponsored by the German Federal Waterways Engineering and Research Institute (*Bundesanstalt für Wasserbau* or BAW)
6. Completion of extensive small-scale and prototype-scale laboratory investigations at the Colorado State University Hydraulics Laboratory
7. Integration of the survey of current practice with laboratory test results and other available guidance into a set of stand-alone Design Guidelines for five pier scour countermeasure systems
8. Development of a selection methodology for pier scour countermeasures considering site-specific conditions

The following sections provide a brief overview of the five countermeasure systems and their applicability for bridge pier scour protection.

Riprap

When properly designed and used for pier scour protection, riprap has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. For the DOTs, riprap has been the most common countermeasure installed at bridge piers. In this study standard (loose) riprap was used as a baseline and benchmark for evaluating the performance of other pier scour countermeasures. The study validated and extended existing guidelines for using riprap for pier scour protection.

Partially Grouted Riprap and Geocontainers

Partially grouted riprap consists of specifically sized rocks that are placed around a pier and grouted together with grout filling 50% or less of the total void space. In contrast to fully grouted riprap, partial grouting increases the overall stability of the riprap installation unit without sacrificing flexibility or permeability. It also allows for the use of smaller rock compared to standard riprap, resulting in decreased layer thickness. The system typically includes a filter layer—either a geotextile fabric or a filter of sand and/or gravel—specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention. Tests conducted under this study confirm the applicability of partially grouted riprap as a scour countermeasure for bridge piers.

Based on prototype-scale testing, the use of sand-filled geocontainers composed of non-woven needle-punched geotextile was confirmed to be an appropriate means of establishing a filter layer around a pier when placement of either standard riprap or partially grouted riprap must occur under water.

Articulating Concrete Block Systems

Articulating concrete block (ACB) systems provide a flexible armor for use as a pier scour countermeasure. These systems consist of preformed concrete units that either interlock, are held together by cables, or both. After installation is complete, the units form a contin-

uous blanket or mat. The term “articulating” implies the ability of individual blocks of the system to conform to changes in the subgrade while remaining interconnected. Block systems are typically available in both open-cell and closed-cell varieties.

There is little field experience with the use of articulating block systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for bank revetment and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. Tests conducted under this study confirm the applicability of these systems as a scour countermeasure for bridge piers.

Gabion Mattresses

Gabion mattresses are containers constructed of wire mesh and filled with rocks. The length of a gabion mattress is greater than its width, and the width is greater than its thickness. Diaphragms are inserted widthwise into the mattress to create compartments. Wire is typically galvanized or coated with polyvinyl chloride to resist corrosion, and either welded or twisted into a lattice. Stones used to fill the containers can be either angular rock or rounded cobbles; however, angular rock is preferred because of the higher degree of natural interlocking of the stone fill. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous armor layer.

The wire mesh allows the gabions to deform and adapt to changes in the bed while maintaining stability. Additionally, when compared to riprap, less excavation of the bed is required and smaller, more economical stone can be used. The obvious benefit of gabion mattresses is that the size of the individual stones used to fill the mattress can be smaller than stone that would otherwise be required to withstand the hydraulic forces at a pier.

There is limited field experience with the use of gabion mattress systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for structures such as in-channel weirs or drop structures, or for channel slope stabilization.

Tests conducted under this study confirm the applicability of these systems as a scour countermeasure for bridge piers.

Grout-Filled Mattresses

Grout-filled mattresses are composed of a double layer of strong synthetic fabric, typically woven nylon or polyester, sewn into a series of pillow-shaped compartments (blocks) that are connected internally by ducts. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Adjacent mattresses are typically sewn together prior to filling with grout.

The benefits of grout-filled mattresses are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the fabric forms and pumping them with concrete grout can be performed in areas where room for construction equipment is limited. When set, the grout forms a single-layer veneer made up of a grid of interconnected blocks. The blocks are interconnected by cables laced through the mattress before the grout is pumped into the fabric form. Flexibility and permeability are important functions for pier scour countermeasures. Therefore, systems that incorporate filter points or weep holes (allowing for pressure relief through the mattress) combined with relatively small-diameter ducts (to allow grout breakage and articulation between blocks) are the preferred products.

There is limited field experience with the use of grout-filled mattresses as a scour countermeasure for bridge piers. More frequently, these systems have been used for shoreline protection,

protective covers for underwater pipelines, and channel armoring where the mattresses are placed across the entire channel width and keyed into the abutments or banks.

Tests confirm that grout-filled mattresses can be effective scour countermeasures for piers under clear-water conditions. However, when dune-type bed forms were present, the mattresses were subject to both undermining and uplift, even when they were toed down below the depth of the bed-form troughs. Therefore, study results do not support the use of these products as pier scour countermeasures under live-bed conditions when dunes may be present.

Design Guidelines

To guide the practitioner in developing appropriate designs and ensuring successful installation and performance of pier scour armoring systems, the findings of Chapter 2 and recommendations of Chapter 3 are combined to provide a detailed set of stand-alone appendices:

- Appendix C, Guidelines for Pier Scour Countermeasures Using Rock Riprap
- Appendix D, Guidelines for Pier Scour Countermeasures Using Partially Grouted Riprap
- Appendix E, Guidelines for Pier Scour Countermeasures Using Articulating Concrete Block (ACB) Systems
- Appendix F, Guidelines for Pier Scour Countermeasures Using Gabion Mattresses
- Appendix G, Guidelines for Pier Scour Countermeasures Using Grout-Filled Mattresses

These application guidelines are presented in a format using the FHWA's Hydraulic Engineering Circular No. 23 (HEC-23) as a guide. As appropriate, these guidelines can be considered by AASHTO, FHWA, and state DOTs for adoption and incorporation into manuals, specifications, or other design guidance documents.

Conclusions and Recommendations

A review of the conclusions and recommendations outlined for each countermeasure type in Chapter 4 reveals a range of commonalities and contrasts for these systems. In most cases a filter layer is essential for successful performance of all pier scour protection. However for the countermeasures that incorporate rock particles, including gabions, the filter should extend only two-thirds of the distance from the pier to the perimeter of the armor. In contrast, ACB mats and grout-filled mattresses should have a filter underlying the full extent of the armor layer. In all cases, a granular filter should not be used when dune-type bed forms are expected in sand channels (i.e., under live-bed conditions). During testing, geotextile filters generally performed well for all countermeasure types when all components of the countermeasure system were properly designed and installed. For the ACB system, granular filters are not recommended under most conditions.

Geotextile sand containers are strongly recommended as a proven technique for placing a filter under water for riprap or partially grouted riprap, and gabion and grout-filled mattresses. For the ACB systems, a conventional geotextile filter should be used because placement and grading tolerances would be difficult to meet if geotextile containers were used as a filter.

For the pier scour countermeasures consisting of a thin veneer of armor (ACBs and the mattresses), termination details and, where necessary, anchor systems play a significant role in successful performance. It should be noted that testing of the grout-filled mattresses in both a "rigid" and "flexible" configuration yielded definitive results only for clear-water conditions. More research will be required before this countermeasure can be recom-

mended for pier scour protection under live-bed conditions. Similarly, the gabion mattress countermeasure, as tested, performed much better when the individual mattresses were physically connected to one another, compared to their performance as individual armor elements. However, laboratory testing could not provide guidance for the strength, composition, or longevity of the “tie.” For all three of these manufactured systems, the product provider should supply appropriate test results along with installation and materials guidance. This information is essential for successful performance of these products.

Suggested Research

In developing the design guidelines, additional information, data, or field experience with various countermeasure systems would have supported more detailed guidance or specificity in several areas. Suggestions for future research that would permit extending the recommendations of this study in these areas are summarized in Chapter 4.

CHAPTER 1

Introduction and Research Approach

1.1 Scope and Research Objectives

1.1.1 Background

Scour causes 60% of bridge failures in the United States. National studies by the 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.

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 (Alberto) in Georgia there were more than 500 state- and locally owned bridges with damage attributed to scour. Thirty-one of the state-owned bridges experienced from 15 to 20 ft of scour; those bridges had to be replaced. Georgia also recommended that, of more than 150 non-federal aid bridges identified as scour damaged, 73 bridges be repaired or replaced. Total damage to the Georgia highway system from Alberto was approximately \$130 million (Richardson and Davis 2001).

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 coded in the NBI 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 situation 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, some progress had been made in developing selection, design, and installation guidelines for pier scour countermeasures. For example, the publication of the first edition of HEC-23 in 1997 (Lagasse et al. 2001) was a first step toward identifying, consolidating, and disseminating information on countermeasure guidance. In addition, NCHRP Project 24-07, “Countermeasures to Protect Bridge Piers from Scour,” (Parker et al. 1998 and 1999) provided the initial results of laboratory and field research to evaluate the performance of pier scour countermeasures and to develop design and implementation guidance.

1.1.2 Scope of Research

From the background discussion, 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, 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 po-

tentially high cost, the most appropriate countermeasures must be selected, designed, and constructed.

The objectives of this research were to develop and recommend (a) practical selection criteria for bridge pier scour countermeasures; (b) guidelines and specifications for design and construction; and (c) guidelines for inspection, maintenance, and performance evaluation. The countermeasures considered included riprap, partially grouted riprap, articulating concrete block systems, gabion mattresses, grout-filled mattresses, and geotextile sand containers. In addition, issues related to riprap at skewed piers and mounded riprap were investigated.

NCHRP Project 24-07 was completed in July 1999 (Parker et al. 1998 and 1999). It involved extensive laboratory testing of riprap, cable-tied blocks, grout-filled bags, permeable sheet piles, pier-attached vanes, and submerged vanes. The laboratory testing demonstrated an enhancement in the performance of riprap and cable-tied blocks when used in conjunction with a geotextile filter. Other countermeasures investigated were shown to be less effective in resisting scour. Several countermeasures such as grout-filled mattresses, gabions, partially grouted riprap, and geotextile sand containers were not investigated. The results of NCHRP Project 24-07 provided significant insight into the behavior of the tested countermeasures; however, additional research of countermeasure performance was needed to develop specific countermeasure selection criteria, guidelines, and specifications.

NCHRP Project 24-07(2) (Phase 2) was initiated in April 2001 to refine the results of the NCHRP Project 24-07 (Phase 1) testing, 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 partially grouted riprap) was completed in December 2004. Continuation funding under the NCHRP Project 24-07(2) effort 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 was conducted at the Colorado State University (CSU) Engineering Research Center Hydraulics Laboratory in the March–December 2005 time frame. This report compiles the results of the Phase 1 and Phase 2 investigations.

1.2 Research Approach

1.2.1 Overview

NCHRP Project 24-07(2) is an extension of the work conducted by the University of Minnesota on NCHRP Project 24-07. In addition to providing additional testing for selected pier scour countermeasures, a fundamental goal was to develop

practical design guidance and specifications for implementation of a variety of countermeasures in field applications.

The development of analysis, design, and installation guidance for selected pier scour countermeasures must consider four important factors:

1. The nature of the mode of failure exhibited by each particular system. For example, typical failure modes associated with various countermeasures investigated in NCHRP Project 24-07 included (a) substrate winnowing, (b) overturning, or (c) flanking.
2. The threshold hydraulic conditions at which the mode of failure is initiated. Performance thresholds are often characterized by the local hydraulic conditions of shear stress, velocity, or a combination of both.
3. The need for ancillary system components or design features such as filter and/or bedding layers, structural connections, and system terminations such as toe downs and keys.
4. The integration of the physical characteristics of the system to the field site (i.e., constructability). Additionally, system characteristics may result in constraints when being placed under water versus in the dry.

Considering these factors, the linkage between laboratory studies and practical design/installation methods and procedures was established in the following generalized steps applicable to all bridge pier scour countermeasures investigated under this research project:

1. Review of available laboratory and field data, building on the experience gained not only from NCHRP Project 24-07 but including information from *NCHRP Research Results Digest 241: 1998 Scanning Review of European Practice for Bridge Scour and Stream Instability Countermeasures* (Lagasse 1999) and other sources as well.
2. Identification of design and installation issues and performance-related factors associated with each particular pier scour countermeasure warranting additional study.
3. Design and implementation of an experimental testing program that considers the available information developed in Step 1 and addresses stability and performance issues identified in Step 2.
4. Development of, and/or calibration to, a dominant-process design model that accurately reflects the mode of failure associated with the particular countermeasure. Typically, these models include local hydraulic conditions characterized by a combination of velocity and shear stress.
5. Development of a rational method for field layout and placement. Typically, layout guidelines are related to pier type and geometry, scour conditions, design hydraulic conditions, and sediment transport characteristics (bed

forms). Additionally, the physical characteristics of the system need to be considered for applications that call for placement under water versus in the dry, and when the system must be installed below the (unscoured) bed level.

6. Development of guidelines for selection and placement of ancillary system components, including filter and/or bedding requirements. Practical matters of installation often dictate that suitable options be developed for these components, particularly when applications must address placement underwater or in flowing water.

These six steps, applicable to any pier scour countermeasure, constituted the generalized research approach. Specific tasks to implement this approach are described in Section 1.3.

1.2.2 Transition from Project 24-07

The results of NCHRP Project 24-07 are documented in Volumes 1 and 2 of the Users Guide and Final Report (Parker et al. 1998 and 1999). NCHRP Project 24-07(2) was a continuation of the NCHRP Project 24-07 work and built on that effort.

The results of NCHRP Project 24-07 were considered in the development of the NCHRP 24-07(2) Research Work Plan. For example, flow-altering devices such as sacrificial piles, vanes, and permeable sheet piles placed upstream of a pier were thoroughly tested during Phase 1 (see Sections 6.2.4 through 6.2.6, Parker et al. 1998, Final Report). While no design suggestions are offered for flow-altering devices, because “none proved effective in and of themselves,” this negative finding is important to the practitioner. Many bridge owners are attracted by the apparently “easy” solution to the pier scour problem offered by such devices as sacrificial piles, scour collars, submerged vanes, etc. Such devices are generally impractical for field installations and ineffective in reducing scour. In fact, the NCHRP Project 24-07 Final Report notes some can even make scour worse. The laboratory findings that confirm the generally poor results to be expected from flow-altering devices are a very useful result of Phase 1, which should discourage further allocation of resources to an area with a relatively low expectation of success and encourage future research along more productive lines. Obviously, for Phase 2 the investigation of flow-altering devices was not an area for further testing.

1.2.3 Integration of European Technology

NCHRP Research Results Digest 241 (Lagasse 1999) identified a number of pier scour countermeasures that have potential application in the United States, including the use of partially grouted riprap and geotextile sand containers. During the scanning review, it was apparent that European counterparts considered riprap as a permanent pier scour countermeasure, whereas guidance in the United States cau-

tioned that riprap at a pier on an existing bridge may not provide adequate long-term protection. This difference in guidance can be attributed to the development in Europe of innovative riprap placement techniques, such as partially grouted riprap, and innovative techniques for placing an effective filter in deep or flowing water, such as geotextile sand containers. Both techniques were recommended in *NCHRP Research Results Digest 241* as high priority concepts for further evaluation. NCHRP Project 24-07(2) provided the opportunity to conduct that evaluation and develop guidelines and specifications, where applicable.

To facilitate integrating this European technology into U.S. practice, the research team had the consulting services of Dr. Michael Heibaum of the German Federal Waterways Engineering and Research Institute (*Bundesanstalt für Wasserbau* or BAW) who introduced the concepts of partially grouted riprap and geotextile containers to the scanning review team. In addition, Mr. Justus Trentmann of Gewatech-Soil and Hydraulic Engineering (*Gewatech Grund- und Wasserbau GmbH & Co. KG*) advised the research team on placement techniques, equipment, and specifications for cement grout as used in the construction industry in Germany.

Partial grouting of riprap with a cement slurry is one of several standard design approaches for permeable revetments used on German inland waterways. The scanning review team had the opportunity to observe wave tank testing of partially grouted riprap at the BAW laboratory in Karlsruhe, and some very general specifications are reported in *NCHRP Research Results Digest 241* (Lagasse 1999). An important consideration for partially grouted riprap is that construction methods must be closely monitored to ensure that the appropriate voids and surface openings are provided. Contractors in Germany have developed techniques and equipment to achieve the desired grout coverage and the right penetration.

Three documents are available from the BAW laboratory, which were not obtained during the 1998 scanning review (because they were available only in German):

- “Code of Practice – Use of Cement Bonded and Bituminous Materials for Grouting of Armor Stones on Waterways” (MAV) (BAW 1990)
- “Guidelines for Testing of Cement and Bitumen Bonded Materials for the Grouting of Armor Stones on Waterways” (RPV) (BAW 1991)
- “Code of Practice – Use of Standard Construction Methods for Bank and Bottom Protection on Waterways” (MAR) (BAW 1993b)

Task 1 of this project included preparing an English translation (see Reference Document [available from the TRB web site: www.trb.org/TRBNet/ProjectDisplay.asp?ProjectID=702]) of these manuals as a basis for developing the recom-

recommendations for Tasks 4 and 9. Thus, maximum advantage was derived from testing and development of guidelines and specifications in Germany before developing the testing program for this project. The BAW “Code of Practice – Use of Geotextile Filters on Waterways” (MAG) was available in English (BAW 1993a).

BAW has also issued a complete report, currently available only in German, on the testing of geotextiles (BAW 1994) including (1) impact tests (to determine punching resistance, e.g., when large stone is dropped on the geotextile); (2) abrasion tests; (3) permeability, clay clogging, and sand clogging tests; and (4) tests of material characteristics such as elongation and strength. This document was compared to ASTM standard test methods and AASHTO M 288 geotextile specifications. Because equivalent U.S. standard tests and specifications exist, this document was not translated.

Thus, our approach to integrating European technology with this project was to rely on the laboratory testing and design experience of the BAW laboratories and the construction experience of German contractors for a survey of current practice, assistance with the translation of guidelines, specifications, and technical articles currently available only in German, and advice as well as on-site consultation during selected testing phases of this study. In addition, by contacting bridge owners and operators of facilities in Germany where partially grouted riprap and geotextile sand containers had been installed, a field site visit was arranged for two research team members and an FHWA representative from the United States. This site visit was performed during Task 2 and provided a first-hand basis for recommending what, if any, additional testing was required to adapt European guidelines and specifications for these countermeasures to U.S. practice.

1.2.4 Continuation Funding

A request for \$350,000 in continuation funding was submitted to the AASHTO Standing Committee on Research (SCOR) in December 2003 for the FY 2005 Program with the following justification:

The first two phases of the research have been successful and will result in practical selection criteria and improved design guidelines and construction specifications for riprap, partially grouted riprap, and geotextile sand containers as pier scour countermeasures. The Panel is pleased with the research results to date and would like to increase the scope of the project to include additional laboratory testing for: (1) a near-prototype experiment of partially grouted riprap with geotextile bags as a filter, (2) mounded riprap with and without a filter, (3) the effect of skewed piers on countermeasures, (4) high velocity live-bed runs, and (5) gabion mattresses. The near-prototype test of partially grouted riprap with geotextile bags as filter is needed to validate the results of smaller flume studies conducted in Phase 2 as well as provide

water quality data associated with the placement of grout into the riprap. The other testing will allow the inclusion of additional counter measures as well as define their limitations and recommended ranges of use.

The request was approved by the SCOR in March 2004 and the additional work was authorized in December 2004.

1.3 Research Tasks

Considering the research approach discussed and outlined above, the following specific tasks were completed to accomplish project objectives. Except for Tasks 5 and 8 these tasks parallel, with minor modifications, those suggested in the original Research Project Statement.

1.3.1 Task 1 – Review Literature

The research team reviewed current practice, performance data, research findings, and other information related to riprap, partially grouted riprap, articulating concrete block systems, gabions, grout-filled bags and mattresses, geotextile sand containers, countermeasure filters, and the use of combinations of countermeasures. This information was assembled from technical literature and from unpublished experiences of engineers, bridge owners, and others.

1.3.2 Task 2 – Analyze Performance at Existing Installations

The research team selected existing installations of each countermeasure, countermeasure filter, and combinations of countermeasures listed in Task 1 and conducted a systematic analysis of the performance of the installations. The analysis of performance included an in-depth documentation of the hydraulic and structural design of the installations as well as documentation of the construction, maintenance, and inspection considerations of each installation and an evaluation of the performance of each installation to date. The installations were geomorphically diverse and included both successful and unsuccessful installations.

1.3.3 Task 3 – Identify Merits and Deficiencies

Based on the information gathered in Tasks 1 and 2, the research team identified the merits and deficiencies of current (1) selection criteria; (2) design specifications and guidelines; (3) construction specifications and guidelines; (4) maintenance and inspection guidelines; (5) performance evaluation guidelines; and (6) life-cycle cost information for each countermeasure, countermeasure filter, and combinations of countermeasures listed in Task 1.

1.3.4 Task 4 – Develop Draft Recommendations

The research team developed draft recommendations for items 1 through 6 in Task 3 for each countermeasure listed in Task 1 and devised a proposed methodology for developing items 1 through 6 into final products that meet the project objectives. In addition to the countermeasures identified in Task 1, the draft recommendations also addressed countermeasure filters and the use of combinations of countermeasures. The completeness of items 1 through 6 for each countermeasure, countermeasure filter, or combination of countermeasures varies depending on the adequacy of available information. NCHRP Project 24-07 provided information pertaining to riprap and cable-tied blocks. Depending on the results of Tasks 1 through 3, additional laboratory studies may be needed for partially grouted riprap, gabions, grout-filled bags and mattresses, geotextile sand containers, some countermeasure filters, and some combinations of countermeasures. The research team identified additional laboratory studies needed and conducted them as part of Tasks 7 and 7C.

1.3.5 Task 5 – Identify Bridge Owners

The research team developed a pier scour countermeasure selection methodology and identified bridge owners willing to evaluate (beta test) the methodology. This task originally envisioned field installation of tested countermeasures but was changed significantly by the NCHRP Project 24-07(2) panel at the Task 6 interim report meeting.

1.3.6 Task 6 – Submit Interim Report

The research team prepared and submitted an interim report describing the information developed in Tasks 1 through 5 and included a revised work plan as an appendix describing in detail how the remainder of the research would be accomplished. The interim report included draft guidelines and specifications incorporating the available findings from Tasks 1 through 5. The guidelines were in a format consistent with the second edition of FHWA HEC-23. The research team met with the NCHRP Project 24-07(2) panel to discuss the interim report and the revised work plan.

1.3.7 Task 7 – Perform Laboratory Studies

The research team performed appropriate laboratory studies identified in Task 4 for riprap, articulating concrete block systems, and partially grouted riprap, as well as countermeasure filters and combinations of countermeasures. The performance of these countermeasures was benchmarked against riprap.

1.3.8 Task 7C – Perform Laboratory Studies (Continuation Funding)

The research team performed appropriate laboratory studies for gabion mattresses, grout-filled mattresses, riprap at skewed piers, and mounded riprap. The research team also conducted an additional study of partially grouted riprap and geotextile sand containers at a near-prototype scale.

1.3.9 Task 8 – Perform Field Evaluation

The research team worked with the bridge owners identified in Task 5 to conduct a field evaluation (beta test) of a countermeasure selection methodology and revise the methodology based on comments received from bridge owners and the NCHRP Project 24-07(2) panel. This task was changed to reflect the changes in Task 5.

1.3.10 Task 9 – Finalize Draft Recommendations

Based on the results of Task 7 and 7C, the research team finalized the Task 4 draft recommendations.

1.3.11 Task 10 – Submit Final Report

The research team submitted a final report that documents the entire research effort and includes the final guidelines and specifications as stand-alone appendixes. In addition, they provided a companion executive summary that outlines the research results.

1.4 Report Organization

Findings from this research are available in two documents:

- **NCHRP Report 593** (this report), which contains
 - Findings from the review of current practice and field site visits
 - Overview of laboratory testing results
 - Interpretation and appraisal of findings and results
 - Conclusions and recommendations
 - Suggested research
 - Countermeasure selection methodology
 - Guidelines for using rock riprap
 - Guidelines for using partially grouted riprap
 - Guidelines for using articulating concrete block systems
 - Guidelines for using gabion mattresses
 - Guidelines for using grout-filled mattresses
- **Reference Document** (available on TRB web site [www.trb.org/TRBNet/ProjectDisplay.asp?ProjectID=702]), which contains
 - Detailed laboratory testing results
 - BAW Code of Practice translations

CHAPTER 2

Findings

2.1 Review of Current Practice

2.1.1 Introduction

Under NCHRP Project 24-07 (Phase 1), Parker et al. (1998) provided a review of the literature on pier scour and the state-of-the-practice knowledge on the design and placement of countermeasures around bridge piers to minimize scour. This section provides a review of the results of additional research that has been conducted since the Phase 1 review (post-1995). References are cited in Chapter 5 and a complete bibliography is provided in Appendix A.

In addition, a thorough review was conducted of current practice, performance data, research findings, and other information related to pier scour countermeasures, in general, with specific emphasis on riprap, partially grouted riprap, articulating concrete block systems, gabions, grout-filled bags and mattresses, geotextile sand containers, countermeasure filters, and the use of combinations of countermeasures. An example of a countermeasure combination is the use of grout-filled tubes (similar to those featured in Design Guideline 7, HEC-23 [Lagasse et al. 2001]) to seal the interface between a pier and an articulating concrete block countermeasure (similar to that described in Design Guideline 4, HEC-23). An alternative would be to consider the use of an articulating grout-filled mattress (as described in Design Guideline 5, HEC-23). This information was assembled from technical literature and from unpublished experiences of engineers, bridge owners, and others.

This review included a detailed analysis of the testing, evaluation, and results of NCHRP Project 24-07 as discussed in Section 1.2.2. The review also included all the material obtained by the scanning review team in 1998, some (but not all) of which is discussed in *NCHRP Research Results Digest 241* (Lagasse 1999). The foreign literature was reviewed as well, particularly guidelines, specifications, laboratory testing, and field evaluation results from Germany, as outlined in Section 1.2.3.

2.1.2 Scour at Bridge Piers

The basic mechanism causing local scour at piers is the formation of vortices (known as the horseshoe vortex) at their base (Figure 2.1). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is re-established between bed material inflow and outflow and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole (Richardson and Davis 2001).

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier called the wake vortex (Figure 2.1). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Factors that affect the magnitude of local scour depth at piers and abutments are (1) velocity of the approach flow, (2) depth of flow, (3) width of the pier, (4) length of the pier if skewed to flow, (5) size and gradation of bed material, (6) angle of attack of the approach flow to the pier, (7) shape of the pier, (8) bed configuration, and (9) ice formation or jams and debris.

Parker et al. (1998) provided a thorough review of the literature on the existing knowledge of scour around bridge piers. In addition to this review, comprehensive reviews on the causes and effects of pier scour are provided by the Centre for Civil Engineering Research and Codes (CUR) (1995),

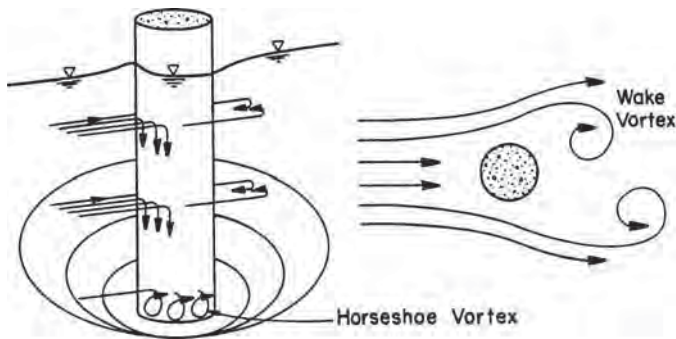


Figure 2.1. Schematic representation of scour at a cylindrical pier.

Lauchlan (1999), Melville and Coleman (2000), Richardson and Davis (2001), and Richardson et al. (2001). Melville and Coleman (2000) have compiled a summary table showing 20 different pier scour equations.

2.1.3 Riprap as a Pier Scour Countermeasure

An extensive review of experiments, model studies, and laboratory tests conducted prior to 1996 on the use of riprap as a scour countermeasure around bridge piers is provided in Parker et al. (1998). However, most of the research, model studies, and laboratory tests were conducted at small scales using clear-water conditions. The ratio of the typical riprap size to the bed sediment size was also considerably smaller than that found under field conditions. Additionally, very few of these studies provided practical guidelines for the design and placement of riprap around bridge piers.

Typically riprap used for pier scour protection is placed on the surface of the channel bed (Figure 2.2a), in a pre-existing scour hole, or in a hole excavated around the pier (Figure 2.2b). However, recent studies as described in the following sections, recommend placing the riprap layer at depth below the average bed level (Figure 2.2c).

Subsequent to the Phase 1 review, additional studies were conducted under both clear-water and live-bed conditions and added a wealth of information on the causes of riprap failure. Most of these studies modeled live-bed conditions, because a live-bed condition with the presence of mobile bed forms is very likely to occur during floods. Many of these studies provide guidelines on the stone size, placement, thickness, coverage, and filter requirements for installation of riprap layers around bridge piers based on additional laboratory experiments.

Modes of Pier Riprap Failure

Most of the early work on the stability of pier riprap is based on the size of the riprap stones and their ability to

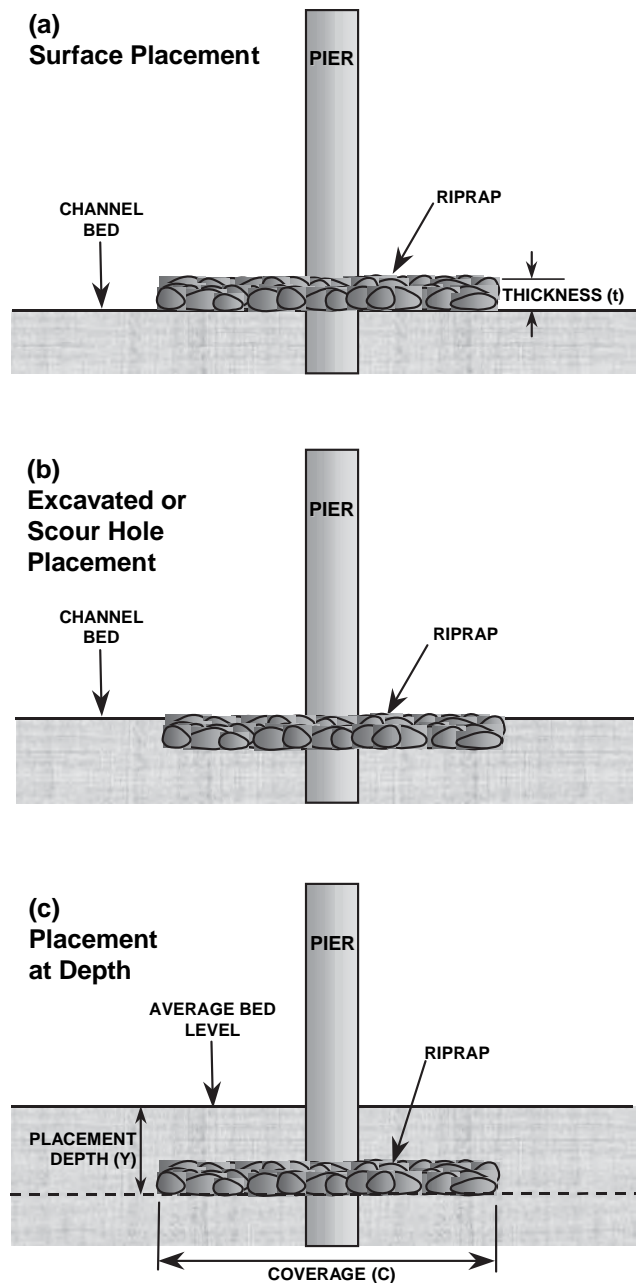


Figure 2.2. Typical pier riprap configurations.

withstand high approach velocities and buoyant forces. Parola (1995) notes 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. Due to the sensitivity of riprap size to velocity, Parola (1995) recommends that the stone size should be based on an acceptable flood level that would initiate riprap instability and that 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 (1995) under clear-water conditions with gradually increasing approach flow velocities defined three modes of failure:

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

Because live-bed conditions are more likely to occur during flood flows, Lim and Chiew (1996) conducted experiments to evaluate the stability of pier riprap under live-bed conditions with migrating bed forms. Subsequent research conducted by Melville et al. (1997), Lim and Chiew (1997, 2001), Parker et al. (1998), Lauchlan (1999), Chiew and Lim (2000), and Lauchlan and Melville (2001) indicates that bed-form undermining is the controlling failure mechanism at bridge piers on rivers with mobile bed forms, especially sand bed rivers.

The most important factors affecting the stability of the riprap layer under live-bed conditions were the turbulent flow field around the pier and the fluctuations of the bed level caused by migrating bed forms (e.g., dunes) past the pier. Lim and Chiew (1996) find that the three failure modes defined by Chiew (1995) under clear-water conditions also exist under live-bed conditions and that they may act independently or jointly with migrating bed forms to destabilize the riprap layer.

Once sediment transport starts and bed forms associated with the lower flow regime (i.e., ripples and dunes) begin to form, the movement of sediments at the edge of the riprap layer removes the support of the edge stones and allows the edge stones to be entrained in the flow (Lim and Chiew 1996). When the trough of a bed feature migrates past the riprap layer, stones slide into the trough, causing the riprap layer to thin. Depending on the thickness of the remaining riprap layer following stone sliding and layer thinning, winnowing may occur as a result of exposure of the underlying fine sediments to the flow. Winnowing can cause the entire remaining riprap layer to subside into the bed. With thicker riprap layers winnowing is not a factor and there is no subsidence.

Once the bed feature passes, the riprap layer may become buried, with the maximum depth of burial being dependent on the maximum size of the dunes. Thus, the maximum riprap scour level is closely related to the maximum scour depth (for a given flow), which is the sum of the equilibrium

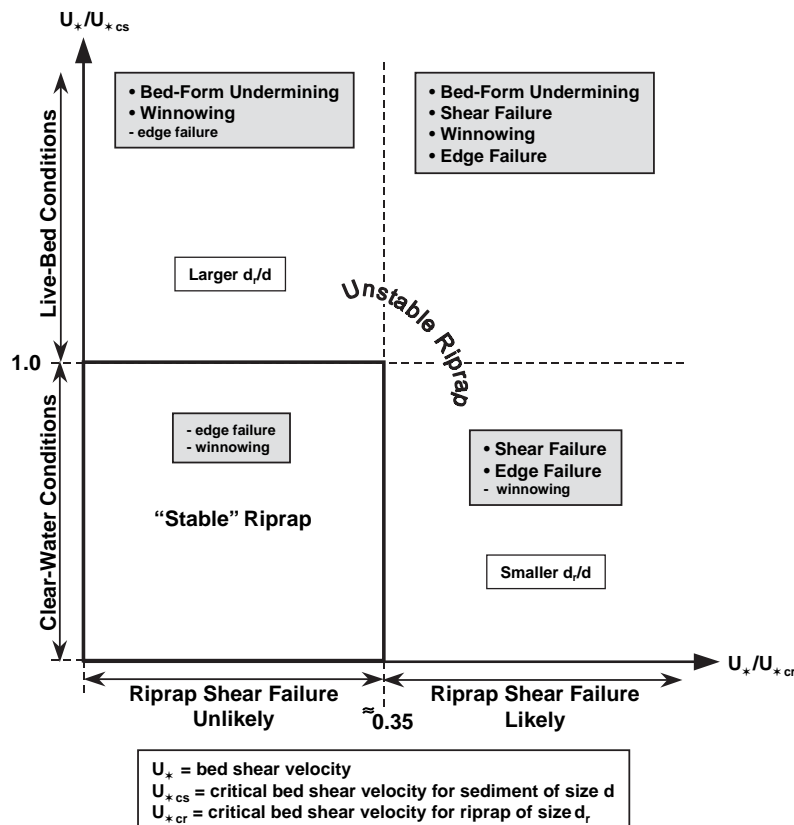
scour depth and the additional bed lowering contributed by the bed forms. The implication of Lim and Chiew's (1996) work is that a riprap layer cannot offer any resistance against scour when large bed forms are present.

Chiew (1995) shows that, under steady flow conditions, the inherent flexibility of a riprap layer can provide a self-healing process. As scour occurs and sediment is removed from around the riprap layer through the three modes of erosion described previously, the riprap layer, if it has sufficient thickness, can adjust itself to the mobile channel bed and remain relatively intact while providing continued scour protection for the pier.

When flow velocity is steadily increased, Lim and Chiew (1997) and Chiew and Lim (2000) note that riprap shear, winnowing, and edge erosion combine to cause either a total disintegration or embedment failure of the riprap layer in the absence of an underlying filter (either geotextile or granular). Total disintegration, which is characterized by a complete breakup of the riprap layer whereby the stones are washed away by the flow field, occurs when the self-healing ability of the riprap layer is exceeded by the erosive power created by higher flow velocity (Lim and Chiew 1997). According to Chiew and Lim (2000), embedment failure occurs when (1) the riprap stones are large compared to the bed sediment and local erosion around the individual stones causes them to embed into the channel bed (i.e., differential mobility) and (2) the riprap stones lose their stability as bed forms pass and the stones drop into the troughs of the migrating bed forms (i.e., bed feature destabilization). Lim and Chiew (1997) propose a semi-empirical equation based on the critical shear velocity for bed sediment entrainment to distinguish between the total disintegration and embedment modes of failure.

Toro-Escobar et al. (1998) present the results of experiments conducted by three cooperating research groups (University of Auckland, Nanyang University, and St. Anthony Falls Laboratory) under NCHRP Project 24-07 (Parker et al. 1998), which verified the four modes of riprap failure (i.e., riprap shear, winnowing, edge failure, and embedment or settlement due to bed-form passage) defined by Lim and Chiew (1996, 1997). The experiments indicated that these processes, which occur even though the flow is unable to entrain the riprap, can produce less effective protection than that assumed in existing designs. In some cases, the riprap settled to the level of the ambient bottom of the bed-form troughs, and, in other cases, the riprap settled to levels slightly above those that would prevail in the complete absence of riprap.

Lauchlan (1999), Lauchlan and Melville (2001), and Lim and Chiew (2001) provide the most comprehensive parametric studies to date on the four modes of pier riprap failure. The conditions under which the failure mechanisms for riprap protection at bridge piers occur are summarized in



Source: modified from Lauchlan (1999)

Figure 2.3. Summary of pier riprap failure conditions for clear-water and live-bed regimes.

Figure 2.3. The figure shows that riprap shear, winnowing, and edge failures are observed in all flow conditions, whereas bed-form undermining or destabilization occurs only under live-bed conditions. The potential for winnowing failure increases with U^*/U_{*cs} , while the potential for edge failures increases with U^*/U_{*cr} . Riprap shear failure occurs only for $U^*/U_{*cr} > 0.35$ and winnowing is more likely at larger relative riprap size to bed sediment size ratios (d_r/d).

Sizing of Pier Riprap

In addition to the literature review conducted by Parker et al. (1998), comprehensive reviews of the literature on sizing of riprap for bridge piers have been conducted by Fotherby (1995), CUR (1995), Lauchlan (1999), Melville and Coleman (2000), and Lauchlan and Melville (2001).

Riprap, which is the most commonly used pier scour countermeasure, often consists of large stones placed around a pier to armor the bed at the pier. This armoring prevents the strong vortex flow at the front of the pier from entraining bed sediment and forming a scour hole. The ability of the riprap layer to provide scour protection is, in part, a function of stone size, which is a critical factor in terms of shear failure.

The stability of riprap is typically expressed in terms of the stability number, N_{sc} , which is used in numerous equations to size riprap. Riprap stone size is designed using the critical velocity near the boundary where the riprap is placed. However, many of the pier riprap sizing equations are modified versions of bank or channel protection equations and, therefore, the use of this approach has limitations when applied at bridge piers because of the strongly turbulent flows near the base of a pier. Most of the remaining equations are based on threshold of motion criteria or empirical results of small-scale laboratory studies conducted under clear-water conditions with steady uniform flow.

Table 2.1 provides a summary of most of the available equations, reduced to a common form, for sizing riprap to protect bridge piers against scour. A comparison of the various equations for a range of Froude numbers from 0.2 to 0.6 with coefficients for round-nosed piers and sediment particle specific gravity (S_s) of 2.65 indicates that there is a wide range of predicted riprap sizes for any given flow conditions (Figure 2.4). Lauchlan (1999), Melville and Coleman (2000), and Lauchlan et al. (2000a) compare these equations in detail. Since there is a lack of consistency among the methods, Melville and Coleman (2000) recommend the use of the

Table 2.1. Equation for sizing riprap at bridge piers.

Reference	Equation	Standard Format (for comparison)	Comments
Bonasoundas (1973)	$d_{r50} \text{ (cm)} = 6 - 3.3V + 4V^2$		Equation applies to stones with $S_s = 2.65$ V = mean approach velocity (m/s)
Quazi and Peterson (1973)	$N_{sc} = 1.14 \left(\frac{d_{r50}}{y} \right)^{-0.2}$	$\frac{d_{r50}}{y} = \frac{0.85}{(S_s - 1)^{1.25}} Fr^{2.5}$	N_{sc} = critical stability number $= V^2/[g(S_s - 1)d_{r50}]$ Fr = Froude number of the approach flow $= V/(gy)^{0.5}$
Breusers et al. (1977)	$V = 0.42\sqrt{2g(S_s - 1)d_{r50}}$	$\frac{d_{r50}}{y} = \frac{2.83}{(S_s - 1)} Fr^2$	S_s = specific gravity of riprap stones y = mean approach flow depth
Farraday and Charlton (1983)	$\frac{d_{r50}}{y} = 0.547Fr^3$	$\frac{d_{r50}}{y} = 0.547Fr^3$	
Parola et al. (1989)	$\frac{d_{r50}}{y} = \frac{C^*}{(S_s - 1)} Fr^2$	$\frac{d_{r50}}{y} = \frac{C^*}{(S_s - 1)} Fr^2$	C^* = coefficient for pier shape; C^* = 1.0 (rectangular), 0.61 (round-nose)
Breusers and Raudkivi (1991)	$V = 4.8(S_s - 1)^{0.5} d_{r50}^{1/3} y^{1/6}$	$\frac{d_{r50}}{y} = \frac{0.278}{(S_s - 1)^{1.5}} Fr^3$	
Austrroads (1994)	$\frac{d_{r50}}{y} = \frac{0.58K_p K_v}{(S_s - 1)} Fr^2$	$\frac{d_{r50}}{y} = \frac{0.58K_p K_v}{(S_s - 1)} Fr^2$	K_p = factor for pier shape; $K_p = 2.25$ (round-nose), 2.89 (rectangular) K_v = velocity factor, varying from 0.81 for a pier near the bank of a straight channel to 2.89 for a pier at the outside of a bend in the main channel
Richardson and Davis (1995)	$d_{r50} = \frac{0.692(f_1 f_2 V)^2}{(S_s - 1)2g}$	$\frac{d_{r50}}{y} = \frac{0.346f_1^2 f_2^2}{(S_s - 1)} Fr^2$	f_1 = factor for pier shape; $f_1 = 1.5$ (round-nose), 1.7 (rectangular) f_2 = factor ranging from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier in the main current of a bend
Chiew (1995)	$d_{r50} = \frac{0.168}{\sqrt{y}} \left(\frac{V}{U_* \sqrt{(S_s - 1)g}} \right)^3$	$\frac{d_{r50}}{y} = \frac{0.168}{(S_s - 1)^{1.5} U_*^3} Fr^3$ $U_* = \frac{0.3}{K_d K_y}$	$K_y = 0.783 \left(\frac{y}{b} \right)^{0.322} - 0.106$ $0 \leq (y/b) < 3$ $K_y = 1$ $(y/b) \geq 3$ $K_d = 0.398 \ln \left(\frac{b}{d_{r50}} \right) - 0.034 \left[\ln \left(\frac{b}{d_{r50}} \right) \right]^2$ $1 \leq (b/d_{r50}) < 50$ $K_d = 1$ $(b/d_{r50}) \geq 50$ K_y = flow depth factor K_d = sediment size factor

Richardson and Davis (1995) (see Lagasse et al. 2001) and Lauchlan (1999) methods for sizing suitable riprap for bridge pier protection, because they lead to conservatively large riprap relative to the other methods. Melville and Lauchlan (1998) use these methods to assess riprap size requirements for the Hutt Estuary Bridge in New Zealand and were found to provide good agreement with model study results (Lauchlan et al. 2000b).

Only recently have studies been conducted to address riprap size with regard to stability at bridge piers under live-bed conditions. Stone size affects shear failure because this failure mode occurs when high flow velocity results in entrainment of

the riprap stones. Stone size also influences winnowing, because an increase in stone size produces a concomitant increase in the size of the voids through which bed material is easily winnowed, particularly in thinner riprap layers. This effect decreases with increasing riprap layer thickness. In terms of edge failure and bed-form destabilization, increasing stone size requires increasing bed-form size to cause the same level of damage for a given layer configuration.

In a comprehensive parametric study, Lim and Chiew (2001) note that the use of very large stones in pier riprap, which has been shown to be beneficial in clear-water conditions, provides little benefit under live-bed conditions,

Table 2.1. (Continued).

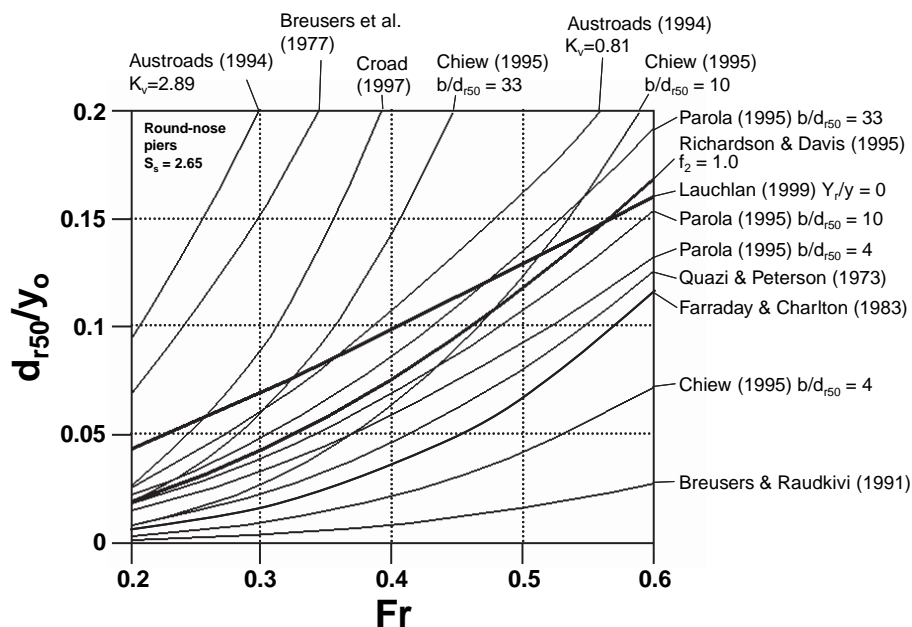
Reference	Equation	Standard Format (for comparison)	Comments
Parola (1993, 1995)	Rectangular: $N_{sc} = 0.8$ $20 < (b_p/d_{r50}) < 33$ $N_{sc} = 1.0$ $7 < (b_p/d_{r50}) < 14$ $N_{sc} = 1.0$ $4 < (b_p/d_{r50}) < 7$ Aligned Round-Nose: $N_{sc} = 1.4$	$\frac{d_{r50}}{y} = \frac{f_1 f_3}{(S_s - 1)} Fr^2$	b_p = projected width of pier f_1 = pier shape factor; $f_1 = 1.0$ (rectangular), 0.71 (round-nose if aligned) f_3 = pier size factor = $f(b_p/d_{r50})$: $f_3 = 0.83$ $4 < (b_p/d_{r50}) < 7$ $f_3 = 1.0$ $7 < (b_p/d_{r50}) < 14$ $f_3 = 1.25$ $20 < (b_p/d_{r50}) < 33$
Crood (1997)	$\frac{V}{A\sqrt{(S_s-1)gd_{r50}}} =$ $1.16\left(\frac{y}{d_{r50}} - 2\right)^{1/6}$ $d_{r50} = 17d_{b50}$	$\frac{d_{r50}}{y} \left(1 - \frac{2d_{r50}}{y}\right)^{0.5} =$ $\frac{0.641}{A^3(S_s - 1)^{1.5}} Fr^3$ $d_{r50} = 17d_{b50}$ Use larger of d_{r50} sizes given by the two equations	A = acceleration factor; $A = 0.45$ (circular and slab piers), $A = 0.35$ (square and sharp-edged piers) d_{b50} = median size of bed material. Equation given for factor of safety = 1.25, as recommended by Crood (1997)
Lauchlan (1999)	$\frac{d_{r50}}{y} = 0.3S_s \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	$\frac{d_{r50}}{y} = 0.3S_s \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	S_s = safety factor, with a minimum recommended value of 1.1 Y_r = placement depth below bed level

Source: Melville and Coleman (2000)

especially at the upper end of the dune regime where large stones offer no additional protection against pier scour. This finding is in contrast to clear-water experiments conducted by Parola (1995) who suggests that large riprap may act to dissipate pier-induced vortices, especially when riprap size approaches the size of the vortices. He reasons that because pier-induced vortices are a function of pier diameter, the stability number, N_{sc} , should increase when the

rock size approaches the pier diameter. However, experimental observations by Lim and Chiew (2001) under live-bed conditions show that large riprap stones, once they are exposed to the flow, act as additional blockages to flow, thereby generating high local turbulence at the pier and resulting in significant riprap degradation.

Lim and Chiew (2001) also show that no matter how large the riprap stones are, they will invariably become embedded



Source: modified from Lauchlan (1999)

Figure 2.4. Comparison of equations for sizing riprap at round-nose bridge piers.

into the scour hole at the upper end of the dune regime as a result of bed-form passage. As bed forms pass, the riprap layer composed of large stones deforms and the stones slip or slide into the trough, thus increasing the number and spacing of voids which, in turn, contributes to winnowing of the bed material and, ultimately, embedment of the stones.

Lauchlan and Melville (2001) conducted experiments on surface-placed riprap of various sizes where the depth of local scour was recorded for each riprap size at specific flow velocities. Riprap failure was considered to have taken place when more than 20% of the maximum unprotected scour depth occurred in the riprap layer (i.e., $d_r/d_{s_{max}} > 20\%$) over the experimental period. Past practices have been to size riprap such that no movement of the material would occur at the design flow velocity, which has led to oversizing of riprap. However, the data from Lauchlan and Melville (2001) provide larger critical stone sizes for particular flow velocities than many of the previous investigations because of the effects of bed-form destabilization of riprap, which was not evaluated in the fixed-bed flume models of many previous researchers.

Recent studies by Lauchlan and Melville (2001) and Lim and Chiew (2001) provide additional information on sizing of riprap around bridge piers under live-bed conditions. Based on the results of their study, Lauchlan and Melville (2001) refined the equation for the minimum critical stone size in relation to flow velocity as defined by Lauchlan (1999). The equation for the minimum stone size is

$$\frac{d_{50}}{y_o} = K_D K_S K_\alpha K_Y 0.3 F^{1.2} \quad (2.1)$$

where

- d_{50} = Median riprap size, ft (m)
- y_o = Undisturbed approach flow depth, ft (m)
- K_D = K-factor for pier diameter-to-bed material ratio (D)
- K_S = K-factor for pier shape (S)
- K_α = K-factor for pier alignment
- K_Y = K-factor for riprap placement depth (Y)
- F = Froude number

Because inadequate data were available to determine K_S , K_D , and K_α from the study, these factors were set to unity.

However, Fotherby and Ruff (1998a) have shown K_D (K-factor for pier diameter-to-bed material ratio) to be a significant factor, especially when riprap diameter is comparable to pier width. Since Lauchlan and Melville (2001) used surface-placed riprap, the K_Y factor was not valid. They used the data from their study to estimate the riprap placed at depth, which allows Equation 2.1 to be rewritten as

$$\frac{d_{50}}{y_o} = 0.3 \left(1 - \frac{Y}{y_o} \right)^{2.75} F^{1.2} \quad (2.2)$$

For high Froude numbers (Figure 2.4), the riprap sizes predicted by the Lauchlan and Melville (2001) equation are similar to those given by equations from Richardson and Davis (1995) and Parola (1995). Their data also indicate that riprap size for a given Froude number decreases with increasing placement depth.

To determine the d_{50} size of pier riprap, HEC-18 (Richardson and Davis 1995) and HEC-23 (Lagasse et al. 2001) recommend using the rearranged Isbash equation to solve for stone diameter for fresh water:

$$d_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} \quad (2.3)$$

where

- d_{50} = Median stone diameter, ft (m)
- K = Coefficient for pier shape (1.5 for round-nose pier, 1.7 for rectangular pier)
- V = Velocity on pier, ft/s (m/s)
- S_s = Specific gravity of riprap (normally 2.65)
- g = Acceleration due to gravity, ft/s² (m/s²)

To determine the velocity on the pier, the average channel velocity (Q/A) is multiplied by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a sharp bend.

Riprap Placement Level, Coverage, Thickness, and Grading

Specifications and guidance on the placement level, areal coverage, thickness, and gradation of a riprap layer placed around a bridge pier vary widely. Table 2.2 summarizes many of the methods used to estimate the extent of coverage, thickness, level of placement, and gradation requirements for pier riprap.

Placement Level. As previously discussed, most studies of pier riprap failure were conducted under clear-water conditions. In most of these studies, the riprap layer was placed on top of the bed surface or buried with the top of the riprap layer flush with the bed surface. Many of the guidelines for placement of riprap are based on considerations of riprap for bank protection. Parker et al. (1998) note that even though the placement level of the riprap layer with respect to the channel bed is believed to be an important factor in the stability of the layer, there are no generally accepted design criteria available for this factor and, in particular, there are conflicting recommendations for the finished level of riprap protection.

Riprap used for pier scour protection is usually placed on the surface of the channel bed (Figure 2.2a) because of the ease

Table 2.2. Methods to estimate riprap extent, gradation, and filter requirements for riprap at bridge piers.

Reference	Riprap Extent		Level	Gradation
	Coverage (C)	Thickness (t)		
Bonasoundas (1973)	Semi-circular upstream shape (radius 3b), semi-elliptical downstream shape; overall length 7b	b/3		
Neill (1973)	Project around the nose of the pier by a distance = 1.5b	>2d ₅₀		
Posey (1974)	1.5b to 2.5b in all directions from the pier face			
Hjorth (1975)	Length = 6.25b, width = 3b, circular arc upstream, triangular shape downstream			
Breusers et al. (1977)	2b from pier face	3d ₅₀	Some distance below bed level to prevent excessive exposure	
Lagasse et al. (2001)	Width > 5b	> 3d ₅₀	Top of riprap at bed level	d _{r50} ≥ 0.5d _{rmax}
Chiew (1995)	$\frac{C_r}{D} \geq 12.5 \frac{V}{V_c} - 2.75$ D = pier diameter			
Parola (1995)	Semi-circular upstream (radius b _p), triangular downstream; overall length = 7b _p			
Croad (1997)	>5.5b _p , of which 1.5b _p is upstream of the upstream face of the pier	2d ₅₀		d _{rmax} ≤ 2d _{r50} d _{r50} ≤ 2d _{r15}
Lauchlan (1999)	1b to 1.5 b in all directions from the pier face. Synthetic filter (if placed) should have lateral extent about 75% of the lateral extent of the riprap layer	2d ₅₀ to 3d ₅₀	A factor for level of placement (Y _r) included in riprap sizing equation	0.5d _{rmax} < d _{r50} d _{r50} < 2d _{r15}
Brown and Clyde (1989)	2b from pier face	≥3d ₅₀	Place mat below streambed a depth equivalent to the expected scour	
Fotherby (1995) Fotherby and Ruff (1998a)	1.5b _a minimum (b _a = adjusted pier width)	2D _u min. (D _u = riprap unit diameter)	Top of riprap installed level with streambed or within 2D _u if approach flow velocity is adjusted	
CUR (1995)	3b in the upstream direction and 4b on both sides and in the downstream direction (as measured from the pier face)	2b	On or flush with the streambed surface	
Parker et al. (1998)	Total lateral coverage (edge to edge) = 4b for excavated or existing scour hole = 5b for placement on streambed	at least 3d ₅₀		
Lim and Chiew (2001)	FHWA coverage of 2b from pier face (extent of coverage has no effect at upper dune regime)	>1.5d ₅₀ or d _{r100}		

Source: modified from Melville and Coleman (2000)

and lower cost of placement and because it is more easily inspected. Parola (1995) hypothesized that mounded riprap on the bed surface may have an increased capacity to resist erosion because it alters the approach flow vertical velocity distribution such that the vortex systems created by the pier have a lower capacity to destabilize the riprap. However, mounding riprap around a bridge pier is unacceptable for design, in most cases, because it constricts

flow, captures debris, and increases scour at the margins of the pier protection.

Many studies suggest that riprap be placed in a flat layer on the bed surface, in an existing scour hole with the top nearly flush with the bed, or in a pre-excavated hole around the pier with the top of the layer level with the bed. The FHWA (Lagasse et al. 2001; Richardson and Davis 1995) recommends placing the top of the riprap layer flush with the chan-

nel bed for inspection purposes (Figure 2.2b). The European practice and the preferred practice of many state DOT maintenance departments in the United States is to place the layer on top of the bed surface (Figure 2.2a), preferably with an underlying filter layer or geotextile to deter the effects of winnowing of the underlying bed sediments.

Most of the studies on the stability of riprap around bridge piers prior to the Phase 1 study by Parker et al. (1998) were conducted under clear-water conditions with the top of the riprap layer placed level with the channel bed. Many of these studies concentrate primarily on riprap size, layer thickness, and filter requirements when evaluating pier riprap stability (Parola 1995; Fotherby 1995; Lim and Chiew 1996; Yoon and Yoon 1997; Fotherby and Ruff 1998a, 1998b; Ruff and Nickelson 1998). The pioneering study by Laursen and Toch (1956) was one of the first studies to propose that riprap used at bridge piers should be placed well below the streambed. Breusers et al. (1977) recommends that riprap near bridge piers would perform most successfully when placed at the trough elevation of the largest bed forms.

As discussed earlier, a live-bed condition with migrating bed forms is more likely to occur during floods and is now believed to be the most important contributor to pier riprap failure. Therefore, many of the experimental studies conducted over the last several years have been concerned with the processes of pier riprap failure under live-bed conditions, and several have addressed the placement level of the riprap layer with regard to the passage of mobile bed forms. Lim and Chiew (1996) propose an empirical equation to compute the maximum displaced riprap level, which is the level contributed jointly by the pier (i.e., equilibrium pier scour depth) and by the passage of the largest dunes (i.e., the dune trough level) just prior to the arrival of the transition to a plane bed (i.e., the transition to upper regime conditions). Studies by Parker et al. (1998) note that riprap performance improved when the top of the riprap layer was buried below the bed surface, but do not provide any guidance on recommended depth of burial.

The comprehensive study conducted by Lauchlan (1999) indicates that placing the riprap layer at depth (Figure 2.2c) was shown to improve the performance of the layer for a specific flow velocity, and that the deepest placement level tested provided the greatest reduction in local scour depths in the majority of tests. Based on experimental results, Lauchlan recommends the use of a placement depth factor, K_Y , to describe the improved performance of riprap when it is placed below the average bed level (see Equations 2.1 and 2.2 for definition of K_Y). Lauchlan suggests that K_Y be used when the ratio of the depth of placement, Y , to the mean flow depth, y_o , is between 0.0 and 0.6. Based on these results, Lauchlan (1999) and Melville and Coleman (2000) recommend that the riprap layer should be placed at about the lowest dune trough level expected. Although Lim and Chiew (2001) find

that riprap layer degradation decreases with greater depth of placement, they indicate that the placement level of a riprap layer ceases to provide any benefit to riprap layer stability at approximately the upper end of the dune regime.

Areal Coverage. As shown in Table 2.2, the recommended coverage varies with pier shape and can extend as little as one pier width from the pier face to as much as seven times the pier width depending on location around the pier. Most studies recommend that the coverage of the riprap layer extend at least to the edges of the predicted or existing scour hole. Various studies suggest shaping the riprap layer into a rectangle, pear, teardrop, or horseshoe shape. According to Lauchlan (1999) in most of the studies conducted using riprap filter layers, “it is unclear as to whether testing of the recommendations [for filter layer shape] was undertaken, which is doubtful, and little reasoning for the proposed shapes is given.”

Layer Thickness. Most of the studies reviewed in the previous paragraphs suggest that thickness of the riprap layer placed around bridge piers should be between two to three times the median stone size of the riprap (Table 2.2). Riprap performance was found to increase significantly with an increase in thickness from $2d_{r50}$ to $3d_{r50}$ (Parker et al. 1998). Melville and Coleman (2000) indicate that there is as much as a 70% reduction in local scour associated with an increase in thickness from $1d_{r50}$ to $3d_{r50}$.

Thin layers tend to fail under the process of winnowing of the underlying bed sediments and the passage of mobile bed forms (Chiew 1995; Lim and Chiew 1996; Parker et al. 1998). Experiments by Lim and Chiew (1996) indicate that thick riprap layers still become thin at the edges, but will not subside into the bed under live-bed conditions. They also found that thicker layers are able to self-heal under the modes of failure previously described. A thick riprap layer behaves similarly to a riprap layer of regular thickness with an underlying filter; winnowing and subsidence are unable to take place because flow is unable to pass through the interstices of the riprap layer. However, riprap stones can still slide into the trough of passing dunes and may be swept away under higher velocities. The parametric study by Lim and Chiew (2001) indicates that riprap layer thickness has no influence on the stability of the layer with the passage of very large dunes.

Gradation. Very few of the previously discussed studies have specifically examined the effects of riprap gradation on riprap layer stability. However, most studies suggest that a graded riprap layer will be more likely to withstand the effects of bed sediment winnowing than one composed of equidimensional stones. A few studies shown in Table 2.2 provide some guidance on riprap gradation. Brown and Clyde (1989) provide gradation limits and classes, and CUR (1995) provides gradation class requirements and grading curves for general use in riprap revetments.

Summary. Based on much of the information in Table 2.2, Melville and Coleman (2000) provide the following recommendations for riprap protection at bridge piers:

- Riprap size: based on Lauchlan (1999) equation (Equation 2.2) for sizing riprap
- Riprap layer thickness: $t = 2d_{r50}$ to $3d_{r50}$
- Coverage of riprap layer: width = 3 to 4 pier widths, or 1 to 1.5 pier widths from pier face
- Placement level: at about lowest dune trough level
- Grading: $0.5d_{r\max} < d_{r50} < 2d_{r15}$
- Synthetic filter layer: lateral extent should be about 75% of lateral extent of riprap layer
- Inverted stone filter layer: $t = d_{r50}$ with grading according to Terzaghi criteria

Riprap Filter Requirements

Two kinds of filters are used in conjunction with bridge pier riprap: granular (stone) filters and geotextile filters. Stone filters are composed of rock that may or may not be graded, and have a median size that is smaller than the overlying riprap but large enough to be more permeable than the underlying bed material. Geotextiles are permeable textiles,

meshes, and nets that are either synthetic or biodegradable (not recommended). Geotextiles can be woven, non-woven, or knitted. Woven geotextiles have evenly spaced fibers that are at right angles to form regularly spaced holes. Non-woven geotextiles have fibers or filaments that are randomly placed to form a wide range of hole sizes. Knitted geotextiles consist of immovable fibers that confer a high degree of strength and flexibility to the fabric. The durability of a geotextile is dependent on the type of fiber used and its mechanical, filtration, and chemical properties.

The importance of the filter component of a riprap installation should not be underestimated. Geotextiles and/or aggregate underlayers are used to perform the filtration function. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of the filter layer. In cases where the base soil is composed primarily of relatively large particles (coarse sands and gravels), a filter layer may not be necessary.

Careful design, selection, and installation of the appropriate filter material all play an important role in the overall performance of riprap. Figure 2.5 provides schematic illustrations of the three most typical types of riprap filter configurations.

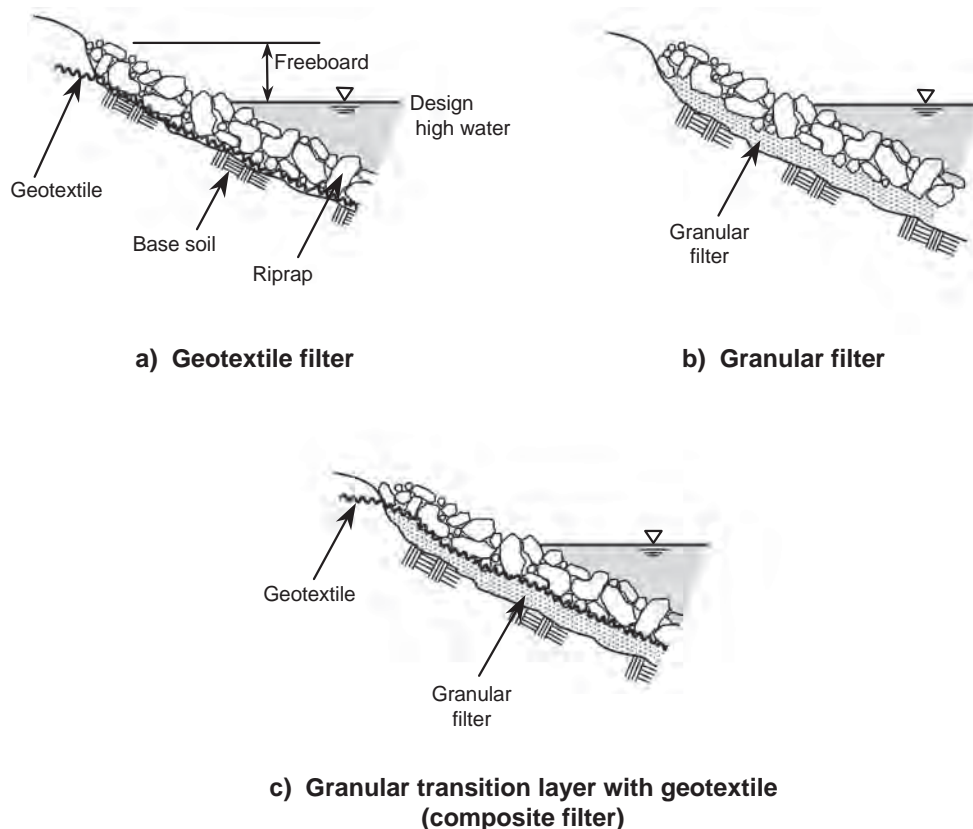


Figure 2.5. Channel cross sections showing common riprap/filter configuration.

The primary roles of a filter component are to (1) retain the soil particles, while (2) providing a zone for the free flow of water through the interface between the riprap armor and the underlying soil. The soil retention function argues for very small pores in the filter, whereas maintaining a large permeability of the filter argues for larger pores, and lots of them. Both of these two contrary objectives must be met to achieve an effective functional balance between retention and permeability.

Filters assist in maintaining intimate contact between the revetment and the base soil by creating a stable interface. Depending upon the internal stability of the soil, several processes can occur over time at this interface. The filter pore size and the base soil stability influence these processes.

As an example, consider the process of “piping.” Piping is basically the washing away of very fine particles, resulting in greater void space in the underlying soil structure. Piping is more likely to occur in non-cohesive/unstable soils that are in contact with a filter that has large openings. The large openings do not retain the smaller particles and therefore these particles are removed by seepage and pressure fluctuations, leaving only the larger particles. This process increases the potential for soil erosion by weakening the underlying soil structure.

The reverse can occur when the pores of the filter are so small that they retain virtually all the particles of the base soil. If the base soil is internally unstable, the finest particles will continue to migrate with the seepage flow until a clogged layer is built up against the filter. This lower permeability zone will eventually create a barrier to flow, and excess uplift pressures can be created beneath the filter. A detailed discussion of the filter requirements is presented in Section 3.12.

In Europe, it is common practice to use fascine mats as a means of placing a geotextile filter in deep water. Fascine mattresses are composed of natural woody material woven in bundles to form a matrix that is placed over a geotextile and then floated into position and sunk into place by dropping riprap on it from a barge (Pagán-Ortiz and Lagasse 1999; Lagasse et al. 2001).

Lauchlan (1999) provides a comprehensive review of the literature on the use of granular and synthetic filters and the criteria for their use with pier riprap. General guidelines on the design and use of granular and fabric filters are provided in Brown and Clyde (1989). Escarameia (1998), Holtz et al. (1995), and Pilarczyk (2000) provide detailed information on the types of filters, potential applications, and specific guidelines on the selection and installation of geotextile filters. CUR (1995) also provides detailed information on the properties, design, and placement of filters used in conjunction with riprap in Europe. Brauns et al. (1993) provide a comprehensive review of the design, placement, applications, and problems associated with the use of filters in geotechnical and hydraulic engineering.

Some studies suggest that a filter may be unnecessary if the riprap layer is of sufficient thickness (Lim and Chiew 1996, 1997; Toro-Escobar et al. 1998; Lauchlan 1999). Yet, a majority of the research on the stability of riprap at bridge piers to date indicates that the use of an underlying filter layer significantly increases the stability of the riprap layer. Many of the more recent experimental studies have evaluated the effects of a filter layer placed below a riprap layer on the stability of the riprap layer under live-bed conditions.

In general, granular filter layers should be of a gradation, size, and thickness sufficient to deter the effects of winnowing of the underlying bed sediments. Geotextiles should also have an effective pore size sufficiently small to block the passage of bed sediments, but have large enough permeability to deter or withstand buoyant forces and potential pressure gradients in the surface and subsurface in the area of the pier.

Parker et al. (1998) determined that using a geotextile with the same areal coverage as the riprap layer it is placed under results in relatively poor performance of the riprap at bridge piers. As a result of the effects of live-bed conditions described previously, the riprap at the edges tended to roll, slide, or be plucked off exposing the underlying geotextile and ultimately resulting in failure of the riprap layer as successive bed forms pass and pluck more stones from the riprap layer. The failure of the geotextile was due in part to the impermeability of the fabric leading to the buildup of uplift forces and the creation of a bulge under the fabric, which contributed to the loss of riprap stones. In addition, the loss of the edge riprap and exposure of the geotextile allowed the geotextile to fold back on itself further reducing the stability of the riprap. If the geotextile was not sealed to the pier face, winnowing around the pier face resulted in a scour hole around the pier face and caused the geotextile and stones at the interface to fall into the scour hole.

For bridge piers, Parker et al. (1998) determined that the tendency for riprap to settle was arrested when (1) the geotextile has two-thirds the areal coverage of the riprap, (2) the geotextile is sufficiently permeable, and (c) the geotextile is sealed to the pier. Lauchlan (1999) recommends that the geotextile have an areal coverage of 75% of the riprap layer so that the edges of the geotextile will be anchored when the edge stone of the riprap layer slide into the trough of passing bed forms.

However, placement of a filter layer at a bridge pier under riverine or tidal conditions can be very difficult and is greatly dependent on the type of filter used, the availability of appropriate equipment, accessibility, and flow conditions. Granular filters can be partially or completely washed away by stream flow when being installed around piers. A geotextile must be able to remain relatively intact and withstand ripping or tearing and displacement during in-

stallation in order to provide stability to the overlying riprap layer. Many European countries have developed special equipment and installation procedures to counter most of these problems (CUR 1995). According to Pagán-Ortiz and Lagasse (1999), a significant investment has been made in Germany and the Netherlands in the development and testing of geosynthetic materials, and innovative installation techniques have been developed that could find application for bridge pier countermeasures in the United States. Heibaum (2000) describes the types of filter materials and systems used and the methods of placement under water, including the use of geotextile containers.

European Applications

Although riprap is used extensively as a pier scour countermeasure in the United States, it is not considered a permanent solution to scour around bridge piers. In contrast, riprap (armor stone in Europe), often in combination with either a geotextile or granular filter layer, is considered a permanent countermeasure for scour and stream instability in European countries (Pagán-Ortiz and Lagasse 1999; Bryson et al. 2000; Lagasse et al. 2001). Because of riprap's highly desirable characteristics of availability, economy, ease of installation, and flexibility, considerable effort has been devoted to techniques for determining the size, gradation, layer thickness, horizontal coverage, filter design, and construction methods for use in riverine and coastal applications. Heibaum (2000) describes several methods used in Europe to counter scour around various structures, including bridge piers, and provides a summary of the types of materials and systems used and the methods of placement (under water) under both mild and strong currents.

The difference between U.S. and European practice is not necessarily derived from the availability of better techniques for sizing riprap, but rather from the European practice of providing a higher standard of care and quality control in placing the stone, and providing an appropriate filter on sand bed channels. In addition, European practice includes inspection and monitoring to verify that riprap is performing properly. European hydraulic engineers have developed innovative techniques for placing an effective filter beneath the riprap in flowing or deep water including the use of large geotextile sand containers, geotextile mattresses filled with granular filter material, and fascine sinker mats. Fascine sinker mats are used for water depths generally greater than about 60 ft (18 m).

According to Bryson et al. (2000), most European engineers recommend the use of the *Manual on the Use of Rock in Hydraulic Engineering* (CUR 1995) as a reference for riprap and filter design. However, most of the applications in Europe are used to counteract erosion caused by wave

wash, tidal fluctuations, and storm surges, and the majority of the design guidelines are based on the use of riprap primarily as bed and bank protection, not as a pier scour countermeasure.

Partially Grouted Riprap

Current practice in the United States discourages the use of grouted riprap, primarily because the voids within the riprap are, in most cases, nearly completely filled with grout, 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 (for example, Brown and Clyde 1989). 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. This rigidity and reduced permeability may also suggest why a survey of U.S. field engineers conducted by Parker et al. (1998) on the feasibility, effectiveness, constructability, durability, maintainability, and cost of various types of countermeasures ranked grouted riprap fairly low.

Partially grouted riprap provides a more suitable alternative to total grouting because it alleviates the concerns and problems associated with completely filling the surface 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 unavailable.

In the United Kingdom, grouting is used primarily at the edges of revetments and at transitions with hydraulic structures (Escarameia 1998). There are two common types of grout material used in Europe: bituminous and cementitious. Bituminous grout consists of a chemically inert and viscous mixture of hydrocarbons and provides considerably more flexibility to the revetment compared to cement. Bituminous grout is the most commonly used material in the Netherlands because it reduces the stone sizes required. Cementitious grout is commonly used with "hand-pitched stone" and, in contrast to bituminous grout, confers rigidity and impermeability to a revetment.

Design manuals by CUR (1995) and Escarameia (1998) provide guidance for grouting stone revetments with both bituminous and cementitious grouts; bitumen is the most commonly used material with riprap. BAW in Germany has developed guidelines for the testing of bitumen-bonded materials used in the grouting of riprap revetments (BAW 1991—see Section 2.1.5 of this report). Wave tank experiments at BAW in Germany, experience on German inland waterways,

and development of design guidance for partial bitumen- and cement-grouted riprap in the United Kingdom provide a wealth of information on the design and installation of partially grouted riprap.

Various degrees of grouting are possible, but the most effective solutions are produced when the bituminous mortar envelops the loose stone and leaves relatively large voids between the stones. The degrees of bituminous grouting follow:

- Surface grouting (grout does not penetrate the whole revetment thickness and fills about one-third of the voids)
- Various forms of pattern grouting (where only part of the surface area of the revetment is filled, between 50% to 80% of voids)
- Full grouting (an impermeable type of revetment)

The two types of pattern grouting procedures, line-by-line and spot-by-spot, produce conglomerate-like elements in the riprap. With the proper grout, partial grouting can be done under water. Grout can be placed by hand only in water less than 1 meter deep. Special devices are required for placement in deeper water. Various European countries have developed special grout mixes and construction methods for underwater installation of partially grouted riprap (Lagasse 1999; BAW 1990, 1993a, 1993b—see Reference Document [www.trb.org/TRBNet/ProjectDisplay.asp?ProjectID=702]).

Partial pattern grouting is obtained when the grout is placed on the riprap leaving significant voids in the riprap matrix and considerable open space on the surface. No grout should penetrate deep enough to come in 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 (2000) indicates that grouting has proven its long-term stability and ability to keep costs low; for example,

laboratory tests at Braunschweig University in Germany proved that partially grouted riprap is stable up to a flow velocity of 26 ft/s (8 m/s). Also, because 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.

In almost all cases, a geotextile filter is recommended or required in conjunction with partially grouted riprap because of the potential for winnowing of underlying bed material. BAW (1993a), CUR (1995), and Escarameia (1998) all provide guidelines on the design and installation of filters used with partially grouted riprap.

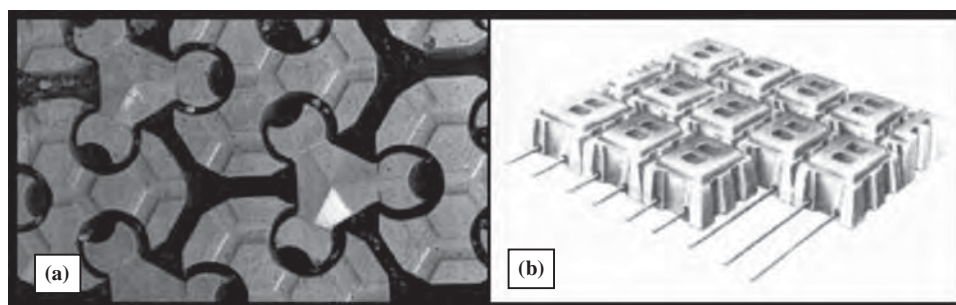
2.1.4 Alternatives to Riprap

In most cases where pier scour countermeasures are required, riprap is often the countermeasure of choice. However, in some cases, riprap may not be an option for a number of reasons. In some areas, riprap may be unavailable due to a lack of supply of durable stone. Where it is available the stone may not be in the size ranges required to provide the necessary protection against scour, or the stone may be prohibitively expensive to use. In some areas, environmental or aesthetic restrictions may preclude its use.

The following review of alternatives to riprap includes articulating concrete block systems, concrete armor units, gabions, grout-filled bags and mattresses, and sand-filled geotextile containers.

Articulating Concrete Block Systems

Articulating concrete block (ACB) systems provide a flexible alternative to riprap and rigid revetments. These systems consist of preformed units that 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 (Figure 2.6). Data sheets for a number of the more common proprietary ACB revetment systems can be found in Escarameia (1998). Parker et al. (1998) provides a brief



Source: (a) American Excelsior Company, (b) Armortec

Figure 2.6. Examples of interlocking block (a) and cable-tied block (b) systems.

review of the limited studies conducted on the use of ACBs for pier scour protection.

There is limited experience with the use of ACB systems as a scour countermeasure 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 ACB systems along banklines and in channels are well documented (e.g., Ayres Associates 2001), but there are few published guidelines on the installation of these systems around bridge piers.

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. Although no additional hydraulic stability is attributed to the presence of cables, they can prevent individual blocks from being plucked out of the matrix when failure is imminent. 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 was important to the overall effectiveness and stability of the ACB.

In some cases, a geotextile, usually composed of a non-woven, needle-punched synthetic, is bonded to the underside of the ACB mat. The gap that separates the blocks in a geotextile-bonded system produces a more flexible mat because of the lack of inter-block friction that can result with interlocking block geometries. By bonding the blocks to the geotextile, winnowing is eliminated; the blocks are not separated from the underlying geotextile by shear forces, uplift pressures, or subsidence; and the blocks and geotextile together can fold down with the bed during the passage of bed forms. Although applied as a bed and bank revetment, this type of system has not been tested as a pier scour countermeasure and it is not known how this type of system will respond to the passage of deep troughs.

Specifications and design guidelines for installation and anchoring of ACBs as bed and bank revetment are documented in HEC-11 (Brown and Clyde 1989) and guidelines on the selection and design of filter material can be found in HEC-11 and Holtz et al. (1995). HEC-11 directs the designer to the manufacturer's literature for the selection of appropriate block sizes for a given hydraulic condition. Because ACBs vary in shape and performance from one proprietary system to the next, each system will have unique performance properties. Manufacturers of ACBs must test their products and develop design criteria based on the results from these tests. HEC-23 (Lagasse et al. 2001) provides equations for determining the factor of safety in determining block sizes and computing the potential effects of pro-

jecting blocks. In Europe, ACBs are designed using guidelines similar to those provided in HEC-23 and are used as bed and bank revetment primarily on relatively straight channels under normal flow conditions and low turbulence. Escarameia (1998) recommends that modeling should be conducted prior to installation for applications where turbulent or other extreme hydraulic conditions are expected. Parker et al. (1998) provide some design recommendations for the use of cable-tied blocks as pier scour countermeasures.

The design procedure for ACBs for revetment and bed armor provided in HEC-23 (Lagasse et al. 2001) quantifies the hydraulic stability of revetment block systems using a "discrete particle" approach (like many riprap sizing methods). The design approach is similar to that introduced by Stevens (1968) to derive the "factor of safety" method of riprap design as described in Hydraulic Design Series (HDS) 6 (Richardson et al. 2001). The force balance has been recomputed considering the properties of concrete blocks, and the Shields relationship utilized in the HDS 6 approach to compute the critical shear stress has been replaced with actual test results (Richardson et al. 2001). The design procedure incorporates results from hydraulic tests into a method that is based on fundamental principles of open channel flow and rigid body mechanics. The ratio of resisting to overturning moments (the "force balance" approach) is analyzed based on the size and weight characteristics of each class and type of block system and includes performance data from full-scale laboratory testing. This ratio is then used to determine the factor of safety against the initiation of uplift and rotation about the most critical axis of the block.

Also incorporated into the design procedure in HEC-23 (Lagasse et al. 2001) are considerations that can account for the additional forces generated on a block that protrudes above the surrounding matrix because of subgrade irregularities or imprecise placement. Because finite movement constitutes failure, the analysis methodology provided in HEC-23 purposely contains no explicit attempt to account for resistive forces due to cables or rods. Similarly, the additional stability that may arise from vegetative root anchorage or mechanical anchoring devices, while recognized as significant, is ignored in the analysis procedures for the sake of conservatism in selection and design.

According to HEC-23, the designer must determine what factor of safety should be used for a particular design. Risks associated with a failure of the project, the uncertainty of hydraulic values used in the design, and uncertainties associated with installation practices are some of the variables that should affect the selection of the factor of safety used for final design. Typically, a minimum factor of safety of 1.5 is used for revetment design when the project hydraulic conditions

are well known and variations in the installation can be accounted for. Higher factors of safety are typically used for protection at bridge piers, abutments, and channel bends because of the complexity in computing hydraulic stresses at these locations.

Although the hydraulic stability of ACB systems at bridge piers can be assessed using the factor of safety method, uncertainties in the hydraulic conditions around bridge piers warrant increasing the factor of safety in lieu of a more rigorous hydraulic analysis. Experience and judgment are required when quantifying the factor of safety to be used for scour protection at an obstruction in the flow. In addition, when both contraction scour and pier scour are expected, design considerations for an ACB mat placed around a pier become more complex. The guidelines in HEC-23 (Lagasse et al. 2001) reflect recommendations developed by McCorquodale et al. (1993, 1998), the Minnesota Department of Transportation (Mn/DOT), and the Maine Department of Transportation (MDOT) for application of ACBs as a countermeasure for pier scour. Specific studies on the use of cable-tied blocks as a means of protection for bridge piers can be found in Jones et al. (1995), Bertoldi et al. (1996), Stein et al. (1998), McCorquodale et al. (1993, 1998), and Parker et al. (1998). Much of the guidance in these reports has been superseded by the findings of this study (see Appendix E).

As part of NCHRP Project 24-07, Parker et al. (1998) evaluated flow altering and armoring alternatives to standard riprap installations as pier scour countermeasures. Based on laboratory testing they conclude that a mattress of cable-tied blocks underlain by a geotextile tied to the pier provides “excellent protection” and present suggestions for the design of cable-tied blocks.

An observed key point of failure for ACB systems at bridge piers occurs at the seal where the mat meets the bridge pier (McCorquodale et al. 1993, 1998; Stein et al. 1998). During the flume studies by McCorquodale et al. (1993, 1998) and Stein et al. (1998), the mat was sealed to the pier to prevent scouring of the sediments adjacent to the pier. This procedure was highly successful in the laboratory; however, in the field the transfer of moments from the mat to the pier may affect the structural stability of the pier. When the mat is attached to the pier, the increased loading on the pier must be considered.

Mn/DOT has installed a cable-tied mat for a pier at TH 32 over the Clearwater River at Red Lake Falls. In addition to grout, Mn/DOT recommends the use of tension anchors around the pier seal. Anchors can provide additional support for the mat and grout at the pier seal will reduce scouring at the mat/pier interface. Mn/DOT provided the following specifications:

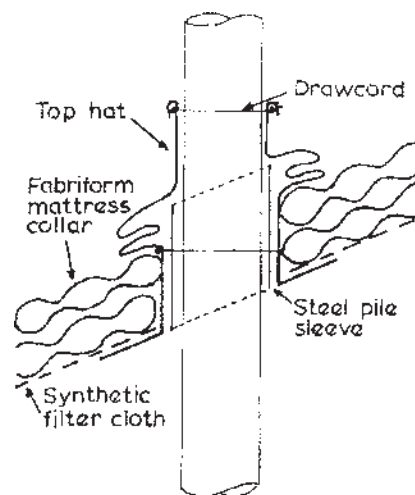
- **Anchors:** Mn/DOT recommends the use duckbill anchors, 0.9 to 1.2 m (3 to 4 ft) deep, at corners and about every 2.4 m

(8 ft) around pier footings. McCorquodale et al. (1998) recommend an anchor spacing of 4 ft (1.2 m) along the edges.

- **Pier Seal:** Research conducted by the FHWA indicates that the space between the pier and the cable-tied concrete blocks must be filled or scour may occur under the blocks. To provide this seal, Mn/DOT proposed that concrete be placed around the pier. Mn/DOT suggested that the riverbed could be excavated around the piers to the top of the footing. The mat could be put directly on top of the footing and next to the pier with concrete placed underneath, on top of, or both, to provide a seal between mat and pier.

A 1998 review of European practice for bridge scour countermeasures (Lagasse 1999) identifies two approaches for solving the problem of providing a seal between the bridge pier and ACB or grout-filled mattress systems. The review references a proprietary system in Germany for installing a collar and tying the geotextile filter underlying a mat (or mattress) to the bridge pier using a flexible tie (Figure 2.7). This approach appears feasible for circular piers. Considering possible settlement of the mat relative to the structure (pile), a steel sleeve and a “top hat” of filter fabric were proposed with a collar of Fabriform® laid on top of the mat and tied to the sleeve as indicated in Figure 2.7. As relative settlement occurs, the sleeve is expected to slide down the pile and the top hat to expand, bellows fashion, with a collar for protection. This approach may be limited in areas where the top hat could be damaged by abrasion.

In the Netherlands, the recommended approach to the problem of sealing the joint between a mat and a bridge pier is to place granular filter material to a depth of about



Fabriform mattress arrangement at a pile
Source: Lagasse (1999)

Figure 2.7. Flexible collar arrangement at a pile to seal the joint with a mattress.

3 ft (1 m) below the streambed for about 16 ft (5 m) around the pier. The geotextile filter and ACB mat placed on the streambed overlap this granular filter layer and the remaining gap between the mat and the pier is filled with riprap (Figure 2.8).

Concrete Armor Units

The group of concrete armor units, also known as artificial riprap, consists of individual pre-cast concrete units with complex shapes that are placed individually or in interconnected groups. These units were originally developed for shore protection to resist wave action during extreme storms. All are designed to give a maximum amount of interlocking using a minimum amount of material. These devices are used where natural riprap is unavailable or is more costly to obtain than fabrication of the artificial riprap units. Parker et al. (1998) provide a review of studies conducted on the use of concrete armor units as pier scour countermeasures.

Various designs for size and shape of concrete armor units are available (Figure 2.9). Because concrete armor units are similar to riprap, they can be susceptible to the same failure mechanisms as riprap. However, the use of a filter layer or geotextile in conjunction with these types of devices is often required, especially in coastal applications, and a geotextile or filter may be critical to the stability of these devices when used as pier scour protection.

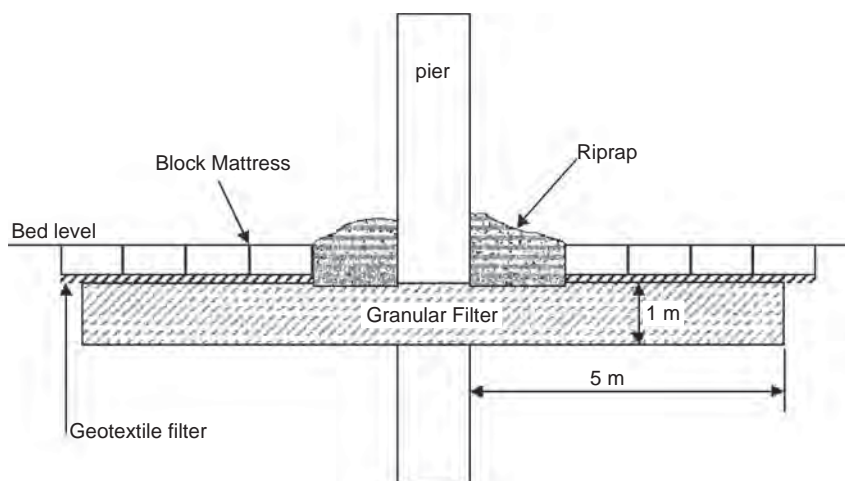
The primary advantage of armor units is that they usually have greater stability compared to riprap particles of equivalent weight. This greater stability is due to the interlocking characteristics of their complex shapes. The increased stability allows their placement on steeper slopes or the use of

lighter weight units for equivalent flow conditions as compared to riprap. This characteristic is significant when riprap of a required size is not available.

The design of armor units in open channels is based on the selection of appropriate sizes and placement patterns to be stable in flowing water. The armor units should be able to withstand the flow velocities without being displaced. Hydraulic testing is used to measure the hydraulic conditions at which the armor units begin to move or “fail,” and dimensional analysis allows extrapolation of the results to other hydraulic conditions. Although a standard approach to the stability analysis has not been established, design criteria have been developed for various armor units using the following dimensionless parameters:

- Isbash stability number (Parola 1993; Ruff and Fotherby 1995; Bertoldi et al. 1996)
- Shields parameter (Bertoldi et al. 1996)
- Froude number (Brown and Clyde 1989)

The Isbash stability number and Shields parameter are indicative of the interlocking characteristics of the armor units. Froude number scaling is based on similitude of stabilizing and destabilizing forces. Quantification of these parameters requires hydraulic testing and, typically, regression analysis of the data. Prior research and hydraulic testing have provided guidance on the selection of the Isbash stability number and Shields parameter for riprap and river sediment particles, but stability values are not available for all concrete armor units. Therefore, manufacturers of concrete armor units have a responsibility to test their products and to develop design criteria based on the results of these tests.



Source: Lagasse (1999)

Figure 2.8. Use of granular filter and riprap to seal the joint between a bridge pier and ACB mattress.

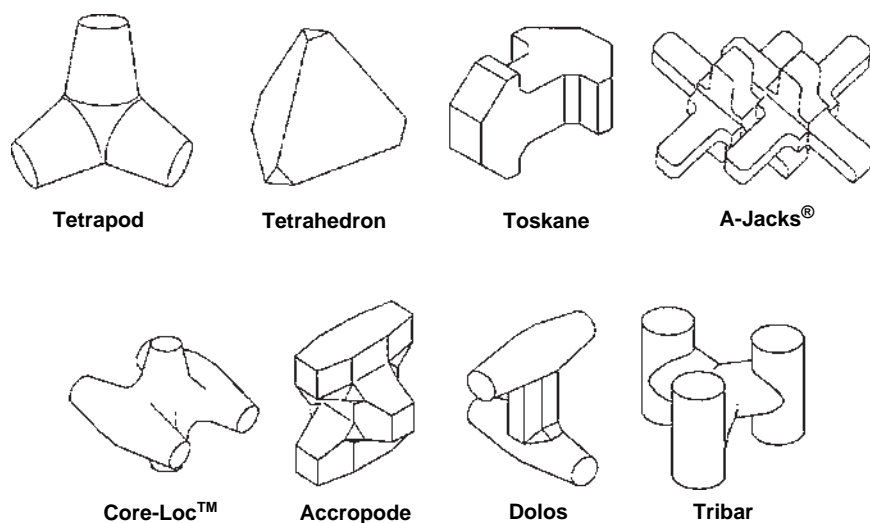


Figure 2.9. Concrete armor units.

Because armor units vary in shape and performance from one proprietary system to the next, each system will have unique performance properties.

Installation guidelines for concrete armor units in stream-bank revetment and channel armor applications should consider subgrade preparation, edge treatment (toe down and flank) details, armor layer thickness, and filter requirements. Subgrade preparation and edge treatment for armor units are similar to that required for riprap, and general guidelines are documented in HEC-11 (Brown and Clyde 1989). Considerations for armor layer thickness and filter requirements are product specific and should be provided by the armor unit manufacturer.

Concrete armor units have shown potential for mitigating the effects of local scour in the laboratory; however, only limited data are available on their performance in the field. Research efforts are currently being conducted to test the performance of concrete armor units as pier scour countermeasures in the field.

Design methods that incorporate velocity (a variable that can be directly measured) are commonly used to select local scour countermeasures. Normally an approach velocity is used in the design equation (generally a modified Isbash equation) with a correction factor for flow acceleration around the pier or abutment (Lagasse et al. 2001).

Although tetrahedrons are currently used for bank protection (Fotherby 1995), they have garnered very little interest with regard to pier scour protection in the United States. This may be primarily related to their lack of appendages and interlock (i.e., their simple, compact shape is similar to riprap and spheres). Dolosse also have not been seriously considered for use as pier scour protection because they have no inherent interlocking property to resist movement under steady state turbulent flow (Brebner 1978). Extensive testing

and research have been conducted on the Core-Loc™ system, which was developed by the U.S. Army Engineer Waterways Experiment Station, but the testing was limited exclusively to coastal applications. Accropode™ and tribar systems are used almost exclusively in coastal applications as well.

In contrast, tetrapods have been extensively studied and evaluated for use as pier scour protection (Fotherby 1992, 1993; Bertoldi et al. 1996; Jones et al. 1995; Bertoldi and Kilgore 1993). Fotherby (1992, 1993) and Stein et al. (1998) suggest that tetrapods offer little advantage compared to riprap in terms of stability. Layering and density had no appreciable effect on the stability of the tetrapods, although the stability increased with the size of the tetrapod pad. Work by Bertoldi et al. (1996) and Stein et al. (1998) indicates that riprap and tetrapods behaved comparably when both stability number and spherical stability number were compared and also suggests that fixing the perimeter and varying the number of tetrapod layers may have an effect on stability.

A specific design procedure for Toskanes has been developed for application at bridge piers and abutments and is described in HEC-23 (Lagasse et al. 2001) to illustrate a general design approach where the Toskanes are installed as individual, interlocking units. The design procedures for Toskanes are based on extensive research conducted at Colorado State University (Ruff and Fotherby 1995; Fotherby 1995; Burns et al. 1996; Fotherby and Ruff 1998a, 1998b). Based on hydraulic model studies conducted at CSU for the Pennsylvania Department of Transportation, Burns et al. (1996) presented procedures for the design of Toskane pads, provided criteria for sizing Toskanes, and suggested techniques for installation of Toskanes. No other concrete armor unit has been as extensively tested and evaluated for use as a pier scour countermeasure.

Another approach to using concrete armor units for pier scour protection has been investigated by the Armortec Company and involves the installation of banded modules of the A-Jacks® armor unit (Ayres Associates 1999; Thornton et al. 1999). Laboratory testing results and installation guidelines developed at CSU by Ayres Associates (1999) for the A-Jacks® system are also presented in HEC-23 (Lagasse et al. 2001) and illustrate the “modular” design approach in contrast with the “discrete particle” approach for Toskanes.

The discrete particle design approach illustrated by the Toskane design guidelines in HEC-23 concentrates on the size, shape, and weight of individual armor units, whether randomly placed or in stacked or interlocked configurations. In contrast, the basic construction element of A-Jacks® for pier scour applications is a “module” composed of a minimum of 14 individual A-Jacks® banded together in a densely interlocked cluster, described as a 5x4x5 module. The banded module thus forms the individual design element as illustrated in Figure 2.10.

It should also be noted that concrete armor units, depending on their size, may be very susceptible to vandalism. In addition, there may be maintenance and degradation issues associated with any cables used to tie groups of concrete armor units together.

Gabions

Gabion systems, which include box gabions, gabion mattresses, and sack gabions, are containers constructed of wire mesh or other material and filled with loose stones or other similar material (Figure 2.11). The stones used to fill the containers can be either angular rock or large cobbles. Unlike cobbles, angular rocks used to fill the gabions interlock naturally, which provides additional strength to the unit.

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 mold themselves to the bed or bank, thus restoring stability to the unit. In addition, the use of a wire mesh allows for the use of relatively small stones, which can yield the same amount of protection characteristic of much larger units in loose configurations.

Maccaferri, Inc. first developed the gabion in 1884 and has since compiled a considerable body of information on the general design and use of gabions in the field. However, much of the information is essentially anecdotal and few independent tests and quantitative design guidelines exist (Parker et al.

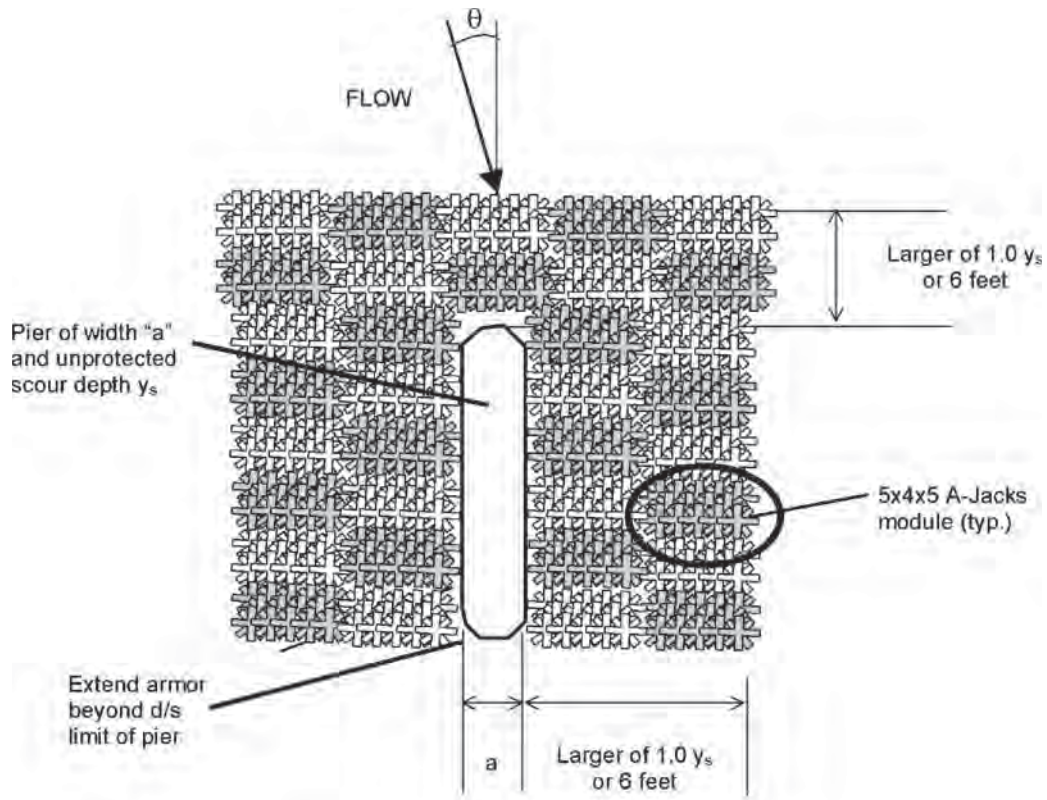
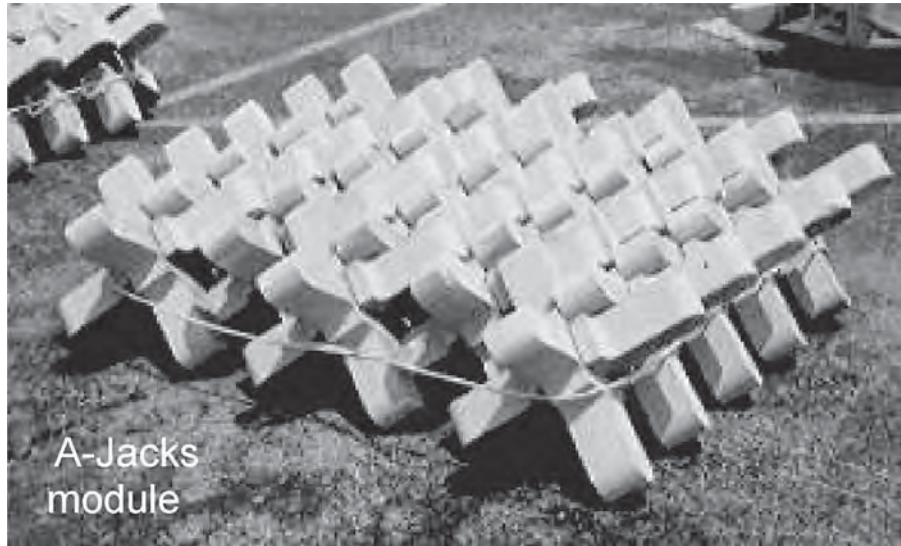
1998). Parker et al. (1998) and Lauchlan (1999) provide comprehensive reviews of the literature on gabions. Brown and Clyde (1989), CUR (1995), Maynard (1995), and Escarameia (1998) all provide guidelines on design and installation of gabions as bed and bank revetment. The model testing conducted by Simons et al. (1984) is probably the most substantial attempt to obtain quantitative design guidelines and criteria for gabion mattresses in the fluvial environment. However, their experiments do not provide a direct test of the performance of gabion mattresses as pier scour countermeasures.

Information on the design and use of gabions as a pier scour countermeasure is scarce. Parker et al. (1998) provide some design recommendations for the installation of gabions around bridge piers. Yoon and Kim (1999) conducted experiments under clear-water conditions to investigate the effectiveness of a sack gabion as a scour countermeasure at bridge piers and used the results to derive formulas for sizing the gabions.

The effectiveness and stability of gabions as pier scour countermeasures appear to vary. According to Parker et al. (1998), a report from New York State suggests that they have not performed well in the field there. Lauchlan (1999) indicates that gabion mattresses with an underlying geotextile filter performed poorly as a pier scour countermeasure at the Whakatane River Bridge on State Highway 30 in New Zealand. Yet in a survey conducted by Parker et al. (1998), gabions received a favorable review from most state engineers surveyed.

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 at the pier is provided. However, the passage of bed forms could cause the wire mesh to break under tension during deformation of the gabion and allow the fill stones to be removed from the basket. In addition, uplift forces or piezometric gradients below the geotextile may cause warping of the gabion mattress and cause it to pull away from the pier, thus inducing or enhancing scour around the pier face and further destabilizing the gabion unit. The gabion mattress may also 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 (Lauchlan 1999). Anchoring the gabion with long steel rods may partially or completely alleviate these problems.

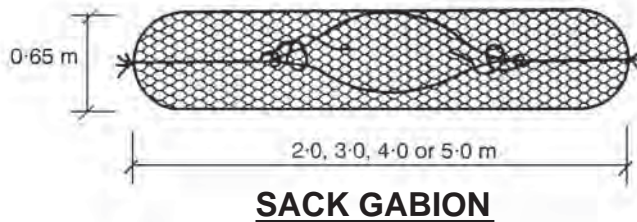
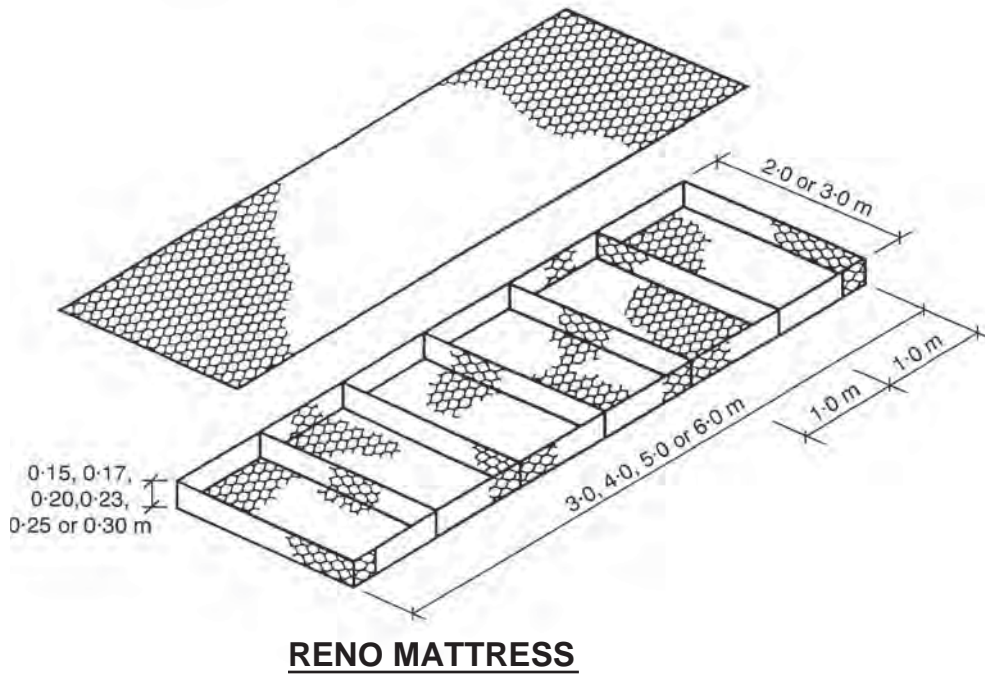
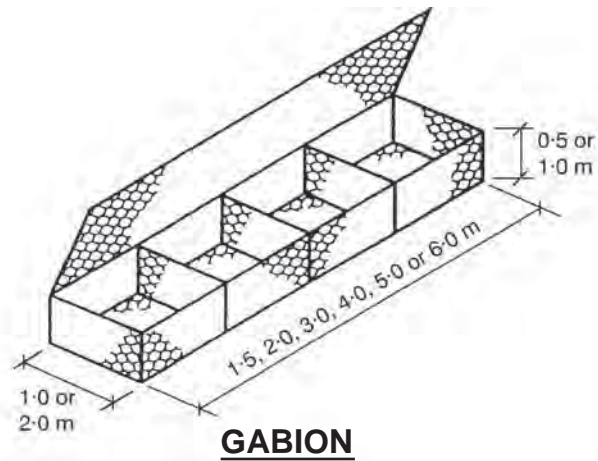
Finally, the maintenance requirements for gabions may be somewhat higher than for other forms of revetment because the wire mesh used to construct the gabion is susceptible to abrasion and corrosion, and because gabions are also very susceptible to vandalism. Based on field studies conducted for Caltrans, Racin and Hoover (2001) have developed standard plans and material specifications for mesh types and corrosion-resistant coatings for use in gabions. Parker et al. (1998) also provide general design recommendations for the use of gabions.



Note: For skew angle θ greater than 15 degrees, increase the above dimensions by $1/\cos\theta$.

Source: Ayres Associates (1999)

Figure 2.10. A-Jacks® modules for pier scour protection.



Source: modified from Hemphill and Bramley (1989)

Figure 2.11. Types of gabions and typical dimensions.

Grout-Filled Bags and Mattresses

Grout-filled bags (including sacked concrete) and mattresses are fabric shells that are filled with concrete. These countermeasures may be the simplest and most cost-effective alternatives to riprap. They are used in areas where the availability of riprap is limited or where it is expensive to use, where there are environmental restrictions that limit the use of riprap, where the size of the bridge opening and channel are small, or where equipment access is limited. Concrete also has the advantage of being a well-known and often-used construction material familiar to bridge engineers.

Parker et al. (1998) and Lauchlan (1999) provide a comprehensive review on the use of grout-filled bags and mattresses as pier scour countermeasures. The bulk of the literature on research pertaining to the use of grout-filled bags and mattresses as pier scour countermeasures is contained in Fotherby (1992, 1993), Bertoldi et al. (1996), Jones et al. (1995), and Stein et al. (1998). The researchers determined that properly installed grout-filled bags and mattresses reduce scour depth to a degree generally comparable with riprap.

Grout-Filled Mattresses. The grout-filled mattress is a single, continuous layer of strong synthetic fabric sewn into a series of compartments that are connected internally by ducts. The compartments are then filled with a concrete grout that, when set, forms a mat made up of a grid of connected blocks or pillows. While the individual blocks may articulate within the mattress and the mattress remains structurally sound, the general design approach is to consider the mattress as a rigid monolithic layer. In some cases, the mattress may be strengthened with cables installed similar to those used in articulating concrete blocks. Depending upon the proprietary system, filter points or weep holes allow for pressure relief through the mattress. Grout-filled mattress 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 revetment is quite specialized, comprehensive technical information on specific mattress types and configurations is available from a number of major manufacturers of this type of revetment.

In a survey of bridge engineers conducted by Parker et al. (1998), grout-filled mattresses were ranked poorly in terms of cost and maintenance but were ranked favorably with regard to debris susceptibility and environmental disruption. In contrast, grout-filled bags were ranked favorably because of their minimal need for expertise or equipment, their rapid installation, and their cost effectiveness. Problems with water quality, climatic conditions, aesthetics, and social acceptability were defined as drawbacks to the use of grout-filled bags.

The primary failure mechanisms for grout-filled mattresses consist of rolling, undermining, and scouring at gaps (Fotherby 1992). Rolling, the most severe form of failure, is related to uplift forces created by flow over the mattress. This flow allows the mattress at midsection to be “lifted up” slightly and then pushed loose by the force of the current or allows the edges of the mattress to be rolled back. Undercutting is a gradual process arising from local scour at the mattress edges and from the main horseshoe vortex. Scouring at the gaps between the mattress and the pier wall allows the horseshoe vortex to generate a scour hole beneath the front edge or side sections of the mattress.

The research to date on the use of grout-filled mattresses as a bridge scour countermeasure found that placement is extremely important for successful performance and effectiveness. Properly placed grout-filled mattresses extending 1.5 to 2 pier widths were found to provide significant protection to bridge piers. Fotherby (1992) recommends that grout-filled mattresses should be placed at bed level and suggests that toeing in the mattress may increase stability with regard to potential rolling failure and undercutting, especially under live-bed conditions. Bertoldi et al. (1996) recommend that anchors be used to protect the leading edge against uplift forces when the mattress is placed on the surface of a loose, erodible channel bed. Jones et al. (1995) and Stein et al. (1998) stress the importance of a tight seal around the pier-mattress interface to inhibit scour and undermining beneath the mattress. Lagasse et al. (2001) provide mattress selection and sizing criteria based on analysis of sliding stability.

Grout-Filled Bags. Grout/cement-filled bags have been used extensively as bank protection and are gaining in popularity as a countermeasure against scour at bridges. Historically they have been used to fill undermined areas around bridge piers and abutments. As scour awareness increases, grout-filled bags are being used to armor channels where scour is anticipated or where scour is detected, such as around bridge piers. They are relatively easy to install and come in a wide range of sizes, depending on the application. Engineering judgment is often used to select a bag size that will not be removed by the flow. As in the United States, grout-filled bags in the United Kingdom and Europe are used primarily as an emergency or temporary scour countermeasure.

Failure of grout-filled bags can occur from undersized bags, local scour around the bags, a shift in the grout bags, and undercutting of the filter fabric when used with the grout bags. Undersized bags can be swept away by currents. The bags may shift or slide either by winnowing and scour around the bags or the passage of bed forms. If the bags protrude into the flow, they create their own local scour pattern, which contributes to the undermining of the filter fabric where

used. Undermining of the underlying geotextile, where used, can occur as a result of local scour induced by grout-filled bags protruding into the flow, by edge erosion, or by the passage of bed forms. As bags slide off the underlying fabric, more fabric is exposed, which contributes to additional undermining and instability.

Research also indicates that the effectiveness of grout-filled bags as a pier scour countermeasure is dependent upon the size, placement, use of a filter fabric or geotextile, tightness of the seal to the pier face, and the lateral extent of the revetment apron. As with riprap and grout-filled mattresses, current practice indicates that the grout-filled bag protection should extend 1.5 to 2 pier widths out from the pier. Bags placed along the side of the pier aligned flush with the front of the pier tend to be prone to failure; a staggered placement provides better protection and greater stability. The use of a geotextile or filter fabric, preferably toed in, is recommended. Studies show that the use of grout-filled bags without a geotextile or filter fabric results in settlement of the bags into the bed and formation of a scour hole beneath them at the front of the pier. A single layer of properly sized grout-filled bags with an appropriate lateral extent was found to be more effective than stacked bags.

Undersized grout-filled bags can be washed away and sound engineering judgment should be used in sizing bags. Because of problems of comparison using a unit diameter (d_{50}) to determine particle stability, studies conducted by Bertoldi et al. (1996), Jones et al. (1995), and Stein et al. (1998) used the height of the grout-filled bags for d_{50} in applying the Shields and Isbash criteria. Fotherby (1992) provides the following limiting criteria for sizing grout-filled bags that are not rigidly connected:

- Shorter height produces less scour when the bag is exposed to the flow field.
- Shorter height in a rectangular bag is better able to resist overturning.
- Under incipient motion tests, length contributes to failure when the bags are aligned perpendicular to flow; the longer bags fail first.
- Longer bags are less able to adjust to bed elevation changes and tend to span scour holes rather than conform to the channel bed
- Wider bags reduce labor and installation costs when covering a large area.
- Increased width helps reduce overturning.
- Wider bags do not adjust as well to bed elevation changes and lose their ability to conform to changes in the channel bed.

Based on a scour evaluation program developed by the Maryland State Highway Administration (MDSHA), Thorn-

ton (1998) documents the few problems and multiple benefits associated with the use of grout-filled bags at small, relatively inaccessible bridges. Based on field experiences with the use of grout-filled bags for scour countermeasures, Thornton (1998) provides tips on their installation. These tips and additional recommendations on the specifications, design, and installation of grout-filled bags, based on information provided by MDSHA, are included in HEC-23 (Lagasse et al. 2001). Parker et al. (1998) suggest that the stability and performance of grout-filled bags can potentially be improved by imbricating (i.e., shingling) the bags or increasing the effective weight of the concrete by spiking it with high-density material. Rigidly connecting the bags by cable or rods or sewing the bags together should be avoided because these techniques significantly reduce the flexibility of the system.

Geotextile Containers

In Europe, a significant investment has been made in the development and testing of geosynthetic materials, and innovative installation techniques have been developed that could find application for bridge pier and abutment countermeasures in the United States. Highly specialized laboratory equipment is available for testing a wide range of geotextile characteristics. For example, BAW published “Code of Practice – Use of Geotextile Filters on Waterways” (MAG) (BAW 1993a) and has issued a complete report on the testing of geotextiles (RPG), including (1) impact tests (to determine punching resistance, e.g., when large stone is dropped on the geotextile); (2) abrasion tests; (3) permeability, clay clogging, and sand clogging tests; and (4) tests of material characteristics such as elongation and strength (BAW 1994). Through this testing program, geotextile materials have been developed that permit innovative approaches to filter placement for riprap and other countermeasures (Lagasse 1999).

Because of the extensive testing program in Europe, geotextile filters can be manufactured with consistent quality and in accordance with the requirements of a specific application. The wide choice in synthetics also allows the use of an inert material that will not interact with the environment. While the filtration capacity of woven geotextiles is restricted to narrowly graded grain size distributions, a non-woven fabric can be designed for nearly any given grain size distribution of the subsoil. It is also possible to combine a woven and a non-woven geotextile to combine, for example, good filtration capacity with high strength. The main function of geosynthetics in scour countermeasures is that of a filter, but they also can be used as containment or as reinforcement (Heibaum 2000, 2001, 2004).

The development in Germany of geotextile containers, or geocontainers, as a filter or as a stand-alone countermeasure is one of the concepts that has benefited from the geosynthetic material testing program. Geotextile containers are large bags

made of mechanically bonded non-woven fabrics up to 44 ft³ (1.25 m³) in volume partially filled with sand and gravel filter material (Figure 2.12). They have been used to provide a filter layer for riprap installation at a number of large projects in Germany (Heibaum 2000). The containers are sewn on three sides at a factory and filled on site to approximately 80% of capacity with sand/gravel filter material using a hopper system. The final seam is sewn on site. The containers are placed in layers using a side-dump pontoon or bottom-dump split barge. The flexibility of the fabric and partial filling allow the containers to conform to irregularities in the channel bed at the installation site, especially where very large scour holes have developed (Lagasse 1999; Heibaum 2000). Riprap or partially grouted riprap can then be placed over the layer of geotextile containers as an armor layer.

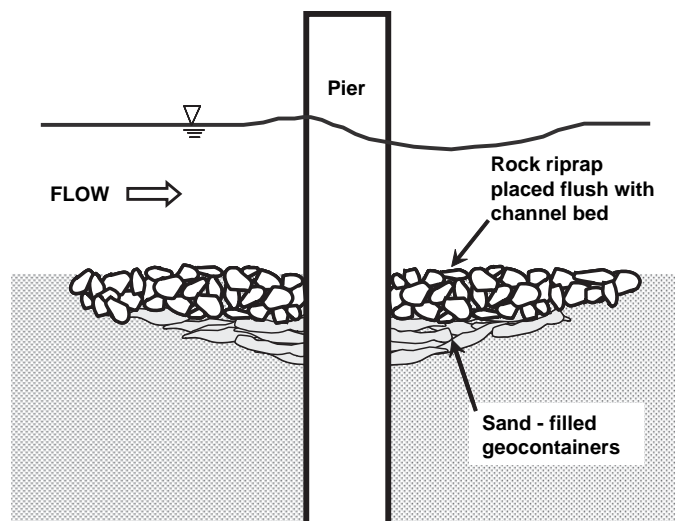
Because geotextile containers are designed as filters for a specific subsoil, it is essential that there are no gaps between the individual containers. Usually at least two layers of containers are required. Figure 2.13 shows a schematic installation of two layers of geotextile containers and riprap as a pier scour countermeasure. Thus, geocontainers are multi-purpose elements. They can be manufactured to site-specific size, shape, filtration capacity, and strength and, according to the demands of a specific site, only a few containers may be necessary, or many may be required (Heibaum 2000).

Heibaum (2000) provides general guidelines on the design and installation of geotextile containers. Pilarczyk (2000) presents the state of the practice in the design and installation of geocontainers including general design considerations, analysis of dumping process, stresses associated with opening the barge and during free fall and impact on the floor, the final shape of the geocontainer, deformations due to lateral loading and wave attack after placement, scaling rules for model tests, and calculation methods that can be used as design rules.



Source: Heibaum (2000)

Figure 2.12. Batch plant for filling numerous geotextile containers on site.



Source: modified from Heibaum (2000)

Figure 2.13. Schematic of pier scour repair using geocontainers as filter and fill with riprap as a cover layer.

2.1.5 Federal Waterways Engineering and Research Institute (BAW) Guidelines and Codes

Task 1 included preparing an English translation of three BAW documents formerly available only in German:

- “Code of Practice – Use of Cement Bonded and Bituminous Materials for Grouting of Armor Stones on Waterways” (MAV) (BAW 1990)
- “Guidelines for Testing of Cement and Bitumen Bonded Materials for the Grouting of Armor Stones on Waterways” (RPV) (BAW 1991)
- “Code of Practice – Use of Standard Construction Methods for Bank and Bottom Protection on Waterways” (MAR) (BAW 1993b)

The initial translation of these documents was accomplished by Dr. Kornel Kerenyi of GKY and Associates. Final translation was completed by Dr. Michael Heibaum of BAW in collaboration with Dr. P.F. Lagasse, the project principal investigator. The translations of these documents are included in the Reference Document (available on the TRB website: <http://www.trb.org/TRBNet/ProjectDisplay.asp?ProjectID=702>).

A fourth BAW document, “Code of Practice – Use of Geotextile Filters on Waterways” (MAG) (BAW 1993a), was obtained in English during the TRB/FHWA 1998 scanning review (Lagasse 1999).

These documents form the basis for many of the guidelines and specifications in Chapter 3 for the application of partially

grouted riprap and specialized geotextiles as pier scour countermeasures.

2.2 Performance Evaluation at Existing Sites

2.2.1 Introduction

During NCHRP Project 24-07, the University of Minnesota research team conducted a survey of field sites in 18 states representing different physiographic regions of the contiguous United States. The survey during 1996 was designed to develop first-hand knowledge of installation and inspection requirements for bridge scour countermeasures. Whenever possible the team identified the actual mode(s) of failure for existing installations. Of paramount importance was the identification of the controlling hydraulic, geomorphic, geotechnical, aesthetic, and environmental parameters that can affect constructability, reliability, maintainability, and cost. Key findings from visits to 88 field sites are discussed in Section 2.2.2.

An additional 15 project sites were visited and evaluated by the NCHRP Project 24-07(2) research team in 2001. Of these 15 sites, 9 specifically involved scour at piers; the remainder

consisted of abutment scour protection, bed or bank revetment, or other scour prevention applications where the use of specific materials or placement equipment of interest to the project team was investigated and/or demonstrated. Table 2.3 provides a brief summary of the field sites evaluated during NCHRP Project 24-07(2), organized by countermeasure type.

A discussion of the NCHRP Project 24-07(2) site visits in the United States—which included installations with riprap, articulating concrete blocks, and grout-filled mattresses as pier scour countermeasures as well as gabions and gabion mattresses installed for abutment scour protection—is provided in Section 2.2.3. The findings from the NCHRP Project 24-07(2) site visits in Germany to investigate partially grouted riprap installations and geotextile sand containers are also presented in Section 2.2.3. The NCHRP Project 24-07(2) interim report included a discussion of concrete armor units installed as a pier scour countermeasure, but no further investigations were made.

2.2.2 Key Findings: Phase 1 Site Visits

The Phase 1 site visit findings are reported in Chapter 7 of Parker et al. (1998) and are summarized briefly here. Two pri-

Table 2.3. Summary of Phase 2 scour countermeasure sites evaluated.

Countermeasure Type/ Application	Location/Structure I.D.	Comment
Riprap		
Multiple piers	California Wash, Nevada Bridge B-839S, Interstate 15 near Moapa, Nevada	Placed dry
Multiple piers	Piute Wash, Nevada Bridge B-420, U.S. Hwy 95 near Cal-Nev-Ari, Nevada	Placed dry
Single pier	Colorado River, Colorado Bridge G-04-BA, Interstate 70 near De Beque, Colorado	Placed under water
Partially Grouted Riprap		
Bed and bank revetment	Dortmund-Ems Canal, Germany Canal lining	Placed under water
Harbor bankline revetment	Wilhelmshaven Harbor, Germany Shore protection	Placed dry
Roof protection for highway tunnel beneath river	Elbe River, Hamburg, Germany Tunnel protection on channel bed	Placed under water
Scour protection for surge gate sills	River Ems Storm Surge Barrier, Germany Scour Protection	Placed under water
Articulating Concrete Blocks		
Piers and bed	Guadalupe River, California Bridge 37-0176, I-880 near Santa Clara, California	Placed dry
Grout-Filled Bags		
None		
Grout-Filled Mattresses		
Piers, bed, and abutments	Gila River, Yuma County, Arizona Three Bridges: Avenue 20E, 45E, 64E	Placed dry
Geotextile Sand Containers		
Materials and equipment demonstration	Colcrete—von Essen equipment yard, Germany Filling, lifting, and dropping 1.0 m ³ non-woven geotextile sand-filled containers	
Gabions and Gabion Mattresses		
Abutment scour protection	Guadalupe River, California Bridge 37-0176, I-880 near Santa Clara, California	
Concrete Armor Units		
Piers	Bridge 45, Marshall County, Kentucky	Placed dry
Piers	Hillsborough County, Florida	Placed dry

mary methods of failure were noted for riprap aside from direct entrainment by the flow. These are failures caused by (1) instability of the river bed and (2) failure caused by an inadequate filter. Stream instability affects countermeasure performance by altering the hydrodynamic conditions the countermeasure experiences. Other types of instability can occur when a bridge opening either significantly increases or decreases the conveyance capacity of the river. Typically, bridge openings are designed to not restrict flow past the bridge under flood conditions. During typical river discharges however, this design practice may lead to sediment deposition affecting countermeasure performance by locally altering flow patterns.

Adequate filtering should be placed under countermeasures to prevent subsidence-related failures of the countermeasures. Riprap in particular can sink well below the bed surface. This subsidence only becomes a serious problem if riprap sinks to a level below which it provides adequate protection; however, it becomes a maintenance problem as soon as riprap sinks to a level preventing routine detection during inspection. Winnowing of fines for other countermeasures can lead to voids underneath the countermeasure and general undercutting of the countermeasure if the countermeasure is not flexible.

A brief summary of the issues relevant to designing and installing pier scour countermeasures follows (Parker et al. 1998):

- Two primary methods of failure were noted for properly sized riprap
 - Instability of the river bed
 - Failure caused by an inadequate filter
- Countermeasure failure due to stream instability was consistently reported by the host engineers in most states. Many designers and nearly all maintenance personnel simply do not have the tools to effectively address stream stability issues.
- When dumped riprap is placed, caution must be exercised to ensure that segregation of the riprap does not occur and areal coverage is sufficient.
- The effect of localized drainage on countermeasure performance must be considered. Roadway ditches often discharge at 90° to the river channel and can subtly undercut scour protection, rendering the countermeasure less effective when a large flood arrives. Mitigation requires that drainage and bridge engineers work together to ensure that designs integrate well and are mutually effective.
- Geotextile must be placed so that no gaps are present, or can form, between geotextile and any structure it is protecting.
- Wire and cabling selection for gabions, gabion mattresses, and cable-tied blocks should be limited to non-corrosive materials. Field experience indicated that even well-specified galvanized, coated wire was subject to internal corrosion.

- Gabions should be inspected for basket tearing caused by riverborne debris following floods that exceed bankfull.
- Grout-filled bags were effective for small bridges, but undercutting was observed at the sides and end of bags when bags were too large to settle effectively.

2.2.3 Key Findings: Phase 2 Site Visits

The Phase 2 research team selected existing installations in the United States of several countermeasure types, countermeasure filters, and combinations of countermeasures listed in Task 1 and conducted a systematic analysis of the performance of the installations (Table 2.3). The analysis of performance included an in-depth documentation of the hydraulic and structural design of the installations as well as documentation of the construction, maintenance, and inspection considerations of each installation as well as an evaluation of the performance of each installation to date. The installations were geomorphically diverse and included underwater and dry placements.

Articulating Concrete Block Performance in the United States

The Phase 2 site visits in the United States are described in detail in a trip report supplied to the NCHRP Project 24-07(2) panel. One site on Guadalupe River at Interstate 880 near San Jose, California, with an ACB countermeasure was visited during Phase 1 (1996) and again during Phase 2 (2001). The comparison of performance over a 5-year period, which included several significant floods, is summarized here.

The I-880 bridge over the Guadalupe River is a two-span structure with a mid-channel wall pier. The structure was identified as scour critical in early 1992. The 100-year discharge at the site is 17,000 ft³/s (482 m³/s); however, the Standard Project Flood (SPF) in-channel discharge of 24,000 ft³/s (680 m³/s) was used for scour analysis and countermeasure design, with an associated maximum design velocity estimated at 13.4 ft/s (4.1 m/s). Unprotected scour depth was estimated at 15.5 ft (4.7 m) at the pier and 20.5 ft (6.25 m) at the abutment.

An ACB scour countermeasure was designed and constructed in mid-1992 under the direction of the U.S. Army Corps of Engineers, Sacramento District. The design utilized the factor of safety method as described in HEC-23 (Lagasse et al. 2001) with a target safety factor of 2.0. Overall project costs for improvements to the channel reach, excluding utility relocations, were \$2.9 million. Of that total, approximately half pertained to clearing, grading, materials, and installation costs associated with scour protection (ACB and gabion mattress). The cost for the materials and installation of 4,700 yd² (3930 m²) of ACB alone was \$324,000. Figure 2.14 provides a



Figure 2.14. Interstate 880 bridge over the Guadalupe River, San Jose, CA (looking downstream).



Figure 2.16. Close-up of interface between cabled ACB and gabion mattress.

photograph of the installation. Figure 2.15 provides a schematic diagram showing relevant dimensions of the installation, while Figure 2.16 shows a close-up of the interface between the ACB system and the gabion mattress.

Initial field inspection was performed by the NCHRP Project 24-07 research team in June 1996 (4 years after construction), and the site was re-examined by the NCHRP Project 24-07(2) research team in December 2001. The condition of the countermeasure in December 2001 appeared to be identical to that described in the NCHRP Project 24-07 report. The majority of the installation is in excellent condition and during the period between these examinations has withstood at least three events of approximately 3,000 ft³/s (85 m³/s) in magnitude with no further deterioration. Gaging station data are available from U.S. Geological Survey (USGS) gage near San Jose, California. A record from 1992 is provided in Figure 2.17.

Minor areas of local subsidence through the reach were noted in the NCHRP Project 24-07 report; in these areas, the ACB mat responded as intended and remained in intimate contact with the subgrade as revealed by the December 2001

examination. The interface between the ACB system and the pier was grouted adjacent to the edge of the concrete pile cap (Figure 2.18).

Downstream of the bridge, two areas of subgrade subsidence that were bridged by the ACB, causing local voids beneath the revetment, were noted in the inspection reports by both research teams. These areas are located at the downstream edge of the ACB mat at a plunging transition back into the natural channel bed. At this point, a continuous discharge of effluent enters the river from an outlet structure on the left bank. The blocks terminate in a plunging transition at the downstream edge of the installation and are toed down into the bed of the natural channel at that point.

These void areas have apparently never been repaired since they were first observed and in December 2001 appeared to be qualitatively identical to the conditions described in June 1996. In addition, the gabion mattresses on the overbanks and abutments were intact and in good condition. The wire of the baskets did not show evidence of corrosion, abrasion, or distortion from vegetative growth that has begun to in-

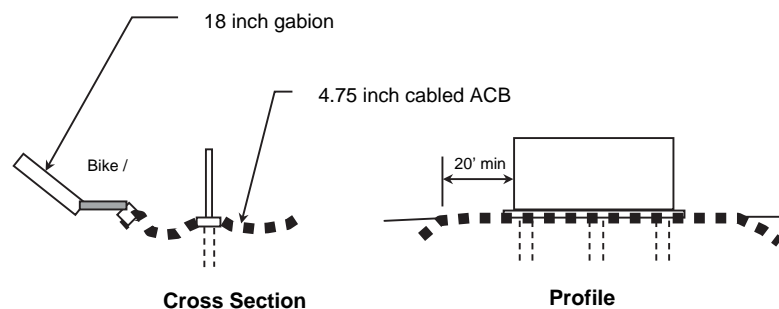


Figure 2.15. Guadalupe River pier/abutment scour countermeasure schematic diagram.

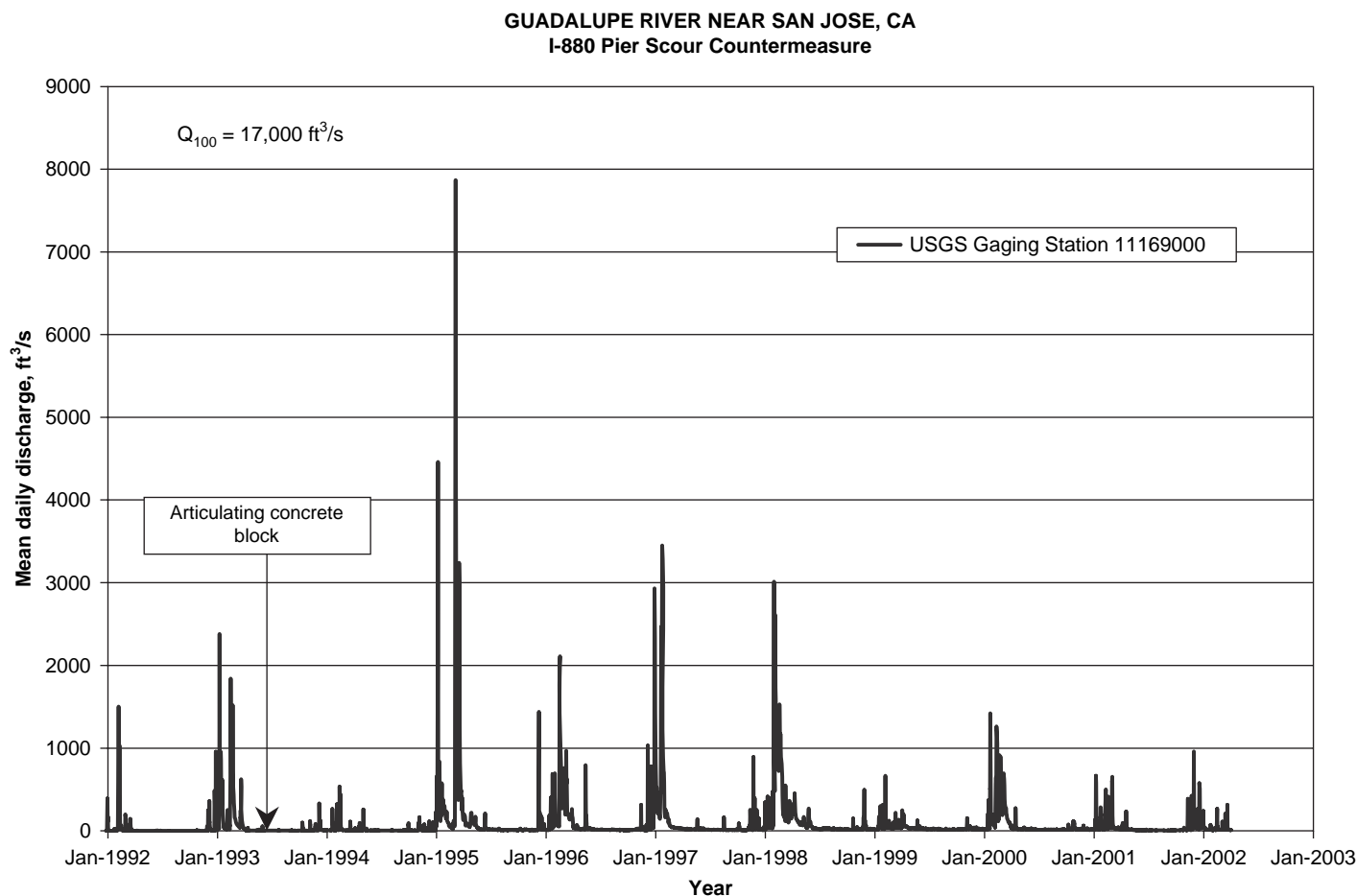


Figure 2.17. Mean daily hydrograph of Guadalupe River near San Jose, CA.

trude into the edges of the construction works both upstream and downstream of the bridge.

Site Visits in Germany

As discussed in Section 1.2.3, two research team members visited installations in Germany to evaluate the potential for application of geotextile containers and partially grouted riprap as a pier scour countermeasure and the performance of partially grouted riprap in a high-velocity, high-turbulence environment. The visit provided an opportunity to evaluate field performance of these countermeasures and the potential for adapting this technology for application in the United States.

The site visit was coordinated by Dr. Heibaum of the BAW laboratory and included evaluation of specialized equipment developed by contractors for placing grout for various specifications of extent of coverage and penetration into the riprap matrix, above or under water. Mr. Trentmann of Gewatech, who is an industry expert in partial grouting techniques, assisted with the site visit. The visit was conducted from September 22–29, 2001, and involved the principal investigator (PI) and Co-PI from the Phase 2 research team and an FHWA representative.

Partially Grouted Riprap. Although the state of the practice in partial grouting of riprap has achieved a high level of reliability and sophistication through methods advanced in Germany, the research team did not identify a site to examine where partially grouted riprap was used as a specific application for mitigating pier scour at a bridge.

Four project sites in Germany were examined where partial grouting of riprap was used. Two of these sites were actively under construction at the time of the visit. The nature of the design loading at the four sites included barge-induced draw-down, barge-induced wave attack, propeller wash, coastal wave attack, high-velocity currents associated with the operation of gates at a tidal storm surge barrier, and anchor drag. In addition, design methods, laboratory test procedures, and field placement quality assurance/quality control (QA/QC) procedures were reviewed with German researchers and contractors.

The following four sites using partially grouted riprap were examined:

- Dortmund-Ems Canal (bed and bank revetment, primarily to mitigate barge-induced hydraulic loading)
- Wilhelmshaven Harbor (coastal wave attack environment)



Figure 2.18. Close-up of grout interface between ACB system and pile cap at pier.

- Elbe River highway tunnel beneath waterway (protection against accidental anchor drag)
- River Ems storm surge barrier (high-velocity currents created during gate operation)

Partial grouting of riprap results in a revetment matrix that achieves greater stability than an ungrouted installation. This greater stability implies that for a given hydraulic loading, a smaller class of riprap can be used when partial grouting is incorporated in the design. The grout creates a larger effective aggregate size, while maintaining a suitable degree of porosity and permeability of the installation (Figure 2.19). This characteristic is obviously desirable under conditions where large hydraulic gradients can occur. Partially grouted riprap also maintains a degree of flexibility compared to rigid, fully grouted rock and can therefore withstand moderate amounts of differential settlement or frost heave without losing integrity.



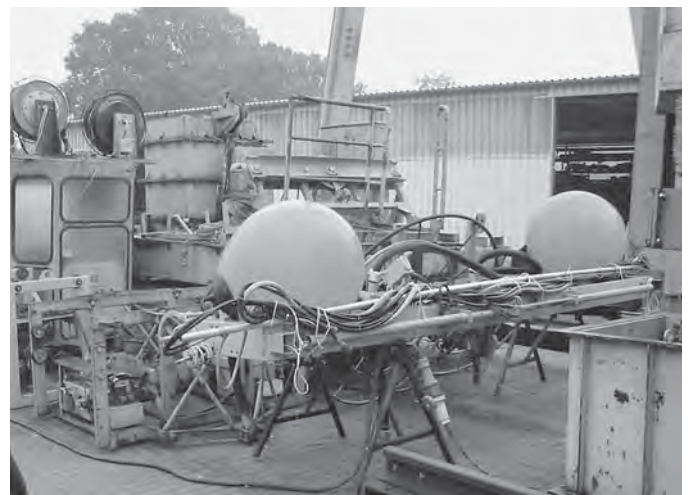
Figure 2.19. Larger effective aggregate size of partially grouted riprap.

Partial grouting can be performed both in the dry and under water. In the latter case, polymer admixtures are included in the mix design to prevent segregation during placement; the generic term for this specialized grout mix is “anti-wash” or “sticky” concrete. Underwater placement is typically accomplished by a global positioning system (GPS)–positioned frame that holds multiple injection nozzles approximately 1 ft above the surface of the riprap (Figure 2.20).

Considerable guidance for the design, installation, and associated testing (both laboratory and field) of partially grouted riprap has been developed by the BAW. Additionally, considerable effort has gone into the understanding of filter requirements and the development of guidelines for the testing, selection, and placement of filters for revetment and scour protection applications.

The NCHRP Project 24-07 research team identified a lack of guidelines for the selection and placement of filters for scour protection works in the United States, and developed preliminary recommendations to accommodate this need. More recent work accomplished in the United States under NCHRP Project 24-23, “Riprap Design Criteria, Specifications, and Quality Control” (Lagasse et al. 2006), combined with the guidance and information derived from the German experience, has effectively bridged this gap. A demonstration project in the United States of placement of partially grouted riprap during Phase 2 represented an ideal vehicle for importing German technology to the United States (see discussion in Section 3.5). Figures 2.21, 2.22, and 2.23 show an installation in Germany being placed in the dry using a small, mobile batch plant and five-man crew.

Geotextile Containers. Geotextile sand containers are made of very thick, high-strength non-woven geotextile fabric



Source: Colcrete–Von Essen Inc.

Figure 2.20. Grouting frame used for underwater placement.



Source: Gewatech-Soil and Hydraulic Engineering

Figure 2.21. *Partial grouting of riprap performed in the dry.*

that is premanufactured as an open-ended “pillow,” filled with sand, and sewn shut at the end. Standard sizes manufactured in Germany are 0.25, 0.50, and 1.0 m³ in volume. The non-woven geotextile, typically 4.6 to 6 mm or thicker, provides exceptional elongation (stretching) before it begins to tear or



Source: Gewatech-Soil and Hydraulic Engineering

Figure 2.22. *Close-up of completed partial grout installation.*



Source: Gewatech-Soil and Hydraulic Engineering

Figure 2.23. *Mobile batch plant used for partial grouting in dry conditions.*

rupture. The research team was not able to visit and examine any field sites utilizing sand-filled geotextiles, although the filling, sewing, and placement techniques were demonstrated in the construction yard of Colcrete-Von Essen in Rastede, Germany (Figure 2.24).

The geotextiles are filled to 80% capacity with sand and sewn shut. Because they are not filled completely, they remain flexible and deformable. The containers may represent a particularly well-suited means of filling existing scour holes under water, where the site cannot be dewatered or where strong currents prevail. Placement by conventional construction equipment is readily achieved (Figure 2.25). Although the BAW has developed laboratory tests and design guidelines for strength, abrasion resistance, and puncture resistance, the long-term survivability of the geotextile containers as a stand-alone countermeasure is not known, particularly in a high-bedload environment.

2.3 Merits and Deficiencies of Pier Scour Countermeasures

2.3.1 Life-Cycle Factors

This section presents an evaluation of merits and deficiencies of life-cycle factors for each scour countermeasure type as assessed in the context of pier scour applications. The evaluation considers existing design, installation, and maintenance guidance derived primarily from HEC-23 and modified, where applicable, by the NCHRP Project 24-07 final report, review of more current literature and studies, the investigation of field sites both in the United States and Germany, and the experience and judgment of the research team.



Source: Colcrete–Von Essen Inc.

Figure 2.24. Geotextile sand container (1.0 m³) being filled from hopper.

The factors considered include the following:

- Selection criteria
- Design specifications and guidelines
- Construction specifications and guidelines
- Maintenance and inspection guidelines
- Performance evaluation guidelines
- Life-cycle cost information

Selection Criteria

This factor was considered from the technical, if somewhat qualitative, standpoint of functional application in a specific river environment under a given set of design hydraulic, geometric, and sediment transport conditions. Other evaluation criteria, such as constructability, maintenance, and cost, also play an important role in the selection of a pier scour countermeasure for application at a specific site. The HEC-23 countermeasure suitability matrix provides a valuable background for this guidance (Lagasse et al. 2001).



Source: Colcrete–Von Essen Inc.

Figure 2.25. Conventional construction equipment handling sand-filled geotextile container.

Design Specifications and Guidelines and Performance Evaluation Guidelines

These factors were used to form the basis for structuring the laboratory testing activities for this project. These two criteria must incorporate the differences in functional application of the various countermeasures as well as the failure mechanisms unique to each countermeasure.

Construction Specifications and Guidelines

Construction specifications and guidelines consider the different needs and challenges required for placing a countermeasure in the dry as well as installing it under water, or in flowing water. In addition, the requirement for specialized equipment must be addressed. For example, the equipment requirements, placement techniques, and construction QA/QC sampling and testing procedures for partially grouted riprap are quite straightforward when working in the dry; however, placement under water requires much more specialized equipment and greater degree of sophistication. Grading requirements and placement tolerances also vary among countermeasure types. For example, a relatively thin veneer of articulating concrete blocks requires finer grading techniques than an equivalent, and much thicker, riprap layer. Alternative placement techniques, particularly for rock riprap, typically dictate the strength requirements for geotextiles in order to meet construction survivability criteria.

Maintenance and Inspection Guidelines

Maintenance and inspection guidelines will vary greatly among countermeasure types. Underwater or buried installa-

tions require different considerations to ensure that the countermeasure can be adequately inspected, compared to surficial treatments in ephemeral or intermittent stream environments. Maintenance requirements can range from “dump more and larger” in the case of riprap to “remove, redesign, repair, and replace” activities for manufactured systems. Even for a single countermeasure type, such as articulating concrete blocks, maintenance requirements may differ depending on the type of damage or deterioration the countermeasure has suffered.

Life-Cycle Cost

Life-cycle cost information is difficult to quantify. Initial construction costs are relatively easy to develop; however, even for a specific countermeasure, these costs can vary widely depending on regional availability of materials, site conditions, and access constraints. Therefore, a countermeasure

type can be very cost effective in one locale and prohibitively expensive in another. Extending these issues to life-cycle maintenance requirements requires an even broader set of assumptions. Riprap, for example, is a standard countermeasure type in many states; however, alternatives to riprap may need to be investigated because of cost and availability limitations. The risks and consequences of failure at any given site further complicate the issue. For these reasons, life-cycle costs were not considered in the tabulation of merits and deficiencies, but are the focus of the countermeasure selection methodology developed in Chapter 3.

2.3.2 Merits and Deficiencies by Life-Cycle Factors

The factors are presented and discussed in Tables 2.4 through 2.9, organized by countermeasure type. Because the

Table 2.4. Riprap merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • HEC-23 suitability matrix provides qualitative guidance • Flexible and porous • Layer thickness allows self-healing 	<ul style="list-style-type: none"> • Often used as default countermeasure with little or no design • Required rock size not always economically available
Design specifications and guidelines	<ul style="list-style-type: none"> • Sizing and gradation criteria well established • Rock suitability requirements well established • Filter characteristics well established (geotextile and/or granular) • HEC-23 provides baseline design guidance 	<ul style="list-style-type: none"> • Layout dimensions preliminary • Filter often overlooked • Prior excavation recommended • Relative impact of various scour mechanisms not well understood • Contraction scour and long-term degradation not considered when determining extent
Construction specifications and guidelines	<ul style="list-style-type: none"> • Standard construction equipment typically used for placement • Can be placed under water or dry • Can accommodate irregular subgrade conditions 	<ul style="list-style-type: none"> • Geotextile filter preferred but difficult to place under water (see geotextile sand containers, Table 2.6) • Effective filter seal against pier required • Difficult to place larger stone in areas with limited access beneath bridge deck
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event • Inspectors are familiar with this countermeasure • Maintenance consists essentially of "dumping more" 	<ul style="list-style-type: none"> • Visual inspection not always reliable because of launching and subsequent redeposition; may require supplemental probing
Performance evaluation guidelines	<ul style="list-style-type: none"> • HEC-23 provides current design recommendations, including prior excavation • Riprap will provide laboratory benchmark to determine performance for this research • Modes of failure well known (typically include particle dislodgement, substrate winnowing, or edge deterioration) 	<ul style="list-style-type: none"> • Phase 2 laboratory testing required to establish benchmark • Phase 2 laboratory testing required to determine adequacy of filter recommendations • Phase 2 laboratory testing required to confirm edge details • Contraction scour and long-term degradation not considered when determining extent
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

Table 2.5. Partially grouted riprap merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • Porous • More flexible than fully grouted riprap • Allows smaller riprap class to be used compared to standard (ungROUTED) riprap 	<ul style="list-style-type: none"> • Less flexible than standard riprap • General lack of familiarity with product in the United States
Design specifications and guidelines	<ul style="list-style-type: none"> • German manuals (MAR, MAV, RPV) provide materials, testing, and installation specifications • HEC-23 provides baseline design guidance for standard riprap • Sizing and gradation criteria well established • Rock suitability requirements well established • Filter characteristics well established • Grout mix specifications well established 	<ul style="list-style-type: none"> • Layout dimensions preliminary • Termination/edge treatment uncertain • Filter extent uncertain • Prior excavation preferred • Hydraulic loading conditions for pier scour application not well defined • Relative impact of various scour mechanisms not well understood • Contraction scour and long-term degradation not considered when determining extent and layer thickness
Construction specifications and guidelines	<ul style="list-style-type: none"> • Can be placed under water or dry • For dry placement, grout can be installed by hand or by mechanized injection frames • Can accommodate irregular subgrade conditions 	<ul style="list-style-type: none"> • Specialized equipment required for underwater grout placement • Specialized grout mix required • Geotextile filter preferred but difficult to place under water (see geotextile sand containers, Table 2.6) • Effective filter seal against pier required
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event • Repair is straightforward 	<ul style="list-style-type: none"> • May be difficult to detect voids beneath bridged areas under water • Inspectors not likely to be familiar with this countermeasure
Performance evaluation guidelines	<ul style="list-style-type: none"> • Exposure of filter or bedding underlayer generally recognized as threshold of performance 	<ul style="list-style-type: none"> • Phase 2 laboratory tests needed to benchmark against standard riprap specifically for pier scour applications
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

long-term survivability of geotextile sand containers is not known, particularly in a high bedload riverine environment, the application considered in this study will be as a filter for either a riprap or partially grouted riprap armor layer. No attempt has been made to rank the factors or provide any quantitative measure for comparison between and among countermeasures. The objective was to identify gaps in the current state of the practice for use in developing a preliminary set of recommended guidelines for each countermeasure, and to lay the groundwork for structuring a laboratory investigation program to address these gaps.

Deficiencies addressed in this study that are common to most of the countermeasures described in these tables include (1) countermeasure extent and edge details, (2) filter extent requirements, and (3) application to pier scour conditions (in comparison to riprap). Deficiencies addressed that are specific to individual countermeasures include (1) riprap

stability as a benchmark for comparison and (2) guidance for selecting an appropriate target factor of safety for pier scour applications using ACB or grout-filled mattress systems.

Deficiencies that are **not** addressed in this study that are common to most of the countermeasures described in the tables include (1) relative impact of various combinations of scour mechanisms (i.e., only local pier scour will be considered); (2) countermeasure extent and/or thickness requirements related to contraction scour and degradation; and (3) testing of proprietary systems.

2.3.3 Summary

The information presented in Tables 2.4 through 2.9 is qualitative in nature. The discussion of merits and deficiencies is not intended to provide a ranking or prioritization of countermeasures; however, the information can be used to

Table 2.6. Geotextile sand container merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • Can be used to repair/fill existing scour hole • Flexible and porous • Geotextile can be selected for compatibility with riverbed sediments • Can provide a filter layer(s) underneath a more durable top layer 	<ul style="list-style-type: none"> • General lack of familiarity with product in the United States • General lack of experience in applications specific to pier scour • Armor top layer recommended
Design specifications and guidelines	<ul style="list-style-type: none"> • Filter characteristics well established • German manual (RPG) provides specifications for laboratory testing methods for required physical properties of geotextile 	<ul style="list-style-type: none"> • Suitability for use without an armor top layer has not been demonstrated; therefore, combination approach is recommended (see Tables 2.4 and 2.5) • Relative impact of various scour mechanisms not well understood
Construction specifications and guidelines	<ul style="list-style-type: none"> • Can be placed under water or dry • Effective seal against pier easily accomplished • Use of heavy (> 4 mm) non-woven fabric provides protection against puncture or tear during installation • Multiple layering of containers ensures overlap to prevent substrate leaching/winning • Can accommodate irregular subgrade conditions 	<ul style="list-style-type: none"> • Finished surface of containers may not be suitable for use with armor that requires a fine finish grade (e.g., articulating concrete blocks)
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event • Flexibility and overlap minimizes potential for voids within system • Inspection of armor top layer commensurate with that particular countermeasure type 	<ul style="list-style-type: none"> • Inspectors not likely to be familiar with this countermeasure
Performance evaluation guidelines	<ul style="list-style-type: none"> • NCHRP 24-07(Phase 1) research team recommended the investigation of sand-filled bags as a flexible, non-rigid alternative to grout-filled bags 	<ul style="list-style-type: none"> • Phase 2 testing recommended
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

distinguish differences between countermeasures with respect to certain attributes.

The information provided in this section is derived from previous studies, drawing from the NCHRP Project 24-07 research effort, and supplemented by the experience and judgment of the research team. The primary objective of this section is to identify those factors that are suitably developed for recommending baseline design, installation, and maintenance guidelines for pier scour countermeasures; conversely, the secondary objective is to identify gaps in the state of practice where additional laboratory and field work are warranted.

The results of this section were used to develop a recommended course of laboratory work to bridge any knowledge gaps or shortcomings of a particular countermeasure prior to the development of implementation guidelines for field application.

Chapter 3 presents an overview of the laboratory testing program and test results for the following pier scour countermeasures:

- Riprap
- Partially grouted riprap and geotextile containers
- ACB systems
- Gabion mattresses
- Grout-filled mattresses

Interpretation and appraisal of the findings of this chapter and the laboratory testing results are combined in Chapter 3 to develop guidelines and specifications for design and construction; guidelines for inspection, maintenance, and performance evaluation; and practical selection criteria for each countermeasure type.

Table 2.7. Articulating concrete block merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • HEC-23 suitability matrix provides qualitative guidance • Relatively flexible and porous • Can accept vegetation where desired • Thinner layer provides equivalent protection compared to standard riprap 	<ul style="list-style-type: none"> • Single-layer "veneer" does not allow for self-healing • General lack of familiarity with product • Full-scale, product-specific performance testing required • Typically proprietary
Design specifications and guidelines	<ul style="list-style-type: none"> • Stability design criteria (sizing requirements) well established • Material characteristics per ASTM D 6684 • Filter characteristics well established • HEC-23 provides baseline design guidance • In general, little or no prior excavation required because of low profile • Target Factor of Safety can be adjusted for site-specific conditions 	<ul style="list-style-type: none"> • Layout dimensions preliminary • Termination/edge treatment uncertain. Pre-excavated turndowns at edges recommended • Filter extent uncertain • Guidance for selecting target factor of safety not established • Contraction scour and long-term degradation not considered when determining extent
Construction specifications and guidelines	<ul style="list-style-type: none"> • Can be placed under water or dry • Can be individually hand placed or placed as pre-cabled mats • Hand placement allows access to confined areas, but typically limited to dry installations • Geotextile may be attached directly to pre-cabled mat 	<ul style="list-style-type: none"> • Geotextile filter preferred but difficult to place under water • Effective filter seal against pier required • Underwater placement of pre-cabled ACB mats may be difficult where access directly beneath bridge deck is limited • Subgrade preparation and block placement more stringent than riprap
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event 	<ul style="list-style-type: none"> • Inspectors not likely to be familiar with this countermeasure • May be difficult to detect voids under bridged areas of blocks if these areas are under water • Single-layer "veneer" does not allow for self-healing • Underwater repair difficult
Performance evaluation guidelines	<ul style="list-style-type: none"> • NCHRP 24-07 (Phase 1) report provided favorable performance review • Overturning of mat or individual blocks most typical mode of failure (mode of failure well known) 	<ul style="list-style-type: none"> • Phase 2 laboratory testing required to determine adequacy of filter recommendations and edge details to supplement 24-07(1) findings
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

Table 2.8. Grout-filled mattress merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • Porosity provided by pre-manufactured filter points • Thinner layer provides equivalent protection compared to standard riprap • Materials and methods of construction allow placement in areas of restricted access • Typically used to provide continuous protection across full width of crossing 	<ul style="list-style-type: none"> • HEC-23 suitability matrix does not address applications at piers • Single-layer "veneer" does not allow for self-healing • General lack of familiarity with product • Essentially rigid • Cannot support vegetation unless pre-excavated and buried below rooting depth • Typically proprietary
Design specifications and guidelines	<ul style="list-style-type: none"> • Stability design criteria (sizing requirements) well established • Grout characteristics per ASTM D 6449 • Fabric characteristics per ASTM D 6685 • Filter characteristics well established • HEC-23 provides baseline design guidance • Target factor of safety can be adjusted for site-specific conditions 	<ul style="list-style-type: none"> • Layout dimensions preliminary • Termination/edge treatment uncertain. Pre-excavated turndowns at edges recommended • Filter extent uncertain • Guidance for selecting target factor of safety not established • Rigidity requires calculation of load transfer to piers unless tension anchors provided on upstream edge • Contraction scour and long-term degradation not considered when determining extent
Construction specifications and guidelines	<ul style="list-style-type: none"> • Can be placed under water or dry • Typically can accommodate irregular subgrade conditions 	<ul style="list-style-type: none"> • Geotextile filter preferred but difficult to place under water • May be difficult to place and secure fabric in flowing water • Effective filter seal against pier required
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event 	<ul style="list-style-type: none"> • Inspectors not likely to be familiar with this countermeasure • Very difficult to detect voids under bridged areas of mattress, whether under water or not • Single-layer "veneer" does not allow for self-healing • Underwater repair difficult
Performance evaluation guidelines	<ul style="list-style-type: none"> • Undermining or flanking are most typical modes of failure (mode of failure well known) 	<ul style="list-style-type: none"> • Not investigated under NCHRP 24-07 (Phase 1) • Flanking, undermining, and uplift most likely modes of failure • Phase 2 laboratory testing required to determine adequacy of filter recommendations and edge details
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

Table 2.9. Gabions and gabion mattress merits and deficiencies.

Factor	Merits	Deficiencies
Selection criteria	<ul style="list-style-type: none"> • HEC-23 suitability matrix provides qualitative guidance • Porous • Gabion mattresses flexible; gabions less so • Thinner layer provides equivalent protection compared to standard riprap 	<ul style="list-style-type: none"> • Not recommended for coarse bedload environments because of potential for abrasion of wire • Even galvanized or PVC-coated wire baskets not proven for long-term use in moderately saline environments • Typically proprietary
Design specifications and guidelines	<ul style="list-style-type: none"> • NCHRP 24-07 (Phase 1) report provides baseline design guidance • Wire basket characteristics per ASTM A 974 (welded wire) or A 975 (twisted wire) • Filter characteristics well established • Rock fill per ASTM D 6711 	<ul style="list-style-type: none"> • Sizing requirements for pier scour applications are preliminary • Layout dimensions lacking • Termination/edge treatment uncertain. Pre-excavated turndowns at edges recommended • Filter extent uncertain • Relative impact of various scour mechanisms not well understood • Contraction scour and long-term degradation not considered when determining extent
Construction specifications and guidelines	<ul style="list-style-type: none"> • Can be placed under water or dry • Can be filled individually or placed as pre-filled mattresses • Individual filling allows access to confined areas, but typically limited to dry installations • Geotextile may be attached directly to pre-assembled basket prior to filling • Typically can accommodate irregular subgrade conditions 	<ul style="list-style-type: none"> • Effective filter seal against pier required • Prior excavation recommended
Maintenance and inspection guidelines	<ul style="list-style-type: none"> • Standard 2-year inspection frequency, and recommended inspection after a flood event • Flexibility of mattresses minimizes potential for bridging over voids 	<ul style="list-style-type: none"> • Inspectors not likely to be familiar with this countermeasure • Underwater repair difficult • Vandalism of wire baskets has been reported as a concern, especially in urban areas • Repair often requires total replacement of the individual baskets involved
Performance evaluation guidelines	<ul style="list-style-type: none"> • Performance threshold typically associated with excessive movement of rockfill within basket, exposing filter layer (mode of failure well known) 	<ul style="list-style-type: none"> • NCHRP 24-07 (Phase 1) did not perform laboratory testing on this type of countermeasure • Phase 2 laboratory testing required to determine adequacy of filter recommendations and edge details • Other performance issues include general flanking/undermining
Life-cycle cost information	<ul style="list-style-type: none"> • Not considered 	<ul style="list-style-type: none"> • Not considered

CHAPTER 3

Testing, Interpretation, Appraisal, and Results

3.1 Introduction

This chapter presents a summary of the research approach and results of laboratory testing of the following selected pier scour countermeasures:

- Riprap
- Partially grouted riprap and geotextile containers
- Articulating concrete blocks
- Gabion mattresses
- Grout-filled mattresses

The summary of the current state of practice in Chapter 2 is combined with an interpretation and appraisal of testing results to provide guidelines and specifications for design and construction, and guidelines for inspection, maintenance, and performance evaluation for the pier scour countermeasures investigated in this study.

Existing design equations for sizing the armor component of each countermeasure were used to develop the laboratory testing program. However, sizing the armor is only the first step in the comprehensive design, installation, inspection, and maintenance process required for a successful countermeasure. A countermeasure is an integrated system that includes the armor layer, filter, and termination details. Successful performance depends on the response of each component of the system to hydraulic and environmental stresses throughout its service life. In this context, filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also necessary.

In addition, a countermeasure selection methodology was developed. It provides an assessment of the suitability of each of five specific countermeasure types based on a variety of factors involving river environment, construction considerations, maintenance, performance, and estimated life-cycle cost of each countermeasure. The output from the selection

method provides a quantitative ranking of countermeasure types by computing a Selection Index. The Selection Index includes a fatal-flaw mechanism to identify situations where a particular countermeasure is unequivocally unsuitable due to one or more circumstances unique to the site being evaluated. The Selection Index is intended to identify the countermeasure best suited for application at a particular site.

To guide the practitioner in developing appropriate scour countermeasure designs and ensuring successful installation and performance of countermeasures at bridge piers, the findings of Chapter 2 and recommendations of Chapter 3 are combined to provide a detailed set of design guidelines for each countermeasure type as stand-alone appendices:

- Appendix C, Guidelines for Pier Scour Countermeasures Using Rock Riprap
- Appendix D, Guidelines for Pier Scour Countermeasures Using Partially Grouted Riprap
- Appendix E, Guidelines for Pier Scour Countermeasures Using Articulating Concrete Block (ACB) Systems
- Appendix F, Guidelines for Pier Scour Countermeasures Using Gabion Mattresses
- Appendix G, Guidelines for Pier Scour Countermeasures Using Grout-Filled Mattresses

3.2 Laboratory Studies

3.2.1 Overview

Laboratory research conducted for this study was performed at the Hydraulics Laboratory of CSU, located at the Engineering Research Center (ERC). CSU's indoor Hydraulics Laboratory is 280 ft (85 m) long by 120 ft (37 m) wide with a maximum ceiling clearance of 32 ft (9.8 m). Covered laboratory space for testing and models exceeds 20,000 ft² (1858 m²). Figure 3.1 is a photograph of the Hydraulics



Figure 3.1. Photograph of CSU's Hydraulics Laboratory.

Laboratory, and Figure 3.2 shows a plan view of the Hydraulics Laboratory with a listing of the available flumes and floor model space.

As indicated in Figure 3.2, the Hydraulics Laboratory maintains and operates a wide selection of flumes. Table 3.1 outlines the dimensions and capacities of the flumes available.

Testing conducted for NCHRP Project 24-07(2) utilized the largest of the laboratory recirculating flumes. The flume is 8 ft (2.4 m) wide by 4 ft (1.2 m) deep by 200 ft (61 m) long and capable of recirculating water and sediment over a range of slopes up to 2%. The maximum discharge in the flume is 100 cfs (2.8 m³/s) with a series of sediment pumps capable of transporting particle sizes up to 0.5 in. (2.7 mm).

A mobile data acquisition cart traverses the flume and provides flexibility in data collection. Any number of point gages or 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

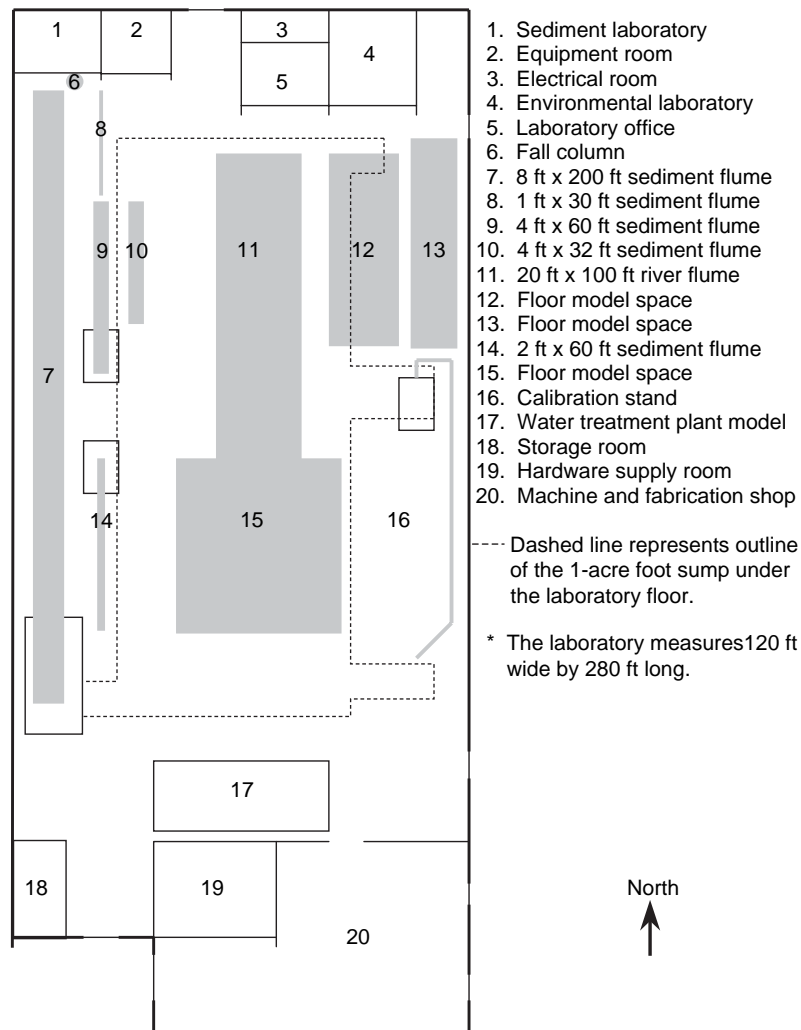


Figure 3.2. Schematic of CSU's Hydraulics Laboratory.

Table 3.1. Dimensions and capacities of flumes in the Hydraulics Laboratory.

Flumes/Models	Width (ft)	Length (ft)	Depth (ft)	Flow (cfs)	Slope (%)	Recirc. (Yes/No)
Sediment Flume 7	8	200	4	100	Variable	Yes
Sediment Flume 10	4	32	4	12	Variable	Yes
Sediment Flume 9	4	60	4	20	Variable	Yes
Sediment Flume 14	2	60	2	10	Variable	Yes
Sediment Flume 8	1	30	1	3	Variable	Yes
Sediment Flume 11	20	100	3	40	Variable	No
Sediment Flumes 12, 13, 15	100	100	10	40	Variable	Yes

collection. The flume is also equipped with a Plexiglas wall for flow and scour visualization. Plexiglas walls provide an ideal viewpoint for flow visualization using dyes or visual scour monitoring. Figure 3.3 provides a schematic of the flume, data cart, and ancillary components.

3.2.2 Research Approach

Laboratory testing was structured to address the countermeasure deficiencies as identified in Section 2.3. Specifically, the design specifications and guidelines and the performance evaluation guidelines were used to design the testing program (see Section 2.3.1). For each of the five countermeasure types, a specific testing approach was developed addressing the deficiencies reported in Tables 2.4 through 2.9. Geotextile sand containers were tested as a filter for partially grouted riprap.

To maximize the amount of testing within the available budget, the research team and the NCHRP Project 24-07(2) panel laboratory subgroup met in August 2002 to develop a

prioritized plan of study. The plan was further modified by the laboratory subgroup in January 2003. From these discussions came the decision to place three piers along the centerline of the testing flume. Square piers 8 in. (200 mm) long by 8 in. (200 mm) wide were used. Spacing between the piers was approximately 40 ft (12 m) to ensure the formation of identical flow lines upstream of each pier. Sand, with a d_{50} ranging from 0.7 to 0.9 mm, was placed in the flume to a depth of approximately 18 in. (460 mm). The flume layout is shown in Figure 3.3.

A matrix of flume tests was completed for the research program. Each 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 sediment size utilized throughout the research program. The V_{crit} and $2V_{crit}$ runs were performed without sediment recirculation. Separate runs on selected countermeasure configurations were performed at $2.5V_{crit}$ with sediment recirculation, therefore, both clear-water and live-bed conditions were examined.

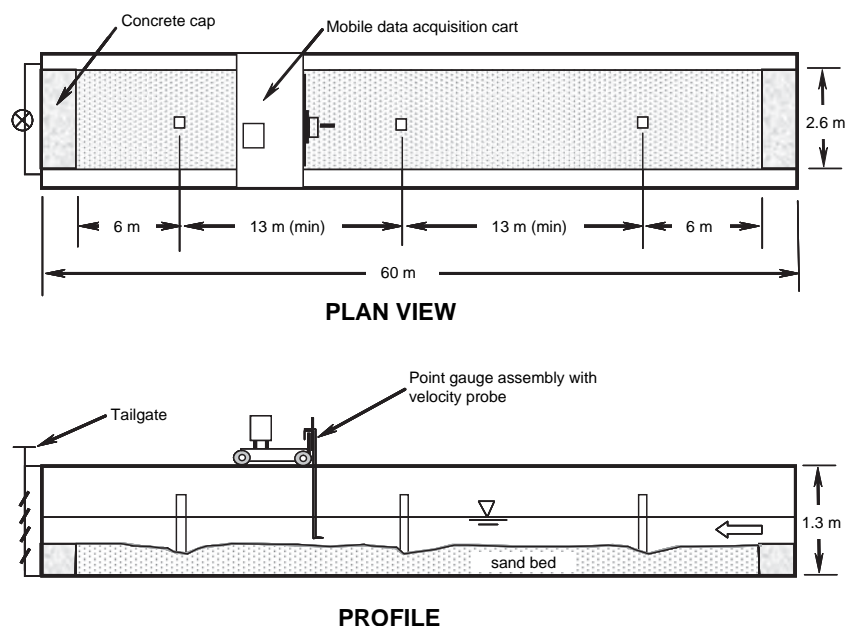


Figure 3.3. Schematic of flume and configuration.

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 flow was performed at $3V_{\text{crit}}$ to determine the baseline 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 of each test. Water surface elevations were collected every 4 ft (1.2 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 ± 0.005 ft (1.5 mm). Velocities were collected with a three-dimensional (3-D) acoustic Doppler velocimeter, accurate to $\pm 2\%$. Where the flow depth was sufficient, 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 a total station. Survey resolution was sufficient to accurately map each scour hole and document system performance.

3.2.3 Laboratory Test Plan

Items identified as gaps in the current state of the practice (Section 2.3) were reviewed and a specific test, or series of tests, was designed to address each deficiency. The following sections detail the findings for each countermeasure type. Each test series was designed to permit one configuration to be carried forward to the next series. This design served to quantify the repeatability of the test program as well as identify inconsistencies that could arise in the experimental set up.

The laboratory tests were not designed to replicate any particular prototype-scale conditions. For example, the $2V_{\text{crit}}$ run was not intended to represent specific scale ratio of a prototype pier or flow condition. However, in each case, the test countermeasure was “designed” to withstand the $2V_{\text{crit}}$ hydraulic condition. For example, the riprap size was selected such that particle dislodgement or entrainment was not anticipated during the $2V_{\text{crit}}$ run. This design did not mean that the riprap (or any other countermeasure) would not fail because of other factors, such as settling, edge undermining, or winnowing of substrate material. Runs utilizing an approach velocity of $2.5V_{\text{crit}}$ were intended to take each system to failure by particle dislodgement.

The performance of each countermeasure was compared with the benchmark performance of riprap. Criteria for rating performance were consistent between countermeasures, but were not necessarily identical for all countermeasures. A countermeasure was considered to have failed if the countermeasure (or its component parts) was dis-

lodged, lifted, or entrained. Relative performance was gauged by whether the countermeasure functioned as intended. Specifically, if settling along the countermeasure 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.

Sections 3.3 through 3.8 provide an overview of the laboratory testing program including the materials used and the design intent for each test series for each countermeasure type. Typical configurations and test runs are illustrated. **For each countermeasure, a “baseline” schematic is shown. These schematics are intended to illustrate the starting point for a test series, not a recommendation for design. The final recommendations for design layout are presented in the design guideline appendix for each countermeasure type. Appendix H provides summary tables of the testing program, and the Reference Document (<http://www.trb.org/TRBNet/ProjectDisplay.asp?ProjectID=702>) contains detailed laboratory testing results.** Section 3.9 summarizes design and specification guidance derived from the testing program as a basis for developing design guidelines for each countermeasure type.

3.3 Unprotected Runs

3.3.1 Materials

Sand composing the bed material was characterized by a d_{50} grain size that ranged from approximately 0.7 to 0.9 mm. The coefficient of uniformity, C_u , defined as d_{60}/d_{10} , ranged from 4.1 to 5.2. A representative grain size distribution graph is shown in Figure 3.4.

3.3.2 Testing

A conservative value of 1.0 ft/s (0.305 m/s) was adopted for establishing the target approach velocities. The intent was to create a condition for the initial run of each countermeasure type that resulted in true clear-water conditions, with no movement of the bed material except for local scour in the immediate vicinity of the piers. Tests confirmed that an approach velocity of 1.0 ft/s (0.305 m/s) resulted in no bed material movement except for local scour; runs performed at 2.0 ft/s (0.61 m/s) or greater resulted in live-bed conditions and the formation of dunes throughout the entire length of the flume.

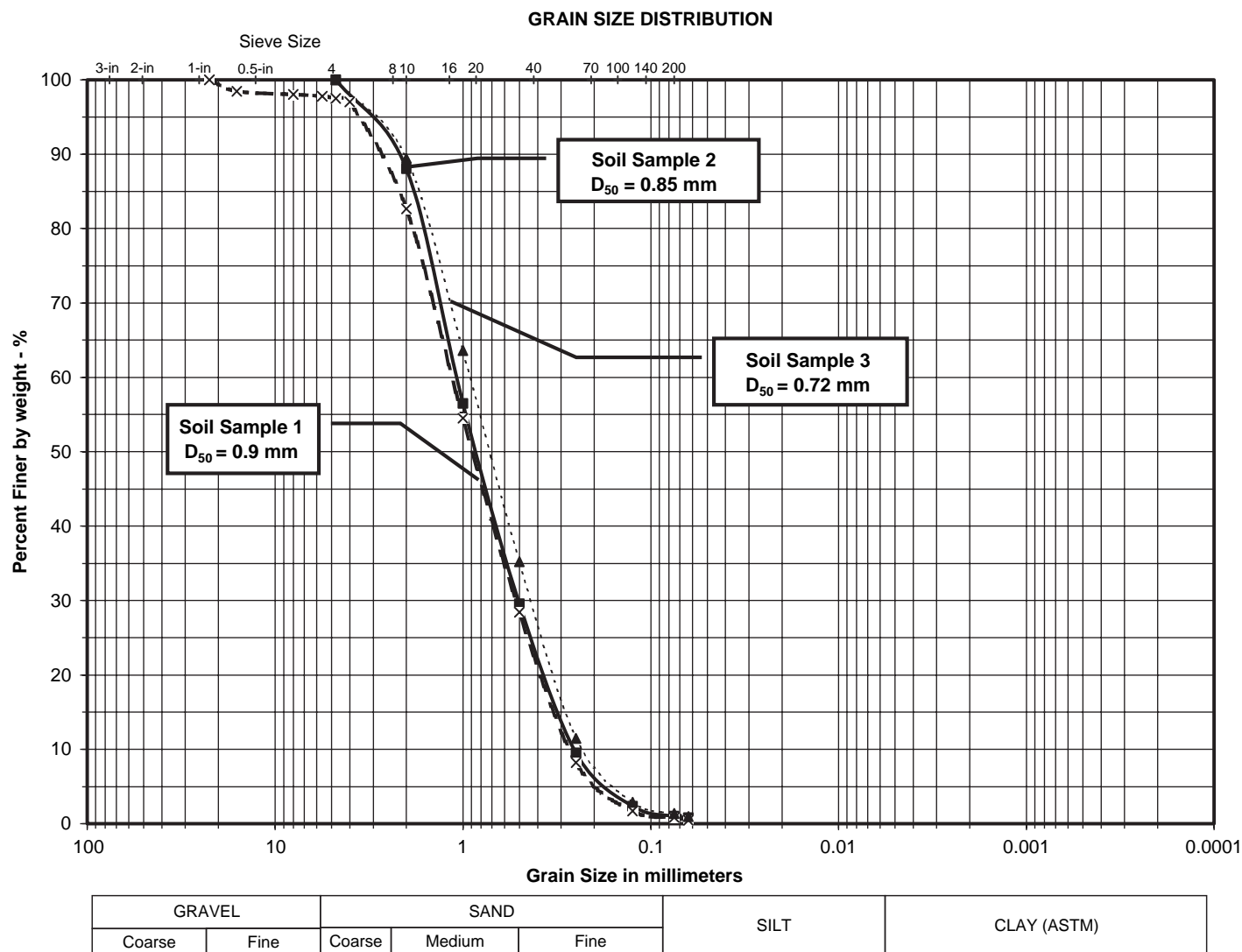


Figure 3.4. Grain size distribution of bed material.

Classification of maximum scour was determined for unprotected square and rectangular piers, under clear-water and live-bed conditions. A live-bed test was run for a sufficient duration (8 hours) to permit bed forms to migrate through the system. Figure 3.5 shows the results of unprotected square pier tests under live-bed conditions in the CSU indoor flume. Figure 3.6 shows the results for unprotected rectangular piers with 0° skew under clear-water conditions (Figure 3.6a) and a rectangular pier with 15° skew under live-bed conditions (Figure 3.6b) in the indoor flume. A white arrow indicates direction of flow on test run photographs.

3.4 Riprap

Most of the early work on the stability of pier riprap considers the size of the riprap stones and their ability to withstand high approach velocities and buoyant forces. Secondary

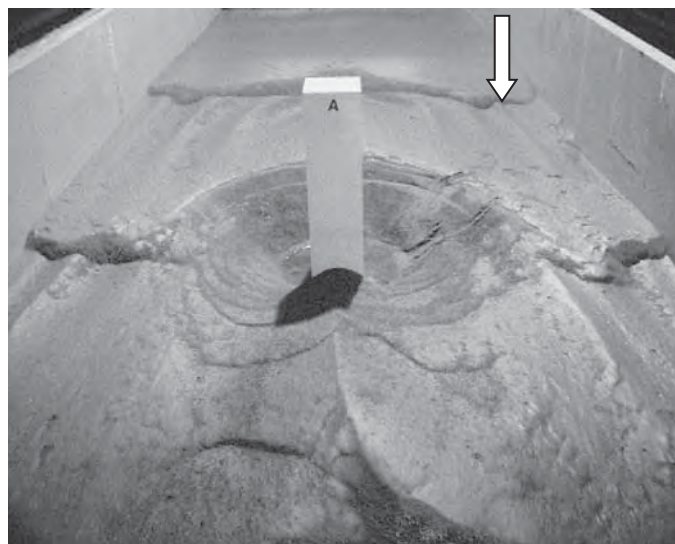
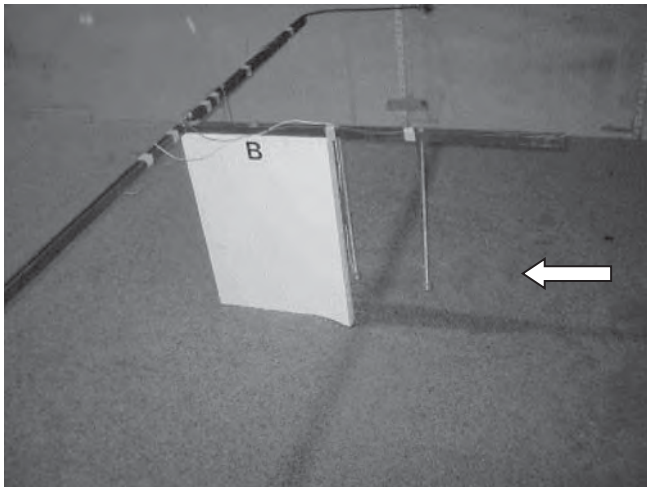
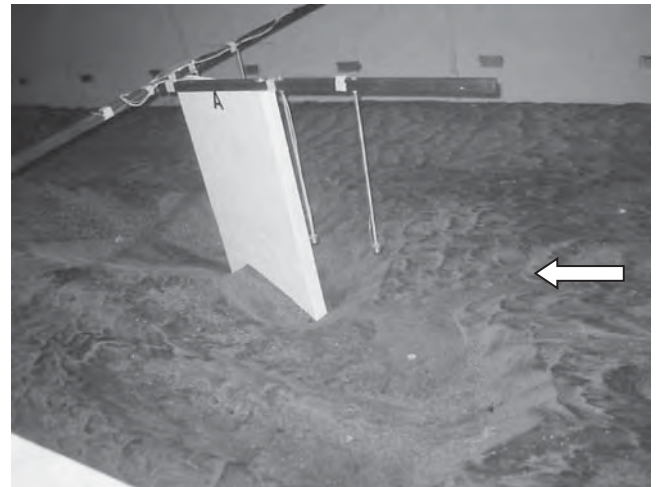


Figure 3.5. Unprotected square piers.



a. Unprotected rectangular pier (0° skew) after $1.0V_{crit}$ test.



b. Unprotected rectangular pier (15° skew) after $2.0V_{crit}$ test.

Figure 3.6. Unprotected rectangular piers.

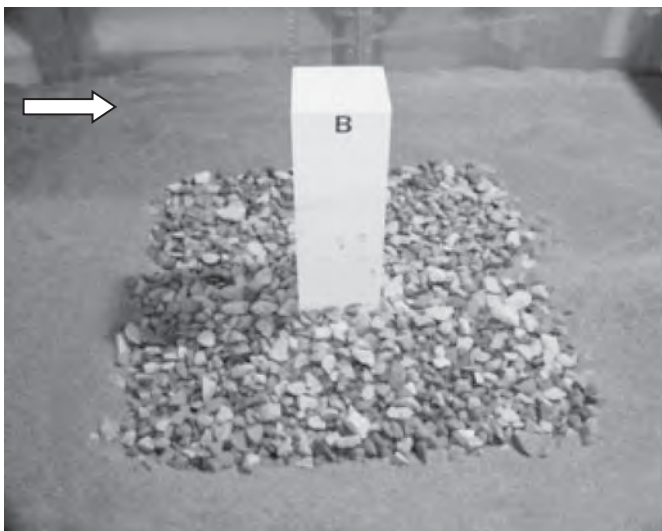
currents induced by bridge piers cause high local boundary shear stresses, high local seepage gradients, and sediment erosion from the streambed surrounding the pier. The addition of riprap also changes the boundary stresses (see Section 2.1.3).

Riprap failure at model bridge piers under clear-water conditions with gradually increasing approach flow velocities can be defined by three modes of failure:

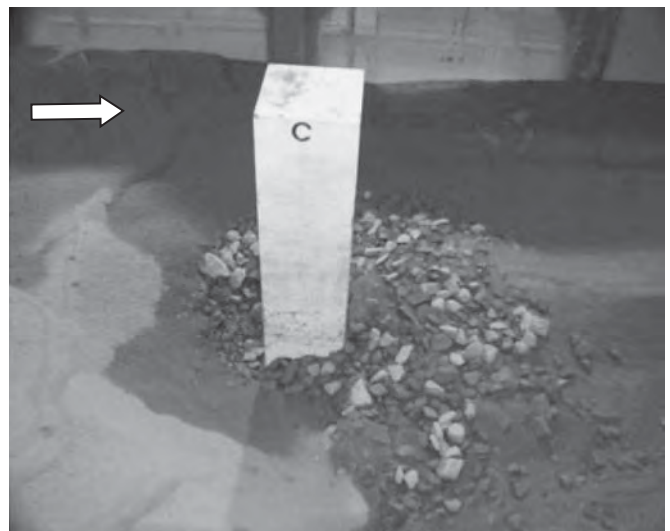
- Riprap shear failure, whereby the riprap stones cannot withstand the down flow and horseshoe vortex associated with the pier scour mechanism
- Winnowing failure, whereby the underlying finer bed material is removed through voids or interstices in the riprap layer

- Edge failure, whereby instability at the edge of the coarse riprap layer initiates a scour hole beginning at the perimeter and working inward until it ultimately destabilizes the entire layer

Prior research has 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 in sand bed rivers (see Section 2.1.3). Both clear-water and live-bed conditions were examined in this study, but the effects of contraction scour and long-term degradation were not investigated. Figure 3.7 shows the results of riprap tests under clear-water (Figure 3.7a) and live-bed (Figure 3.7b) conditions in the CSU indoor flume.



a. Riprap after $1.0V_{crit}$ test.



b. Riprap after $2.5V_{crit}$ test. Note particle displacement when areal extent is insufficient under live-bed conditions.

Figure 3.7. Riprap tests under clear-water and live-bed conditions.

3.4.1 Materials

Armor Stone

Riprap is the most commonly used pier scour countermeasure and usually consists of large stones placed around a pier to armor the bed. This armoring prevents the strong vortex flow at the front of the pier from entraining bed sediment and forming a scour hole. The ability of the riprap layer to provide scour protection is, in part, a function of stone size, which is a critical factor in terms of shear failure (Lagasse et al. 2006).

Riprap used for testing in the indoor flume was sized for stability at an approach velocity of $2V_{crit}$ in accordance with the procedures outlined in HEC-23 (see Equation 2.3) (Lagasse et al. 2001). As recommended in HEC-23, the cross-sectional average velocity was multiplied by 1.7 for a square-nose pier shape and 1.2 to account for flow distribution

across the flume, which yielded a design velocity of $(1.2)(1.7)(2V_{crit})$ or 4.1 ft/s (1.25 m/s) for the riprap sizing calculations.

Riprap d_{50} was determined using the standard Izbash formula for sizing riprap on a channel bed presented in HEC-23. The required d_{50} was 33 mm (1.3 in.). Two limiting gradation curves were developed given the riprap d_{50} of 33 mm, in accordance with guidelines presented in HEC-11 (Brown and Clyde 1989). See the Reference Document for computation details. Figure 3.8 shows the grain size distribution of riprap utilized in the testing program as well as the gradation limits. The riprap actually produced for the test runs had a d_{50} of 30 mm (1.2 in.) due to characteristics of the locally available supply.

The riprap size was selected such that particle dislodgement or entrainment was not anticipated during the $2V_{crit}$ run. However, the riprap could still fail due to other factors, such as settling, edge undermining, or winnowing of substrate

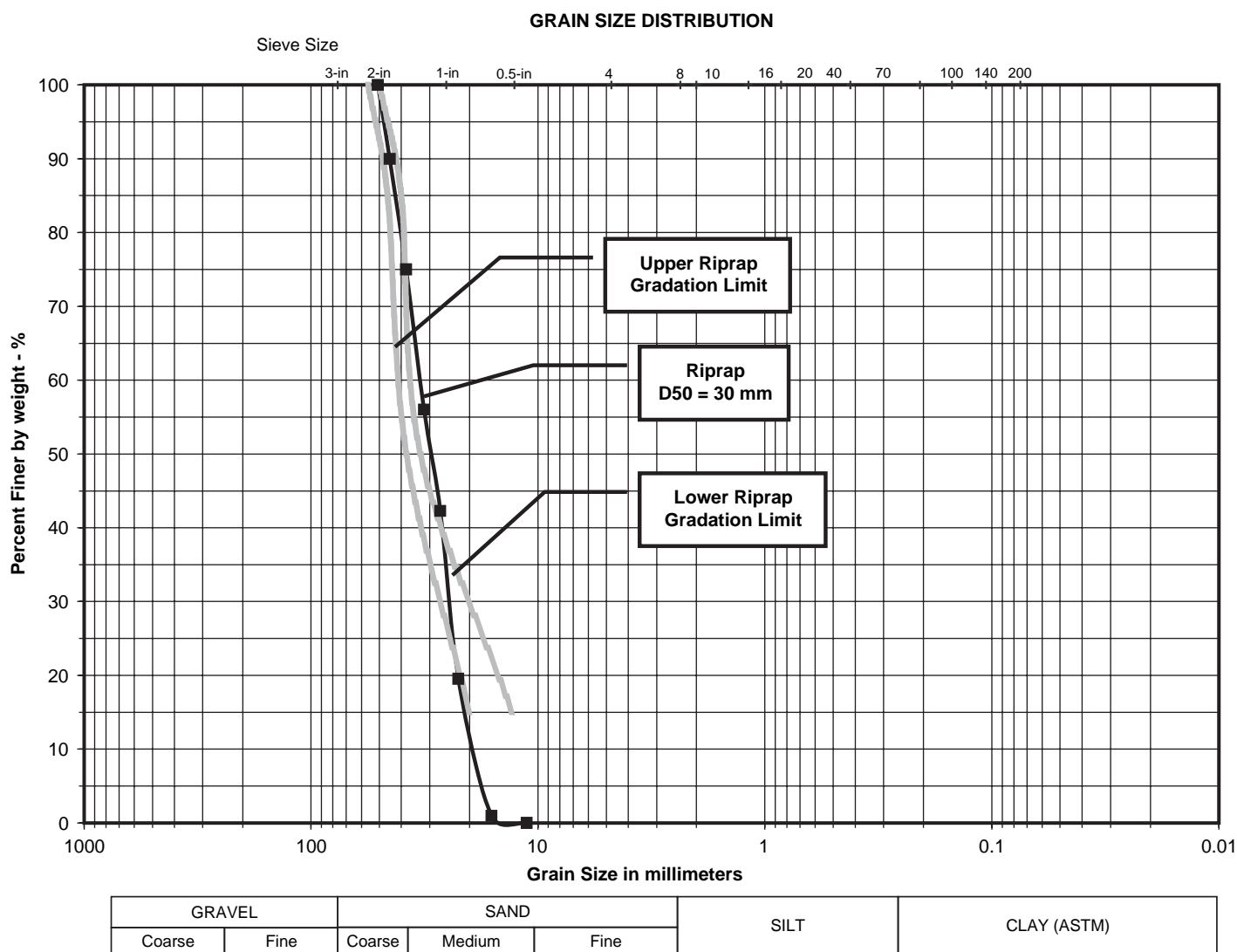


Figure 3.8. Riprap grain size distribution.

material. Riprap runs utilized approach velocities of $1V_{crit}$, $2V_{crit}$, and $2.5V_{crit}$. Runs utilizing an approach velocity of $2.5V_{crit}$ were intended to take each system to failure by particle dislodgement.

Riprap used in the laboratory tests consisted of a hard, durable sandstone having a specific gravity of 2.55 to 2.60. Other types of rock materials having different densities were not tested during this study; long-term weathering potential also was not investigated.

Filters

Geotextile Filter. Selection of geotextile for filter fabric was made using the method outlined in *Designing with Geosynthetics* (Koerner 1998). The method establishes a maximum allowable aperture size and minimum allowable permeability to achieve compatibility with the bed material. According to this method, the geotextile for this application should exhibit a permeability that is more than four times greater than that of the bed material, i.e., $K_g/K_s > 4.0$. For particle retention, the effective aperture size of a geotextile filter must be less than the d_{90} of the bed material (approximately 2.0 mm) in this application. This method places more emphasis on permeability and less emphasis on particle retention compared to other procedures, such as HEC-11 or AASHTO M 288 (Lagasse et al. 2006). Table 3.2 summarizes the hydraulic and physical properties of the geotextile filters used in this study.

The areal extent of filter placement around the pier was identified as a parameter to be investigated under this testing program. For geotextile filters, both full and two-thirds coverage were examined. The term “full coverage” indicates that the geotextile extended beneath the riprap all the way to the periphery of the installation, whereas “two-thirds” indicates that the geotextile extended only two-thirds of the distance from the pier face to the periphery of the riprap. The two-thirds geotextile coverage corresponds to recommendations developed in NCHRP Project 24-07 (Parker et al. 1998) and confirmed in this study.

Granular Filter. Granular filter requirements were developed using the criteria specified in HEC-11. The initial

step establishes the compatibility of the filter with the sand bed material in terms of both particle retention and permeability by defining upper and lower limits of d_{15} for the filter. This determines the largest size allowable to maintain particle retention and smallest size allowable to ensure the filter has greater permeability than the sand. The upper limit compatibility criteria of the filter must be large enough so that the filter does not pass through the riprap (see the Reference Document for computation details).

The material selected for use was a nominal 10-mm (3/8-in.) crushed rock from a local source. A grain size distribution graph for the granular filter layer is presented in Figure 3.9. Grain size distribution curves for the riprap stone, and the bed sand are included for comparison.

Figure 3.10 shows the woven geotextile “W1” as well as the granular filter used in the testing program. Sand bed material and the 30-mm (1.2-in.) riprap are also shown in the photograph for comparison.

3.4.2 Testing Program

The testing program addressed stability and performance issues associated with the extent of riprap placement around the pier, and the termination details at the pier and around the periphery of the installation. In addition, various filter types and extents were investigated by varying this aspect for selected test runs. Two 8-in. (200-mm) square piers (with no skew) and a 2-in. by 10-in. (51-mm by 254-mm) rectangular (wall) pier with 0°, 15°, and 30° skew angles were tested. (See Appendix H for details of the configurations tested.)

Baseline

Baseline riprap installation conditions were based on current HEC-23 layout guidelines, where the riprap is extended a minimum of two pier widths in all directions and thickness of the riprap layer is a minimum of three times the d_{50} of the armor stone. The NCHRP Project 24-07 recommendation to extend the geotextile from the pier face to two-thirds of the distance to the periphery of the riprap was adopted for baseline runs. Figure 3.11 presents the design layout for the baseline riprap tests.

Table 3.2. Hydraulic and physical characteristics of geotextile filters.

Filter Name	Geotextile Type	Mass/ Unit Area	Apparent Opening Size (AOS)	Permeability	Trade Name	Manufacturer	K_g/K_s
W1	Woven	205 g/m ²	0.850 mm	0.20 cm/s	Geotex® 117F	SI Geosolutions (Propex)	5.0
NW1	Non-woven	163 g/m ²	0.212 mm	0.21 cm/s	Mirafi® 140 N	Mirafi Construction Products	5.25
NW2	Non-woven	250 g/m ²	~ 0.10 mm	0.4 cm/s	HaTe® B 250 K4	Huesker Synthetic GmbH	10.0
NW3	Non-woven	278 g/m ²	0.18 mm	0.21 cm/s	Mirafi® 180 N	Mirafi Construction Products	5.25

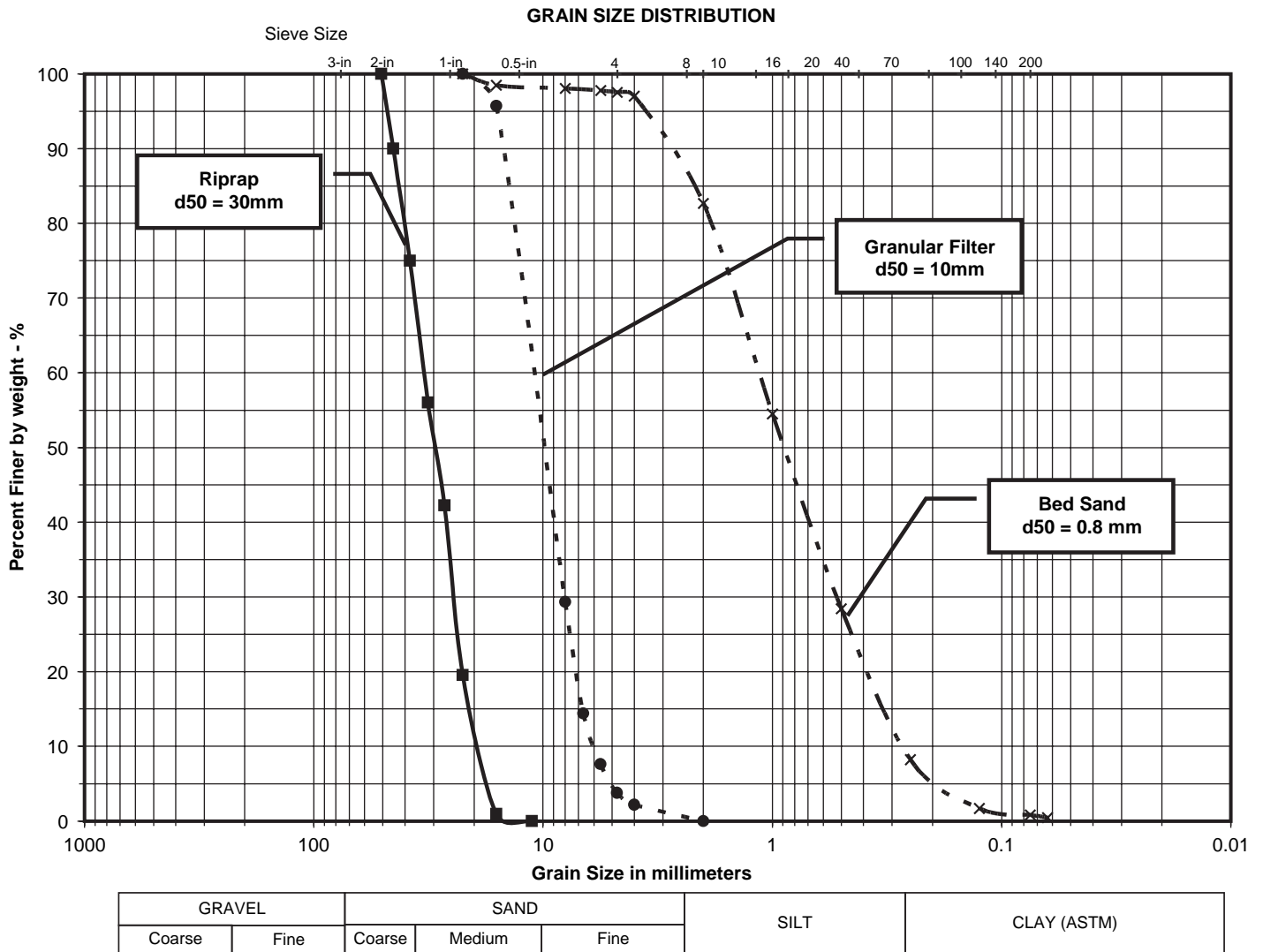


Figure 3.9. Granular filter, riprap stone, and bed sand grain size distributions.

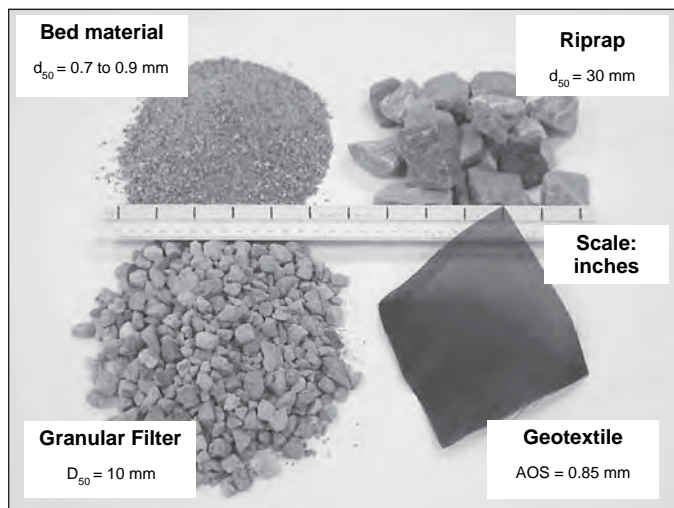


Figure 3.10. Various materials used in the riprap testing program.

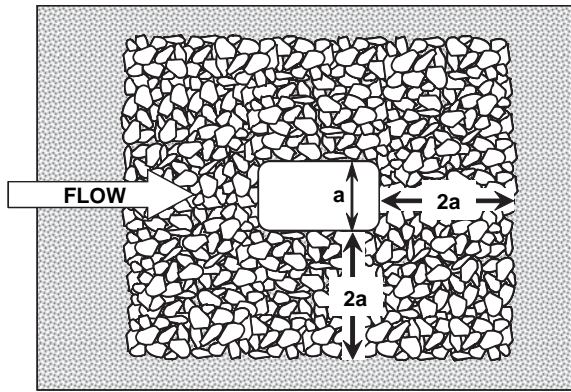
The design intent for the riprap baseline tests included examination of the following:

- HEC-23 guidelines with recommended geotextile, no skew
- HEC-23 guidelines with recommended geotextile, pier skewed to flow

Representative results of the baseline conditions tests are shown in Figure 3.12.

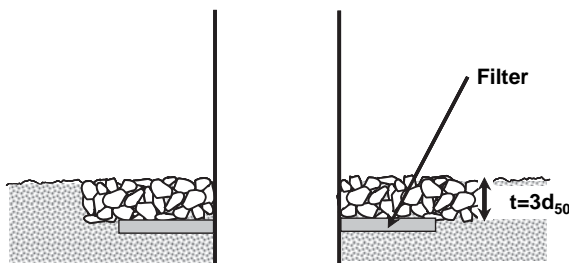
Extent of Coverage

Typically, riprap used for pier scour protection is placed on the surface of the channel bed, in a pre-existing scour hole, or in a hole excavated around the pier (Figure 2.2). The FHWA recommends placing the top of the riprap layer flush with the channel bed for inspection purposes (Lagasse et al. 2001, Richardson and Davis 1995).



Pier width = "a" (normal to flow)

Riprap placement = $2(a)$ from pier (all around)



Riprap thickness = $3d_{50}$ (minimum)

Filter placement = $4/3(a)$ from pier (all around)

Figure 3.11. Baseline riprap design.

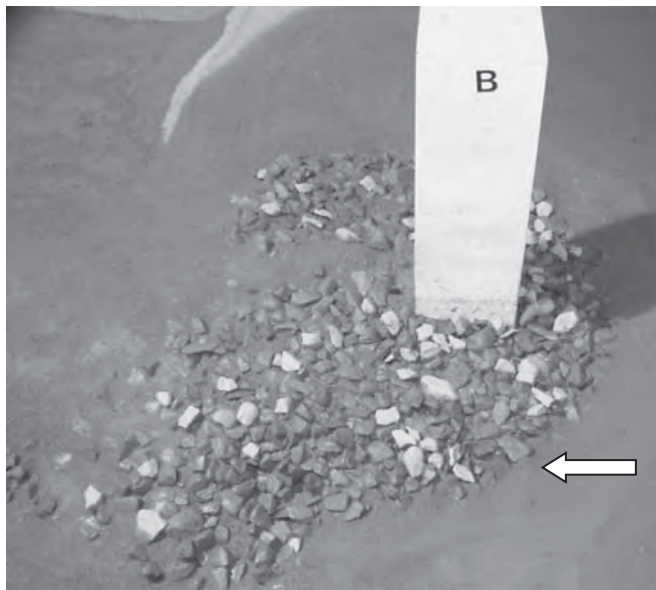
The design intent for the riprap coverage tests included examination of the following:

- Areal riprap coverage and edge treatment with recommended geotextile
- Areal riprap coverage variation from HEC-23 with recommended geotextile
- Areal riprap coverage and thickness variation from HEC-23 with recommended geotextile
- Scour hole extent with recommended geotextile
- Scour hole extent without filter
- HEC-18 guidelines
- Thickness and filter variation from HEC-23 guidelines
- Mounded riprap without filter

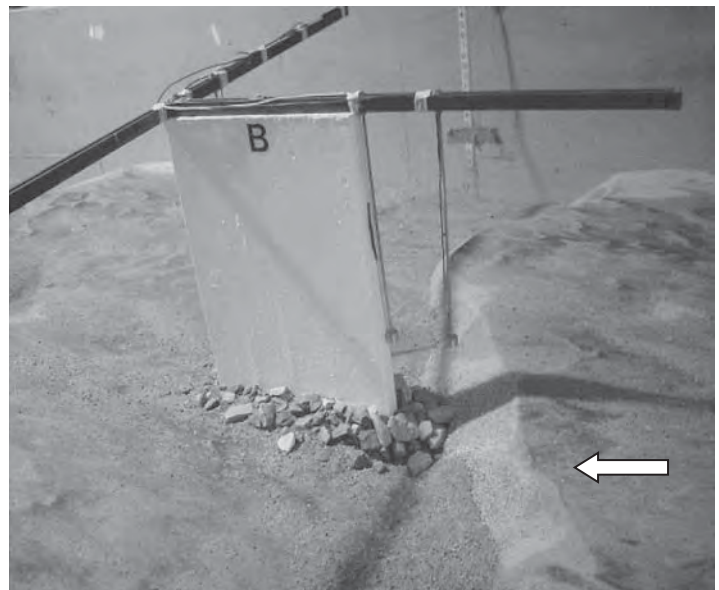
Appendix H, Table H.2, provides details on the filter alternatives tested, which included geotextile and granular filters with two-thirds and full coverage and, in some cases, no filter.

Test results indicated that best performance was achieved when riprap extended at least two times the width of the pier (as measured perpendicular to the approach flow on all sides) in a flat pre-excavated hole with the top surface flush with the bed. Figure 3.13 shows the poor performance when the areal coverage was reduced to less than two pier widths on all sides.

Riprap used for pier protection is often placed on the surface of the channel bed because of the ease and lower cost of placement and because it is more easily inspected. Test results indicated that, when the stable baseline riprap configuration was mounded on the surface without a filter, performance was poor. None of the mounded riprap in tests performed as well as the riprap in tests where it was level with the bed, given the

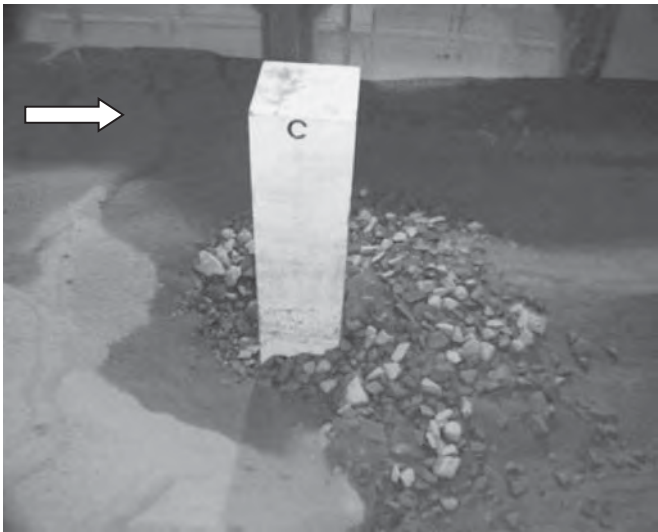


a. Baseline riprap installation after $2.5V_{crit}$ live-bed test at square pier.

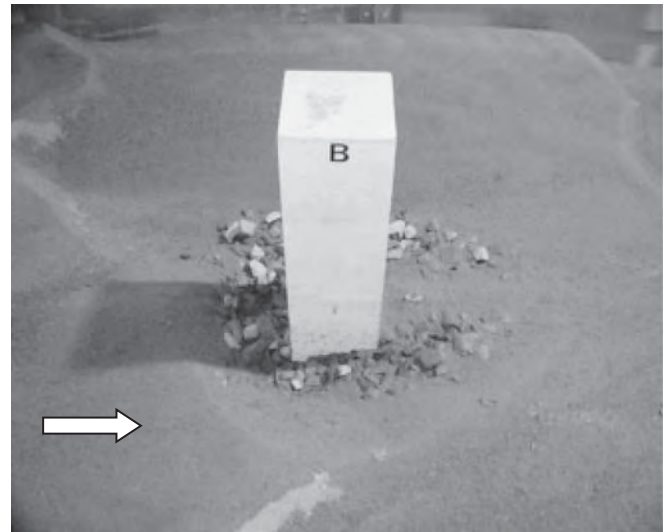


b. Baseline riprap installation after $2.0V_{crit}$ test at rectangular pier (no skew).

Figure 3.12. After baseline riprap installation testing.



a. Areal extent decreased to 4a after $2.5V_{crit}$ live-bed test. Note scour hole at nose of pier.



b. Areal extent decreased to 4a, thickness increased to $4d_{50}$ after $2.5V_{crit}$ live-bed test.

Figure 3.13. Decreased areal coverage riprap tests.

same areal extent of riprap coverage. Figure 3.14 shows the results of a mounded riprap test.

Numerous riprap studies (see Lagasse et al. 2006) suggest that thickness of the riprap layer placed around the bridge piers should be between two to three times median stone size ($2d_{50}$ to $3d_{50}$) of the riprap. Testing results indicate that $3d_{50}$ is appropriate for specifying minimum thickness and that performance improved with increasing riprap layer thickness.

Termination Detail

The design intent for the riprap termination detail tests included examination of the following:

- Areal coverage and edge treatment with recommended geotextile (two-thirds coverage)
- HEC-18 guidelines with geotextile filter (full coverage)

Four tests were performed to test conditions where the bottom of the riprap layer was not horizontal but instead sloped away from the pier while the surface of the riprap remained flush with the bed. Areal extent was decreased from the recommended 2a from the pier face in the installations, and thickness increased with distance from the pier. Particle launching and loss were observed after completion of several of the tests. Figure 3.15 shows the construction details for one of the termination tests. The results of two of the termination runs after $2V_{crit}$ tests are shown in Figure 3.16.

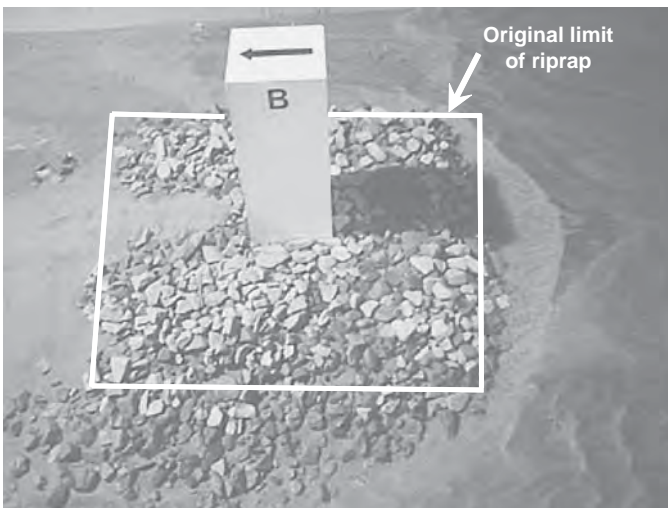


Figure 3.14. Mounded riprap after $2.0V_{crit}$ test.

Filter

NCHRP Project 24-07 (Parker et al. 1998) determined that placing a geotextile under a riprap layer with the same areal coverage as the riprap layer resulted in a relatively poor performance of the riprap. As a result of the effects of live-bed conditions, the rock at the edges tended to slide or be plucked off, exposing the underlying geotextile and ultimately resulting in failure of the riprap layer as successive bed forms pass and pluck more stones from the riprap layer. Parker et al. suggest extending the geotextile from the pier to about two-thirds of the way to the periphery of the riprap would result in better performance. Additional test results for this study confirmed that riprap performance was best when a geotextile filter extended two-thirds the distance to the periphery of the riprap.

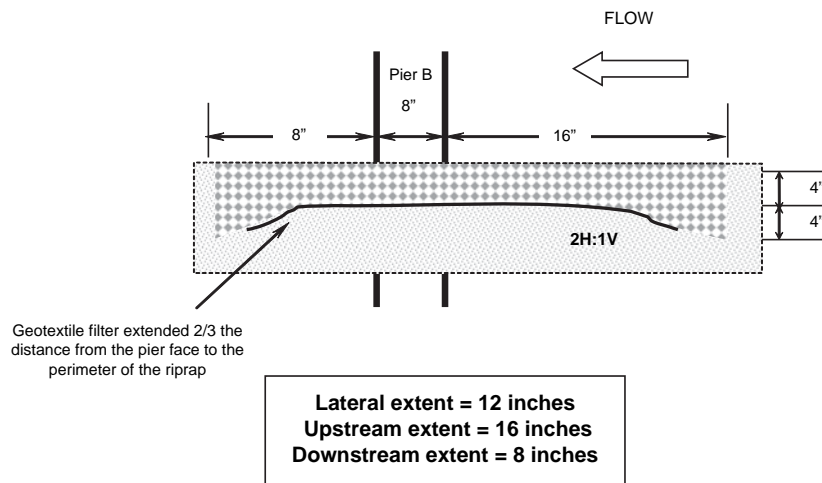


Figure 3.15. Example riprap termination test configuration.

The design intent for the riprap filter tests included examination of the following:

- Thickness and filter variation from HEC-23 guidelines
- HEC-23 guidelines and filter type variations
- HEC-18 guidelines
- Current practice and guidelines
- Thickness variation from HEC-23 guidelines
- Mounded riprap

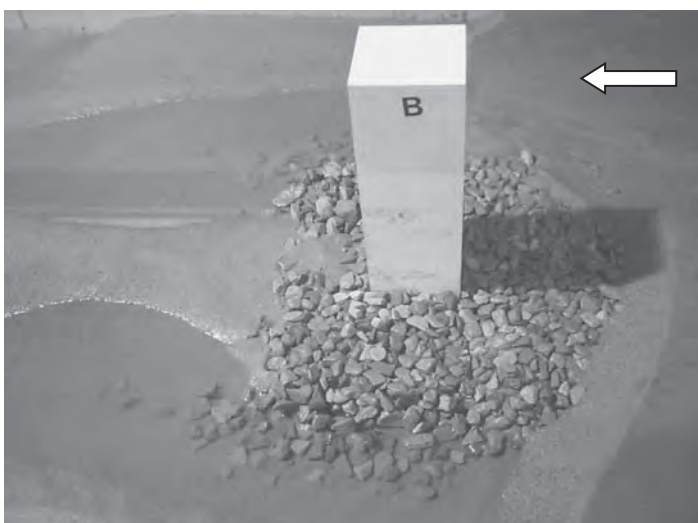
Appendix H, Table H.4, provides details on the filter alternatives tested which included geotextile and granular filters with two-thirds and full coverage and, in some cases, no filter.

Granular filters were found to perform poorly where bed forms are present. Specifically, where dune troughs that are

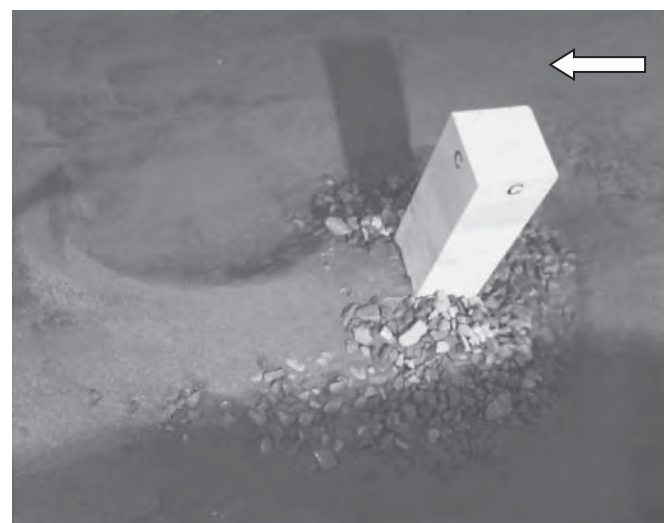
deeper than the riprap armor pass the pier, the underlying finer particles of a granular filter are rapidly swept away. The result is that the entire installation became progressively destabilized beginning at the periphery and working toward the pier. Figure 3.17 shows two piers after testing: one pier had a geotextile filter that extended two-thirds the distance from the pier face to the periphery (Figure 3.17a) and the other pier had a granular filter that extended the full distance from the pier face to the periphery of the riprap (Figure 3.17b).

3.5 Partially Grouted Riprap

Partial grouting of riprap with a cement slurry is presented as one of several standard design approaches for permeable revetments in a discussion of considerations regarding the ex-

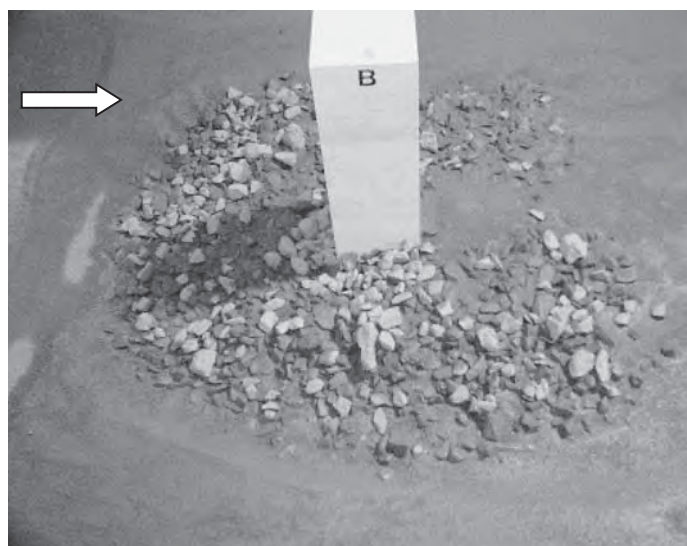


a. Riprap configuration from Figure 3.15 after $2.0V_{crit}$ test.

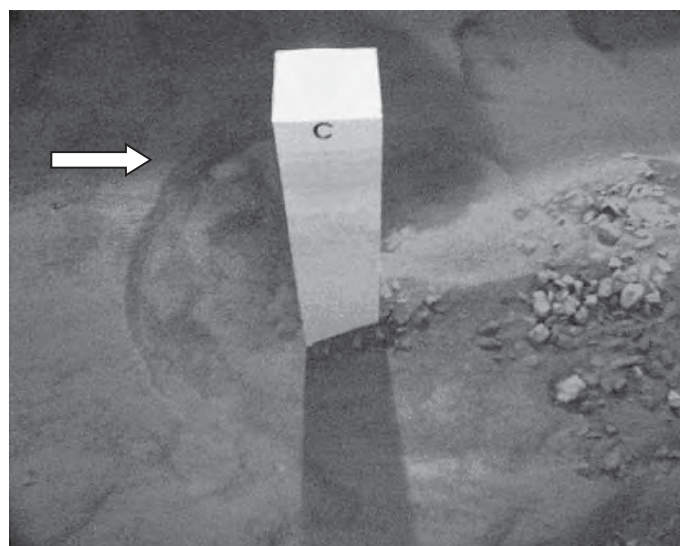


b. Increased depth of riprap at perimeter with insufficient areal extent after $2.0V_{crit}$ test.

Figure 3.16. Riprap termination tests.



a. Test 5d, riprap with two-thirds extent geotextile filter.



b. Test 5d, riprap with full extent granular filter. Note displacement of riprap.

Figure 3.17. Testing of granular and geotextile filters.

perience and design of German inland waterways (BAW 1990). As with standard (loose) riprap, when partially grouted riprap is properly designed and installed for erosion protection, it has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. The grout is placed on the riprap leaving significant voids in the riprap matrix and considerable open space on the surface.

For bridge pier protection, partially grouted riprap consists of rocks that are placed around a pier and grouted together with grout filling 50% or less of the total void space. The holes in the matrix allow for drainage of pore water; therefore, a filter is required. The grout forms conglomerates of riprap so the stability against particle erosion is greatly improved and a smaller thickness of stone can be used (Lagasse et al. 2001).

3.5.1 Materials

Grout

For the indoor partially grouted riprap installations, various Portland cement grout mix designs were developed and tested in the dry using d_{50} riprap sizes of 0.58 in. (14.7 mm), 1 in. (25.4 mm), and 1.2 in. (30 mm). Consistency of the cementitious grout mix was determined by trial and error. The initial mix design was based on pumpable fine aggregate concrete mix used in the construction of grout-filled mattresses. Test pours were performed for all three riprap sizes and a grout mixture was chosen based on flowability.

For testing installations, riprap was installed around a pier and then a measured volume of grout was hand poured into

“spots” on the riprap in a stagger pattern. The target fill value of between 15% and 40% of the original void space volume was maintained for all installations tested. Conglomerate-like elements in the riprap were produced using the spot-by-spot grouting procedure. Figure 3.18 shows the conglomerates produced during a test pour.

Filter

Only geotextile filters were tested with partially grouted riprap. Geotextile selection for filter fabric was made using the method outlined in Koerner (1998), as summarized in Section 3.4.1. Table 3.3 summarizes the hydraulic and physical properties of the geotextile filter used in the partially grouted riprap portion of this study.

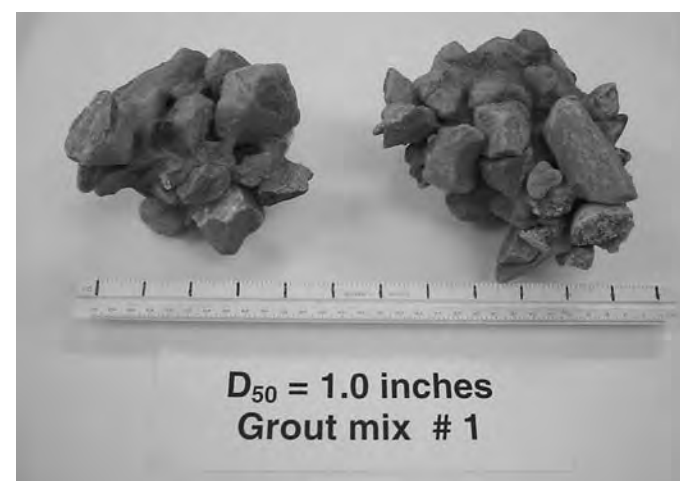


Figure 3.18. Conglomerates produced by partial grouting of riprap.

Table 3.3. Hydraulic and physical characteristics of geotextile filter used for partially grouted riprap tests.

Filter Name	Geotextile Type	Mass/ Unit Area	AOS	Permeability	Trade Name	Manufacturer	K_g/K_s
NW2	Nonwoven	250 g/m ²	~ 0.10 mm	0.4 cm/s	HaTe® B 250 K4	Huesker Synthetic GmbH	10.0

3.5.2 Small-Scale Testing Program

The partially grouted riprap testing program in the indoor flume addressed stone size, stability, and performance issues associated with the extent of partially grouted riprap placement around an 8-in. (200-mm) square pier, and the termination details at the pier and around the periphery of the installation (see Appendix H for details on the configurations tested).

Baseline

Because limited information is available on the use of partially grouted riprap as a pier scour countermeasure, baseline partially grouted riprap installation conditions were based on observations from the riprap testing program (Section 3.4). All baseline partially grouted riprap tests incorporated the same design layout with variation in stone size (0.6, 1.0, and 1.2 in. d_{50}) between the tests. Initial installation of partially grouted riprap extended a horizontal distance of one and a half pier widths on all sides for a total areal coverage of four pier widths; the geotextile filter extended two-thirds the coverage of the countermeasure, and thickness was $3d_{50}$ of the largest stone examined. Figure 3.19 presents the design layout for the baseline partially grouted riprap tests. The design intent for partially grouted riprap baseline tests was to examine rock size performance with a two-thirds extent geotextile filter.

Initial partially grouted riprap test results indicated that armor stone size could be reduced from the design riprap size for standard (loose) riprap without compromising stability when exposed to $2V_{crit}$ flow conditions with 4- to 6-in. (100- to 152-mm) dunes. The partially grouted smaller stones produced the desired conglomerates.

Extent of Coverage

The design intent of the extent of coverage tests for partially grouted riprap included examination of the following:

- Layer thickness and termination detail
- Areal coverage and thickness
- Areal coverage, layer thickness, and termination detail

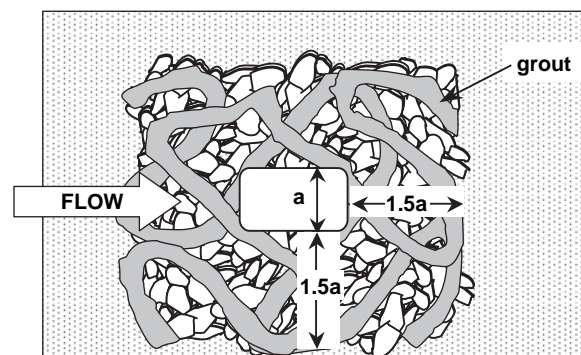
All tests were performed with a two-thirds extent geotextile filter.

When thickness was increased from the baseline installation design and grout quantity remained constant, undermining on the sides and downstream of the countermeasure was observed. Grout quantity was reduced by a third and the desired flexibility of the countermeasure was achieved. With the grout quantity reduced, stable conditions resulted for all partially grouted riprap tests.

Termination Detail

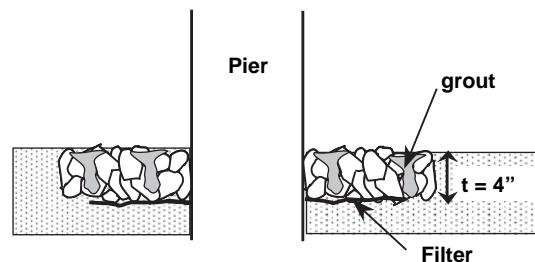
Four tests were performed to test conditions where the partially grouted riprap layer was not horizontal but sloped away from the pier. All tests were performed with a two-thirds extent geotextile filter. The design intent for these tests included examination of the following:

- Layer thickness and termination detail
- Areal coverage, layer thickness, and termination detail



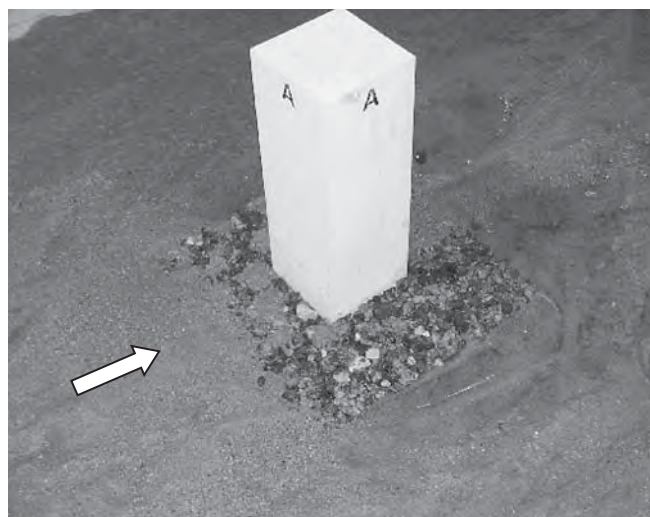
Pier width = "a" (normal to flow)

Extend partially grouted riprap a distance of 1.5(a) from pier (minimum, all around)

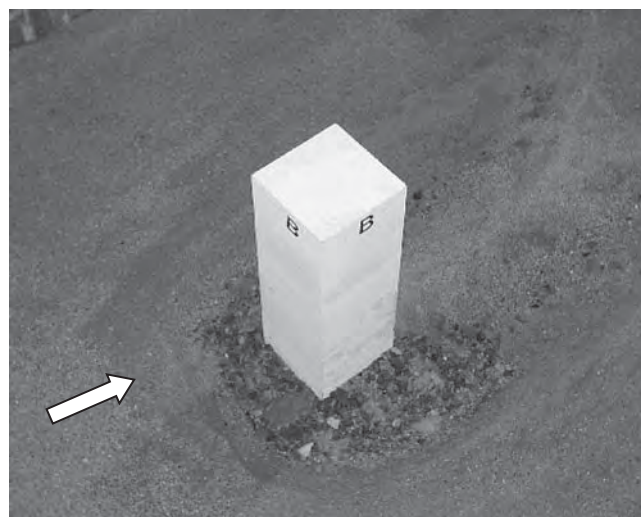


Filter placement = 1.0(a) from pier (all around)

Figure 3.19. Partially grouted riprap baseline design layout.



a. Partially grouted riprap installation with no turndown detail after $2.0V_{crit}$ live-bed ($d_{50} = 0.6$ in.).



b. Partially grouted riprap installation with a 4H:1V turndown after $2.0V_{crit}$ test ($d_{50} = 0.6$ in.).

Figure 3.20. Partially grouted riprap termination detail test results.

Good performance was observed for all termination tests; some particle launching was observed when the areal extent was decreased. The countermeasure was stable when exposed to live-bed $2.5V_{crit}$ flow conditions when the areal extent and thickness were increased from baseline conditions and a turndown to the depth of passing dune troughs was added. The results of two of the termination tests at $2V_{crit}$ are shown in Figure 3.20.

Filter

Results from the riprap portion of the testing program (Section 3.4) confirmed that a filter should not be extended fully beneath a riprap layer; instead, it should be terminated two-thirds the distance from the pier face to the edge of the riprap. All partially grouted riprap tests incorporated this partial coverage filter recommendation (Figure 3.19).

3.5.3 Prototype-Scale Tests of Partially Grouted Riprap

Tarbela Flume

The Tarbela flume at CSU measures 108 ft (33 m) long by 20 ft (6 m) wide and 8 ft (2.4 m) deep. Flow enters the flume by a 36-in. (900-mm) diameter pipe fed by a nearby reservoir. Flow enters the headbox and is discharged into the flume through a sluice gate with dimensions 6.25 ft (2 m) by 3.9 ft (1.2 m). A rock baffle 5.25 ft (1.6 m) tall and spanning the width of the flume was installed 15 ft (4.6 m) downstream of the headbox. The baffle was intended to uniformly distribute the flow across the width of the flume. Tail water depths were controlled by four sluice gates at the downstream end of the flume. Bed slope of the flume was 0.003 m/m (0.3%).

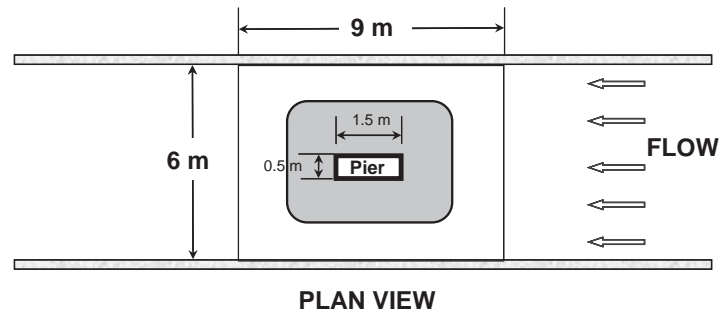
A test section was created 30 ft (9 m) downstream of the rock baffle. The test section was 30.7 ft (9 m) long and spanned the width of the flume. It was filled with sand level with the approach section. Upstream and downstream of the test section the flume bed consists of smooth concrete floors. A rectangular pier measuring 1.5 ft (0.5 m) by 4.5 ft (1.5 m) was installed in the center of the test section. Figure 3.21 is a layout diagram for the prototype partially grouted riprap testing program. Surrounding the pier, a scour hole measuring 12 ft by 16 ft (4m × 5 m) was pre-formed into the sand bed to a maximum depth of 3 ft (0.4 m) as shown in Figure 3.22.

Partially grouted riprap was tested at prototype scale to demonstrate constructability and to examine water quality issues during installation and performance in high-velocity flow conditions. In addition, partially grouted riprap was compared side by side to loose riprap under high-velocity flow conditions.

Materials

Geocontainers. Sand-filled geotextile containers were constructed using a geotextile fabric with the characteristics presented in Table 3.4. The geotextile containers measured 4 ft by 1.5 ft by 0.33 ft (1.2 m by 0.5 m by 0.1 m) with a typical volume of 2 ft³ (0.6 m³). Approximately 220 lbs (100 kg) of sand were placed in each container. Commercial concrete sand meeting appropriate filter criteria was used to fill the geotextile containers. Figure 3.23 shows the geotextile containers before being placed around the pier.

Bed Material. Commercial concrete sand with a d_{50} of approximately 0.7 mm was used for the sand bed (see Figure 3.4 for a graph of grain size distribution of the bed material).



PARTIALLY GROUDED RIPRAP:

$d_{50} = 15 \text{ cm}$
 Thickness of layer = 45 cm
 Area covered = 19.8 m²

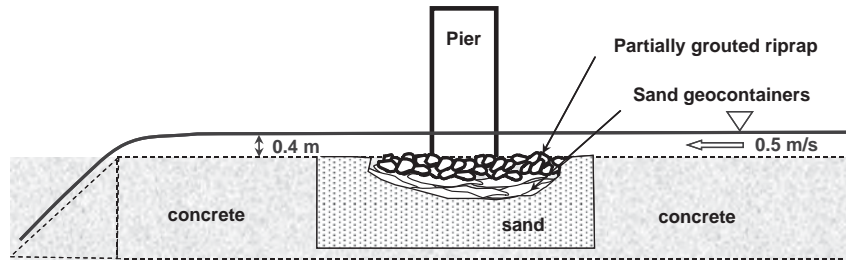


Figure 3.21. Schematic layout for prototype partially grouted riprap tests (dimensions approximate).

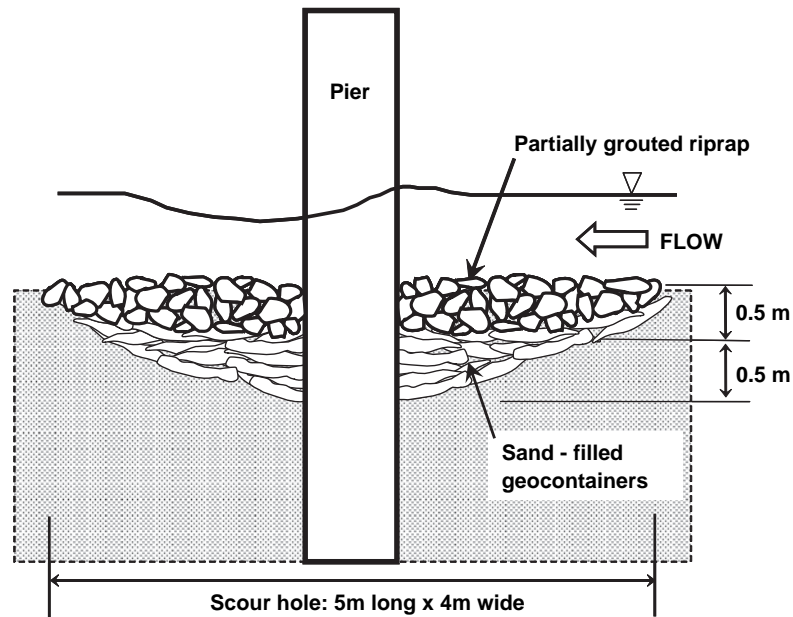


Figure 3.22. Tarbela installation.

Table 3.4. Characteristics of geotextile.

Trade Name	Mass/ Unit Area	AOS	Permeability	Geotextile Type	K_g/K_s
Mirafi® 180 N	278 g/m ²	0.18 mm	0.21 cm/s	Non-woven needle punched	5.25

Armor Stone. Durable sandstone riprap for testing had a d_{50} of 6 in. (152 mm). Figure 3.24 shows the grain size distribution of riprap used in the prototype portion of the testing program.

Grout. A grout mixture created for underwater application was used in the testing program. A proprietary admixture was included in the grout to prevent dilution and dissipation of the grout into the water. Table 3.5 presents the approximate grout component quantities.

Grout was mixed at a commercial batch plant. During the mixing process, water was added to the mixture in order to achieve the desired consistency and slump characteristics. Figure 3.25 shows the grain size distribution curve for the coarse aggregate in the grout mix.



Figure 3.23. Geocontainers before installation around the pier.

Testing Program

The design intent for the prototype-scale partially grouted riprap tests included examination of the following:

- Constructability
- Environmental issues

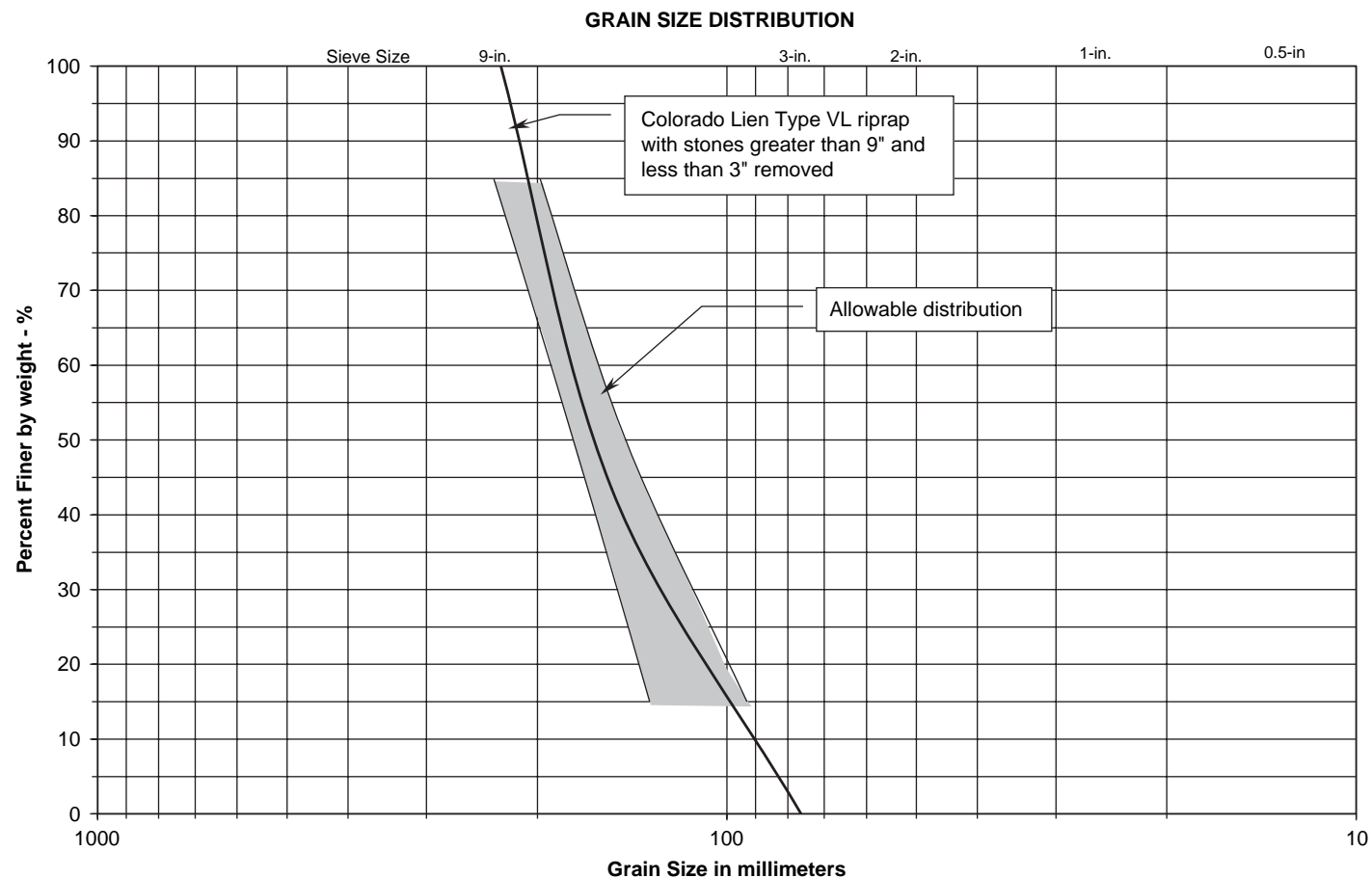


Figure 3.24. Six-inch riprap grain size distribution.

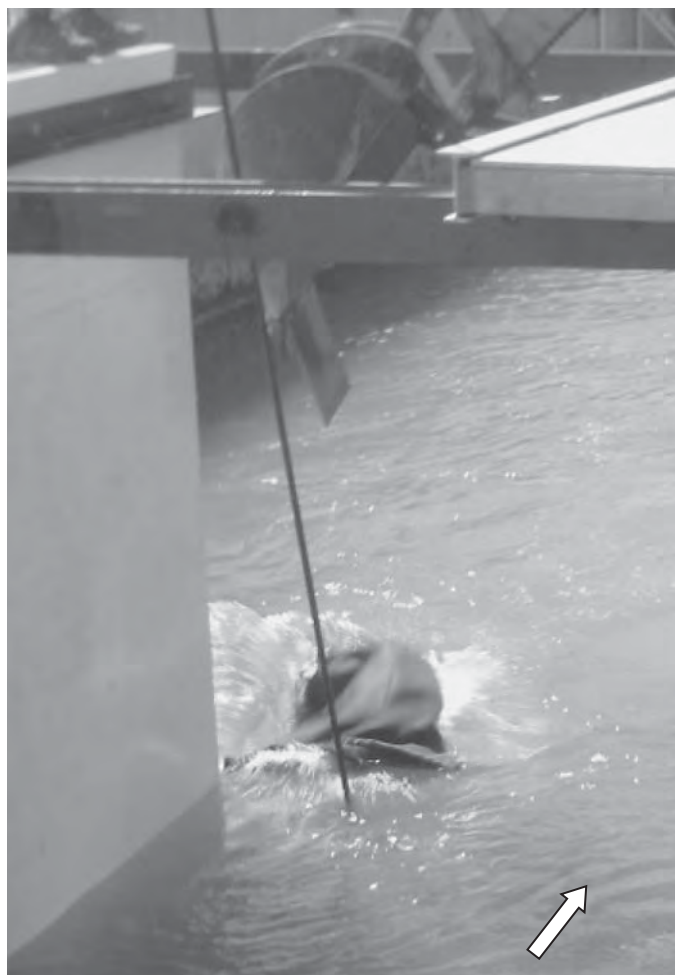


Figure 3.26. Installation of geotextile containers (pier is on the left).

Figure 3.31 shows the preliminary trial grout application in the dry. Figure 3.32 shows the surface of the riprap after partial grouting, and Figure 3.33 shows the interior of the dry riprap pile after several exterior stones had been removed to display penetration of the grout. Note in Figures 3.32 and 3.33

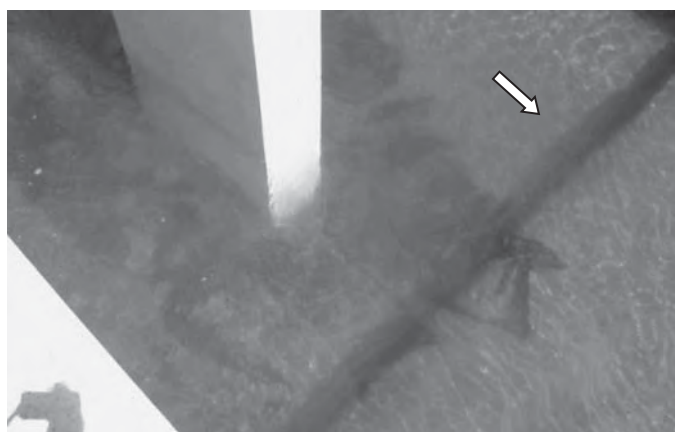


Figure 3.27. Geotextile containers after installation.



Figure 3.28. Installation of riprap around pier.

how the grout bridges riprap stones forming larger conglomerate particles. In Figure 3.33, note that less than 50% of the total void space has been filled with grout. The preliminary application confirmed that the equipment planned for the underwater partial grout application was satisfactory.

Grout placement in the flume was performed by an experienced underwater grout installation specialist from Germany. The specialist was located in the flume and placed the grout directly on the riprap in 1 ft (0.305 m) of water with a velocity 1 ft/s (0.305 m/s), as illustrated in Figure 3.34.

Application of grout on the riprap lasted approximately 20 min. Approximately 1.4 yd³ (1.1 m³) of grout was placed, resulting in an application of 1.6 ft³/yd² (56 L/m²). Typical grout application rates in German practice are 60 L/m², so this test was representative of standard practice for this countermeasure type.

Water Quality Monitoring. Water quality was monitored before, during, and after the grout placement. Water quality parameters monitored continuously were pH, conductivity, temperature, and turbidity. Based on research performed by the Virginia DOT (VDOT), pH is the only water quality parameter that is expected to change significantly dur-



Figure 3.29. Riprap prior to grouting.

ing grout placement (Fitch 2003). In the VDOT study, permit conditions required that pH levels remain below a value of 9.0, otherwise grouting activities were to be stopped, and mitigation measures such as silt curtains were to be employed. VDOT did not monitor turbidity during their study.



Figure 3.30. Concrete mixer truck and pump truck with boom.



Figure 3.31. Preliminary trial grout application in the dry.

Water quality monitors, placed in stream at the seven locations depicted in Figure 3.35, continually recorded measurements of pH, conductivity, turbidity, and temperature. Baseline conditions were established prior to initiation of the grout placement 12 ft (3.7 m) upstream of the pier along the centerline of the flume (Station A in Figure 3.35).

During the test, the water discharge was 20 cfs (0.6 m³/s) and the average rate of grout placement was 0.032 cfs (0.001 m³/s); therefore, the water:grout dilution ratio was 20:0.032, or 625:1. Three grab samples were selected for analysis: a baseline sample taken at Station A when testing commenced, a sample taken at Station C 5 minutes after grout application began, and a sample taken at Station F when grout application finished. Grab samples were collected in 250 mL polyethylene bottles that had been washed and rinsed with distilled water. Bottles were filled by dipping the bottle into the water upstream of where the sampling



Figure 3.32. Surface of the riprap after partial grouting.



Figure 3.33. Interior of the dry riprap pile (some surface rocks removed).

personnel were standing in the flume. The grab samples were analyzed for selected inorganics and metals. The laboratory results for the samples are presented in Table 3.6.

Continuous water quality data were calibrated to background data collected at Station A prior to grout placement. Results from the water quality monitoring program are presented in the following paragraphs.

pH. Background pH was 7.0 at all stations located in the flume itself. Downstream of the flume, Station J (located in

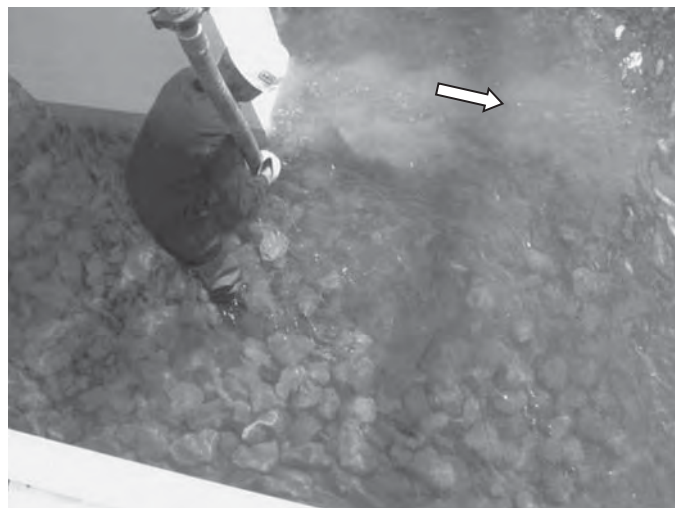
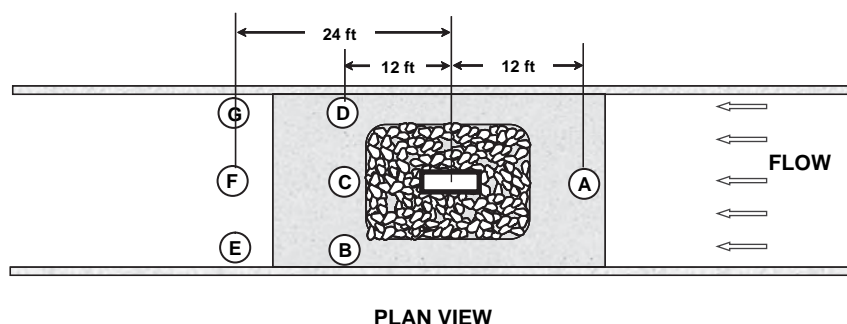


Figure 3.34. Underwater partial grouting of riprap.

the natural channel 150 ft (46 m) downstream of the flume tailgates) exhibited a background pH of 7.4.

A spike in pH was observed at the locations directly downstream of the pier during grout pumping. A maximum pH of 9.9 was recorded by the continuous monitor located 12 ft (3.7 m) directly downstream of the pier 3 minutes after pumping began. After grout pumping was completed, pH values dropped off quickly and typically returned to baseline conditions within 30 minutes. The one exception was the probe at Station C, which was directly in the wake of the pier and at the downstream edge of the grouted area. At this location, the pH returned to background levels after about 4 hours. Considering its location, this probe was in position to record the cumulative effect of the entire grouted area for the duration required for it to cure. At Station F, located 12 ft (3.7 m) directly downstream of Station C, a much less pronounced pH profile and more rapid decay of concentration was observed. Results of monitoring are presented in Table 3.7, and Figure 3.36 shows the pH measurements at all stations. Figure 3.37 shows the maximum pH values at any time during the test as a function of distance from the pier.



PLAN VIEW

Note: Stations H, I, and J are located further downstream and are not shown in this illustration.

Figure 3.35. Location of water quality monitoring stations.

Table 3.6. Detailed water quality analyses of selected grab samples.

		STATION A 10:14 am		STATION C 10:19 am		STATION F 10:34 am	
LABORATORY VALUES		mg/L	meq/L	mg/L	meq/L	mg/L	meq/L
Sodium	Na ⁺	2.78	0.12	3.06	0.13	2.94	0.13
Potassium	K ⁺	1.00	0.03	2.40	0.06	1.60	0.04
Calcium	Ca ²⁺	9.93	0.50	23.60	1.18	16.40	0.82
Magnesium	Mg ²⁺	1.77	0.15	1.80	0.15	1.77	0.15
Carbonate	CO ₃ ²⁻	0.00	0.00	44.00	1.47	14.00	0.47
Bicarbonate	HCO ₃ ⁻	32.00	0.52	23.00	0.38	34.00	0.56
Chloride	Cl ⁻	2.00	0.06	3.00	0.08	2.00	0.06
Sulfate	SO ₄ ²⁻	3.30	0.07	7.80	0.16	5.40	0.11
TDS (lab ROE)		< 25		< 25		< 25	
FIELD MEASUREMENTS							
Conductivity, µmhos/cm		83		142		101	
pH, Standard Units		7.1		10.0		9.4	
Turbidity, NTU		2.6		8.4		7.1	

Turbidity. Background turbidity was about 3 to 4 nephelometric turbidity units (NTUs). Turbidity peaked at 53.9 NTUs immediately after grout application began. This peak was maintained for less than 30 seconds, after which turbidity measurements ranged from about 30 to 35 NTUs for approximately 5 minutes. Turbidity returned to pregrouting levels almost immediately after grout application was completed. Results of monitoring are presented in Table 3.8, and Figure 3.38 is a plot of turbidity measurements. Note in Figure 3.38 an increase in turbidity can be seen prior to grout application, corresponding to personnel walking around the test section in preparation for grout application.

Temperature. Temperature remained nearly constant, ranging from 44.5°F to 44.7°F (6.9°C to 7.1°C) throughout the testing period, indicating the grout application process did not adversely affect water temperature. Results of monitoring

are presented in Table 3.9, and Figure 3.39 shows a plot of temperature measurements.

Conductivity. Background conductivity was 45 to 50 µmhos/cm prior to the test. Contrary to the findings of the VDOT study, conductivity values did appear to follow the pattern of grout installation. A notable increase in conductivity was observed at the two monitoring stations immediately downstream of the pier beginning at 10:17, 3 minutes after grouting application commenced. Results of monitoring are presented in Table 3.10, and Figure 3.40 is a plot of conductivity measurements.

After pH values returned to pre-grouting levels, as indicated by the grab sample monitoring, the tailwater control gates were shut and water was backed up in the flume. The installation remained submerged for 96 hours to allow the grout to cure. After 96 hours the tail gates were opened, the flume was drained, and the installation was allowed to dry.

Table 3.7. Summary of pH measurements.

Station	Initial Condition	End Condition	Maximum value	Average During Grout Placement
A	6.9	7.1	7.1	7.0
B	6.9	7.1	9.4	8.4
C	6.9	7.3	9.9	9.7
D	6.9	7.0	8.6	7.8
E	6.9	7.1	9.2	7.9
F	6.9	7.1	9.5	9.0
G	6.9	6.9	8.5	7.8
H	7.0	7.0	8.3	7.1
I	7.0	7.2	8.6	7.3
J	7.4	7.5	8.4	7.7

Note: Data at Stations A through G from continuous monitors
Data at Stations H through J from grab samples

High-Velocity Performance Test. Loose riprap around the surface perimeter of the installation that was not firmly secured during the grouting process was removed and replaced with sand. To prevent degradation of the sand bed during high-velocity testing, the upper 4 in. (100 mm) were stabilized by tilling 4% Portland cement by dry weight (of the sand) into the sand bed. The material was compressed with a vibrating plate compactor after addition of the Portland cement.

The high-velocity test ran for 2 hours and was terminated when the soil cement bed began to visibly fail. Approach velocities at 60% of depth during the high-velocity test ranged from 4.2 to 5.6 ft/s (1.3 to 1.7 m/s). After draining the flume, several scour holes were observed in the soil cement bed, and a significant scour hole was observed downstream of the riprap installation. The soil cement in these areas had been

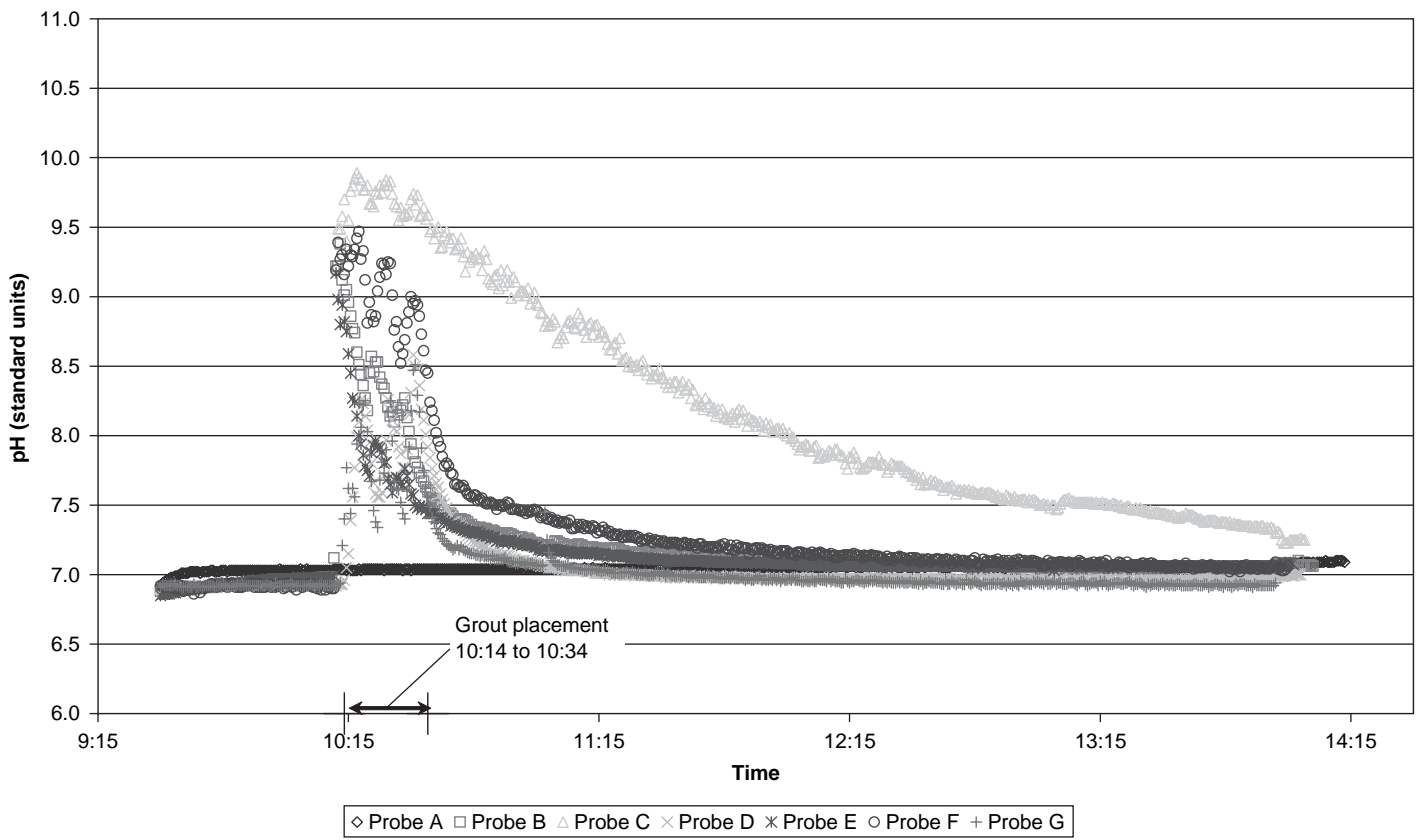


Figure 3.36. pH vs. time.

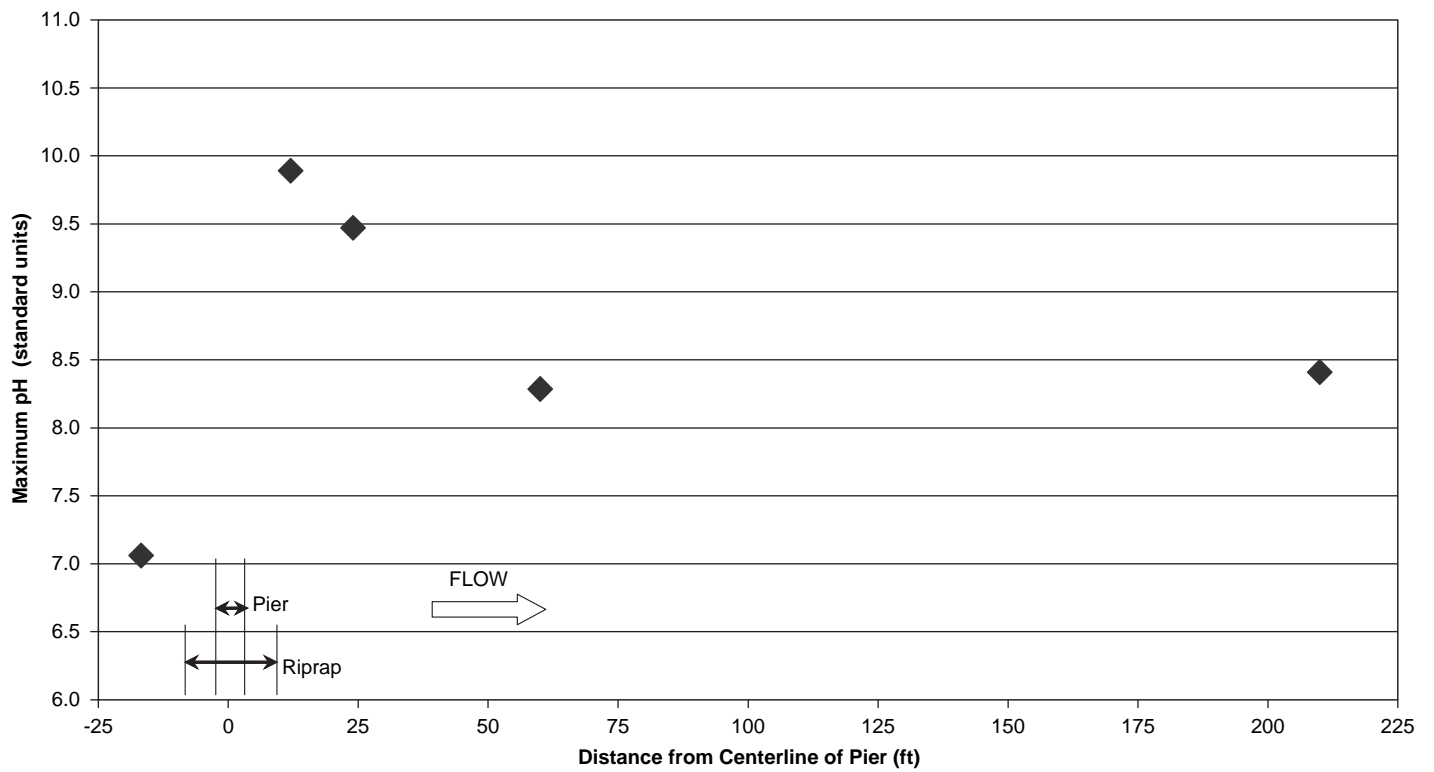


Figure 3.37. Maximum observed pH vs. distance from pier.

Table 3.8. Summary of turbidity measurements (NTUs).

Station	Initial Condition	End Condition	Maximum Value	Average During Grout Placement
A	3.6	3.3	8.3	3.7
B	3.6	3.6	27.9	7.1
C	4.0	3.7	51.3	22.7
D	3.8	3.4	20.6	7.1
E	3.1	2.9	19.1	6.1
F	7.1	4.0	53.9	19.5
G	3.7	3.7	9.1	4.9
H	3.2	2.6	3.2	3.3
I	2.5	2.6	3.0	2.7
J	3.3	3.4	4.6	3.6

Note: Data at Stations A through G from continuous monitors
Data at Stations H through J from grab samples

destabilized and the underlying sand scoured to a depth of about 2.5 ft (0.8 m). The partially grouted riprap installation and underlying geotextile containers remained intact. Figure 3.41 shows the test section after the high-velocity test.

High-Velocity Comparison Test. To facilitate a comparison of the performance of loose riprap to partially grouted riprap, all riprap and grout were removed from the

left side of the pier and replaced with loose riprap of the same gradation and d_{50} shown in Figure 3.24. Because the soil cement proved to be inadequate to stabilize the area around the partially grouted riprap, it was completely removed from the bed, exposing the underlying sand bed 4 in. (100 mm) lower than the surrounding flume floor and top surface of the riprap. A geotextile fabric, with the hydraulic and physical characteristics presented in Table 3.4, was installed over the exposed sand portion of the test section. Four-inch (100-mm) thick ACBs were installed on the geotextile fabric adjacent to the riprap. The ACBs were intended to prevent degradation of the bed in the test section as well as facilitate a smooth transition from the flume floor to the test section.

Temporary walls were installed to reduce the width of the flow area and increase velocity in the test section. Walls were installed 2.5 ft (0.76 m) from the existing flume walls, transitioning the section from 20 ft (6 m) to 15 ft (4.6 m). Figure 3.42 shows the test section after the modifications were completed.

The high-velocity comparison test ran for 4 hours, during which time the discharge was steadily increased to the full flow capacity. At maximum discharge, the approach velocity upstream of the pier reached a maximum of 6.4 ft/s (2 m/s). At the higher flows, the loose riprap began to displace. Figure 3.43

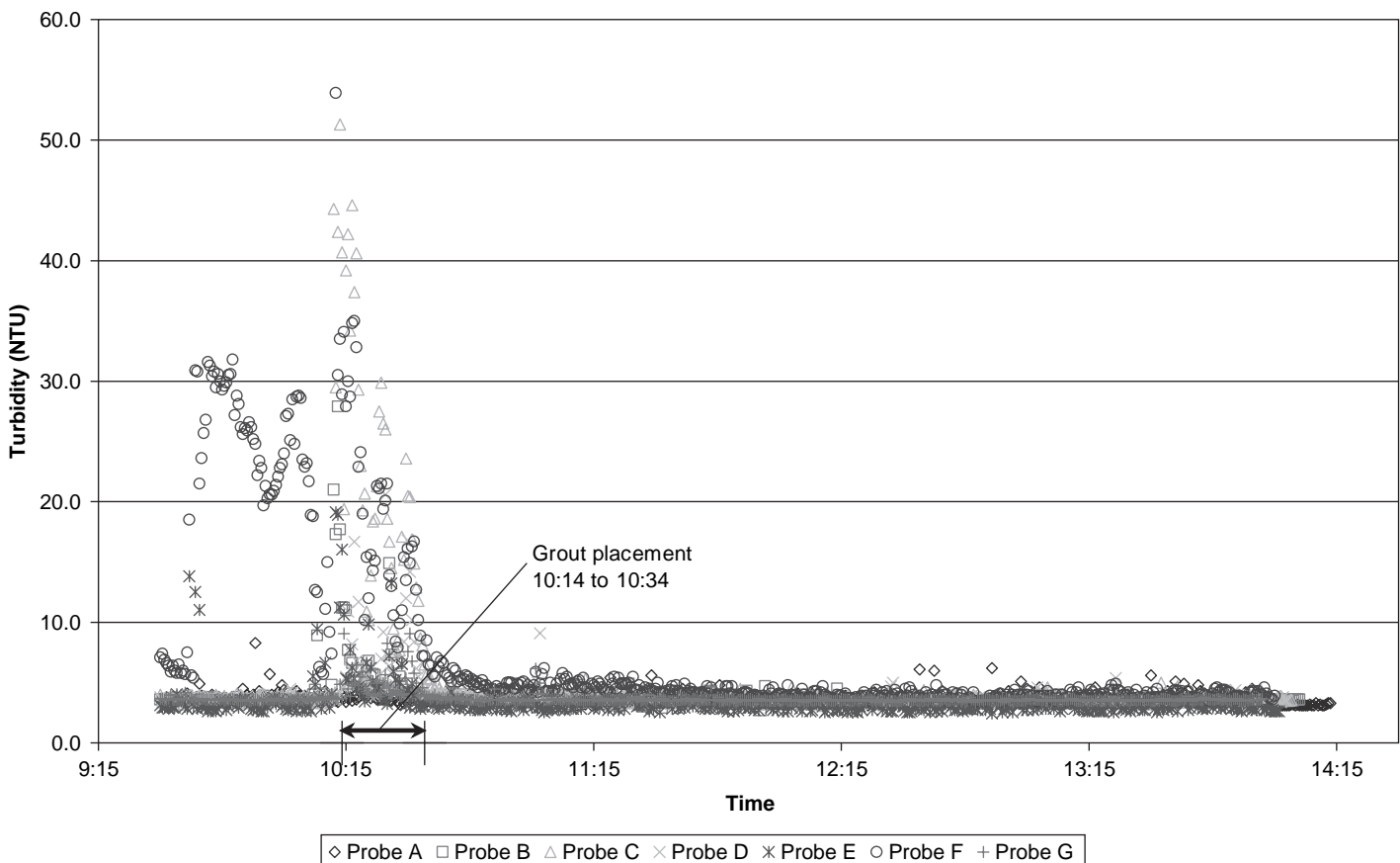
**Figure 3.38. Turbidity vs. time.**

Table 3.9. Summary of temperature measurements (°F).

Station	Initial Condition	End Condition	Maximum Value	Average During Grout Placement
A	44.5	44.5	44.7	44.6
B	44.5	44.5	44.7	44.5
C	44.5	44.5	44.7	44.6
D	44.5	44.5	44.7	44.5
E	44.5	44.5	44.7	44.5
F	44.5	44.5	44.7	44.6
G	44.5	44.5	44.7	44.5
H	44.5	44.5	44.7	44.6
I	44.5	44.5	44.7	44.5
J	44.5	44.5	44.7	44.6

Note: Data at Stations A through G from continuous monitors
No temperature data available at Stations H through J

shows the loose riprap side of the installation after completion of the second half of the high-velocity comparison test. Note the scour hole on the near side of the pier and the displaced riprap behind and downstream of the pier compared to the previous figure. The partially grouted side of the riprap installation can be seen in this figure, and remained essentially

undisturbed. Figure 3.44 shows the partially grouted side of the installation after the end of this test.

3.6 Articulating Concrete Block Systems

There is limited experience with the use of ACB systems as a scour countermeasure 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 ACB systems along bank lines and in channels are well documented (e.g., Ayres Associates 2001), but there are few published guidelines on the installation of these systems around bridge piers.

There are two failure mechanisms for ACB systems: overturning and rollup of the leading edge of the mat where it is not adequately anchored or toed in, and 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 fab-

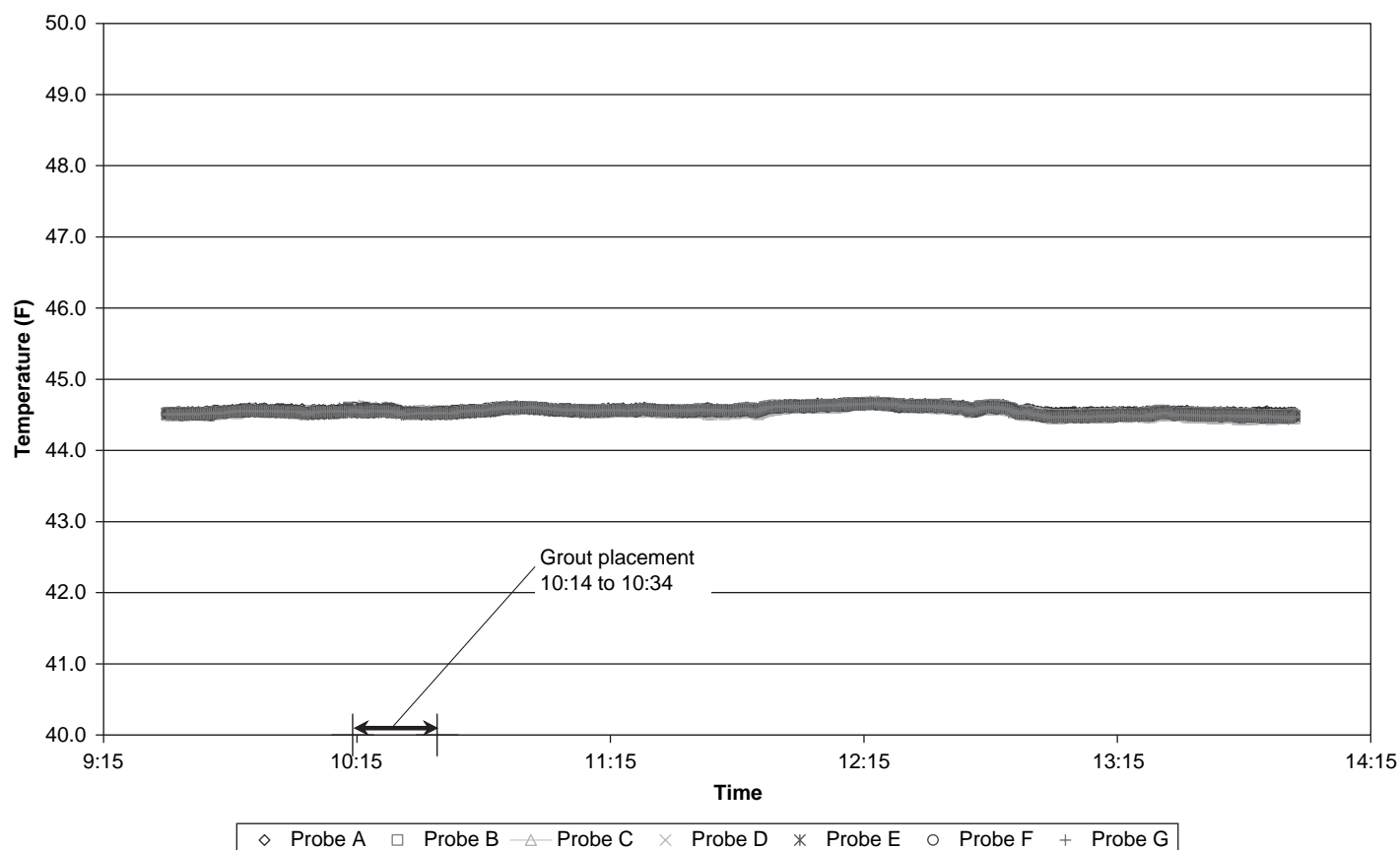


Figure 3.39. Temperature vs. time.

Table 3.10. Summary of conductivity measurements ($\mu\text{mhos/cm}$).

Station	Initial Condition	End Condition	Maximum Value	Average During Grout Placement
A	48	48	48	48
B	48	48	62	51
C	48	49	74	61
D	48	49	57	50
E	48	48	56	50
F	48	49	76	62
G	48	48	58	50
H	45	44	46	44
I	47	44	49	47
J	51	43	53	49

Note: Data at Stations A through G from continuous monitors
Data at Stations H through J from grab samples

ric or geotextile was important to the overall effectiveness and stability of the ACB system.

3.6.1 Materials

Blocks

ACBs were examined for their suitability as a pier scour countermeasure. Many ACB systems in use today are pre-

cabled and installed as mats. Cabling is primarily a construction convenience, and while cables may prevent blocks from being lost entirely, they do not keep blocks from failing through loss of intimate contact with the subgrade, which is the criterion generally accepted for stability design. The testing procedure for this ACB examination did not incorporate any simulation of cabling.

Miniature open-cell blocks measuring 1.6 in. (40 mm) long by 1.4 in. (36 mm) wide by 0.7 in. (17 mm) high were used in the testing program. The blocks were made from a sand-cement mortar having a specific gravity of 1.84 and a moisture absorption of 16% by weight. The critical shear stress for the blocks was determined in a smaller flume prior to placement around the test piers. The blocks were sized using the factor of safety method for hydraulic conditions representative of the CSU 8-ft flume. Results indicate that at a flow depth of 1.0 ft (0.305 m) and an approach velocity of $2V_{\text{crit}}$, a target factor of safety of 1.0 (incipient failure) was achieved under these conditions. Because the factor of safety method presented in HEC-23 (Lagasse et al. 2001) for ACB countermeasure design does not account for any added stability that may be afforded by cables, the testing procedure reflects HEC-23 philosophy. Table 3.11 provides a summary of the physical and hydraulic characteristics of the miniature

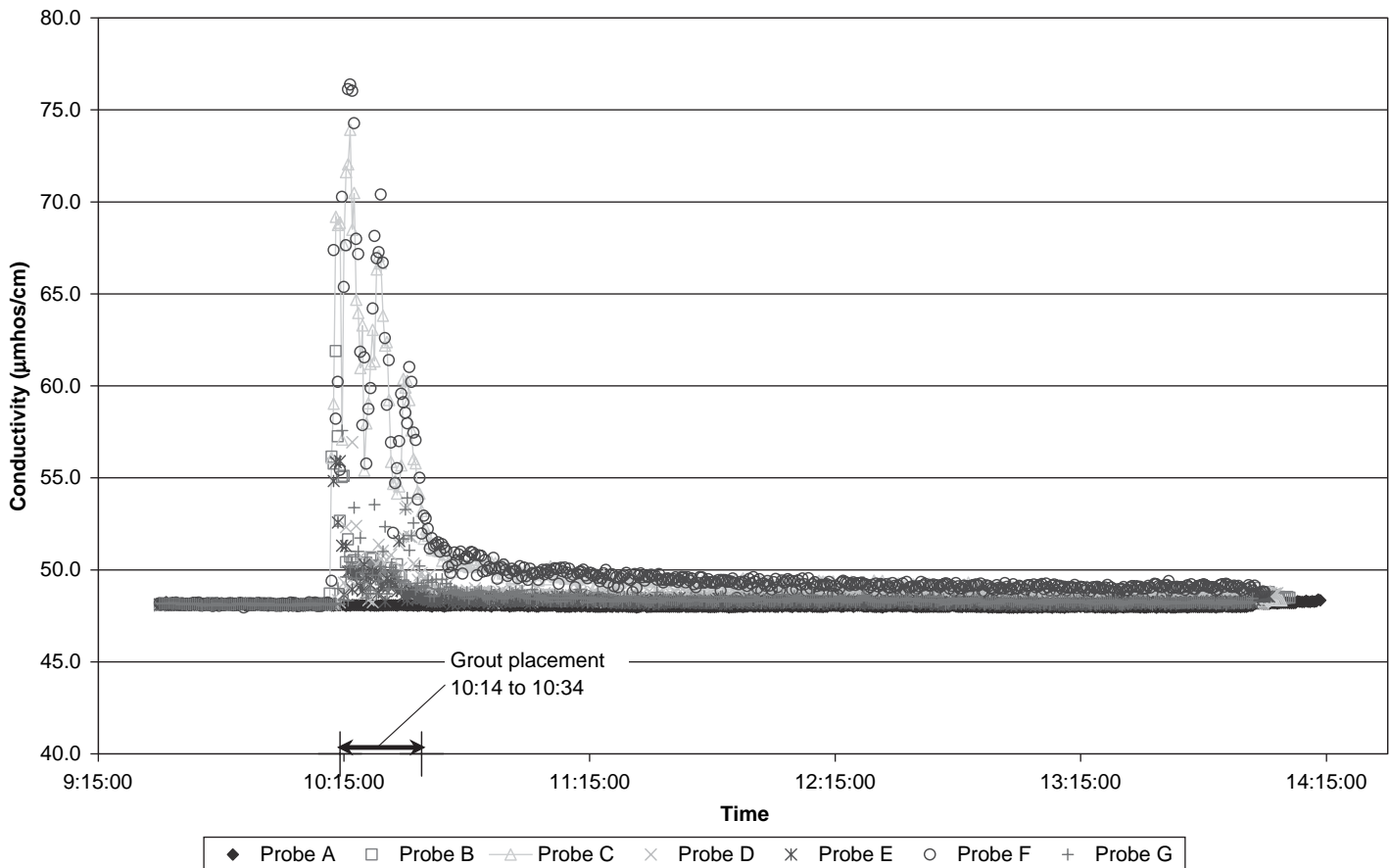


Figure 3.40. Conductivity vs. time.



Figure 3.41. Damage to the soil cement and scour at the downstream left corner after the high velocity performance test.



Note: Loose riprap is on the near side of the pier and partially grouted riprap on the far side

Figure 3.42. Loose riprap, ACB, and contraction wall installation.

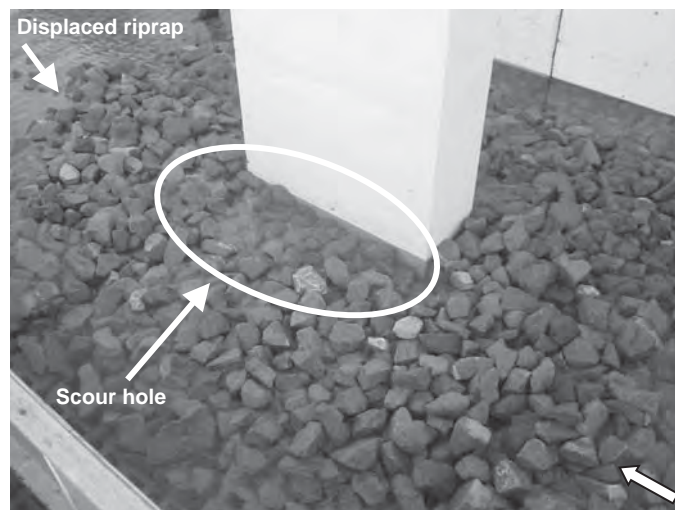


Figure 3.43. Loose riprap after completion of the high-velocity comparison test.

blocks used in this study. Figure 3.45 shows a close-up view of the blocks and their interlocking installation pattern.

The blocks were placed directly on a non-woven, needle-punched geotextile but were not glued or otherwise affixed to the geotextile, and no cables were used in any of the tests. In all cases, a sand-cement grout seal was placed between the blocks and the pier. For some tests, grout seams were also used at the intersection of plane surfaces where typical field applications would normally call for saw-cut blocks and grout seams to be used.

Filter

Geotextile selection for filter fabric was made using the method outlined in Koerner (1998), as summarized in Section 3.4.1. Table 3.12 summarizes the hydraulic and physical

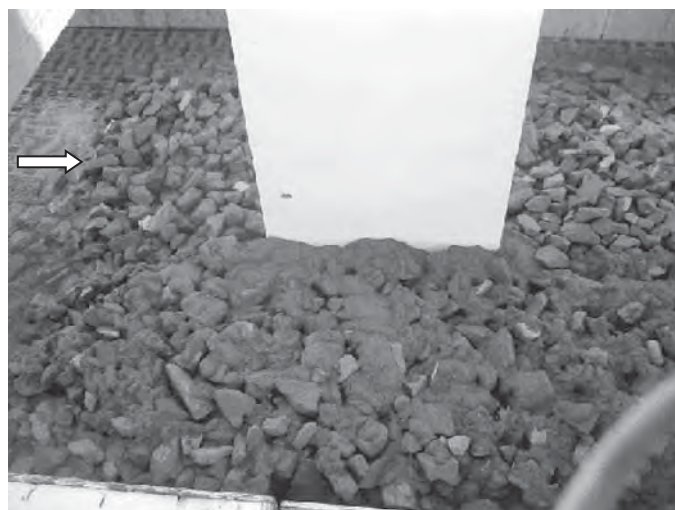


Figure 3.44. Partially grouted riprap after completion of the high-velocity comparison test.

Table 3.11. ACB properties.

Property	U.S. Customary Units	Metric Units	Comments
Length	1.5625 in.	40 mm	
Width	1.4375 in.	36 mm	
Height	0.6875 in.	17 mm	
Average weight/block	0.056 lb	25.30 g	Saturated
Average density	134 lb/ft ³	2.15 g/cm ³	Saturated
Critical shear stress	0.305 lb/ft ²	14.6 N/m ²	Tested at horizontal
Manning's n value	0.016	0.016	

**Figure 3.45. ACBs used in testing program.**

properties of the geotextile filters used for the ACB portion of this study.

3.6.2 Testing Program

The testing program addressed stability and performance issues associated with the extent of ACB placement around the

pier, and the termination details at the pier and around the periphery of the installation. In addition, various filter extents were investigated by varying this aspect for selected test runs (see Appendix H for details on the configurations tested).

Baseline

Because limited information is available on the use of ACBs as a pier scour countermeasure, baseline ACB installation conditions were based on observations from the riprap testing program and experience with ACBs in other erosion control applications. Initial installation of ACBs extended a horizontal distance of two pier widths on all sides for a total areal coverage of five pier widths. ACBs were toed down into the bed at a 2H:1V slope. Figure 3.46 presents the design layout for the baseline ACB tests. For all ACB tests the interface of the blocks and pier was sealed with grout and a full coverage geotextile filter was used.

The design intent for ACB baseline tests included examination of the following:

- Standard ACB layout
- Standard ACB layout with grouted interface

Initially, the baseline ACB installation was tested without grouting the interface of the ACB planes. A loss of blocks was observed under these conditions, as shown in Figure 3.47. The baseline ACB installation was stable with the ACB interfaces grouted when exposed to $1.9V_{crit}$ flow velocity and 4- to 6-in. (100- to 152-mm) dune passage. After being exposed to $2.5V_{crit}$ flow velocity and 8- to 10-in. (200- to 254-mm) dune passage, loss of blocks was observed. Results of two baseline conditions tests are shown in Figure 3.48. The ACBs were stable at $1.9V_{crit}$ (the design condition). Loss of blocks occurred at $2.5V_{crit}$ (20% greater than the design condition).

Table 3.12. Hydraulic and physical characteristics of geotextile filters.

Filter Name	Geotextile Type	Mass/ Unit Area	AOS	Permeability	Trade Name	Manufacturer	K_g/K_s
NW1	Non-woven	163 g/m ²	0.212 mm	0.21 cm/s	Mirafi® 140 N	Mirafi Construction Products	5.25
NW2	Non-woven	250 g/m ²	~ 0.10 mm	0.4 cm/s	HaTe® B 250 K4	Huesker Synthetic GmbH	10.0

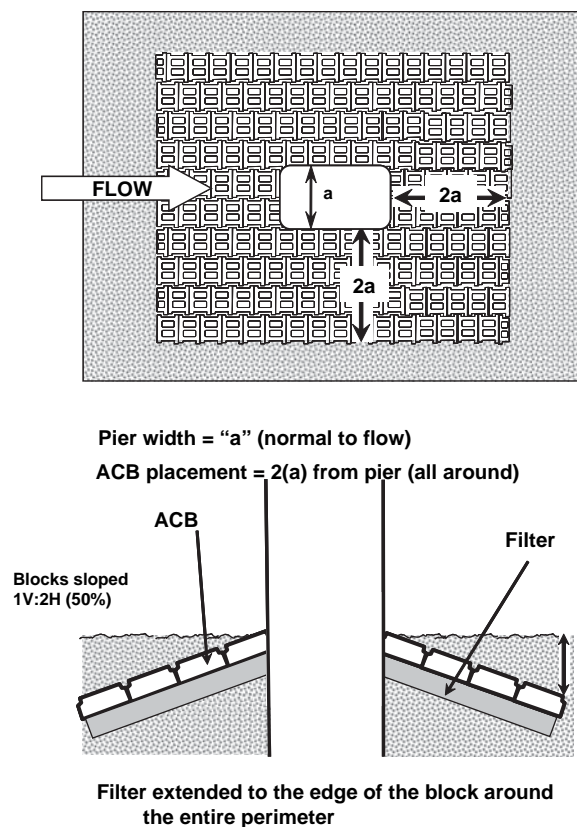


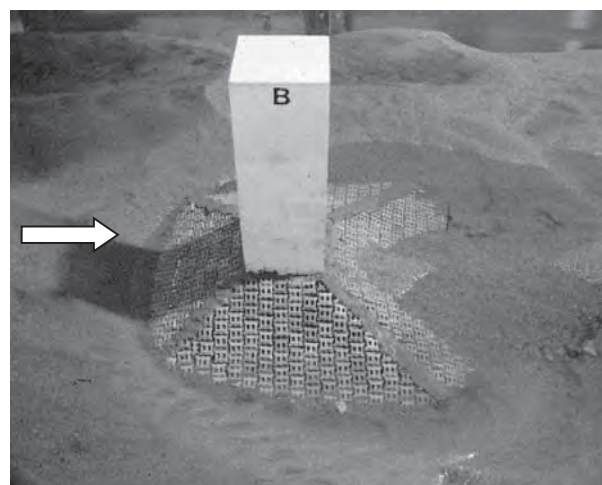
Figure 3.46. Baseline ACB design.

Extent of Coverage

The design intent for ACB extent of coverage tests included examination of the following:

- Areal coverage and termination detail
- ACBs in conjunction with riprap

For the tests with ACBs and riprap, the geotextile filter extended beyond the perimeter of the blocks under the riprap.



a. Baseline ACB installation after $1.9V_{crit}$ test.



Figure 3.47. Loss of blocks during testing without the ACB interfaces grouted.

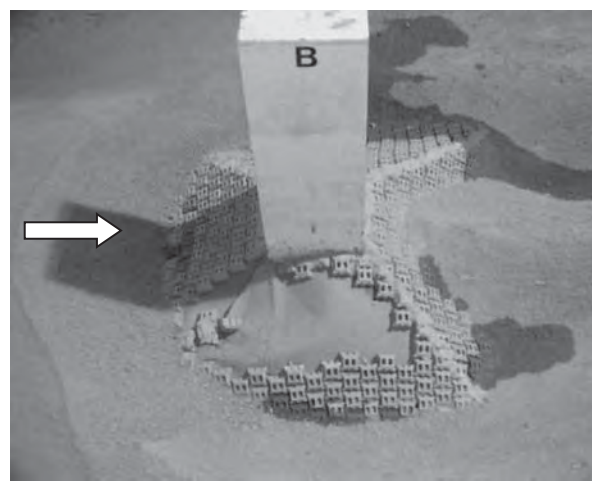
The results of two of the ACB coverage tests after exposure to $1.9V_{crit}$ flow conditions are shown in Figure 3.49.

Termination Detail

The design intent of the ACB termination detail tests included examination of the following:

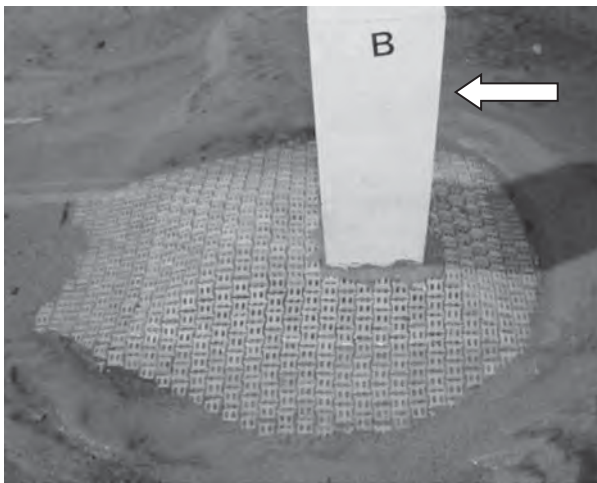
- Termination detail
- Areal coverage and termination detail
- ACB used in conjunction with riprap

For the tests with ACBs and riprap, the geotextile filter extended beyond the perimeter of the blocks under the riprap. When the ACBs did not extend below the dune troughs, severe loss of blocks was observed regardless of the turndown detail at the periphery. Figure 3.50 shows the construction details for

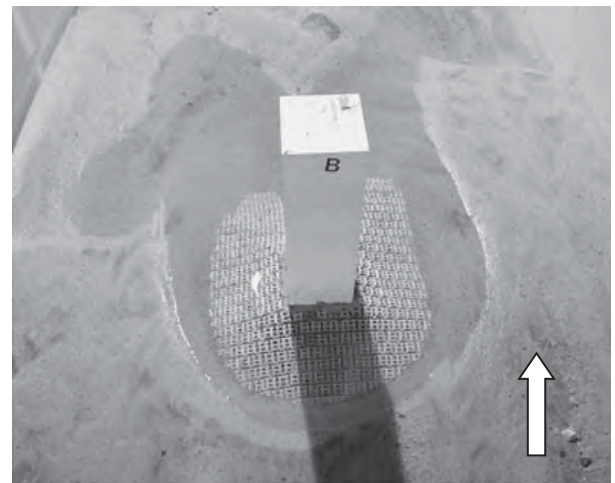


b. Baseline ACB installation after $2.5V_{crit}$ live-bed test.

Figure 3.48. After baseline ACB with grouted interface testing.



a. ACB configuration with coverage increased downstream of pier, after 2 hours of exposure at $1.9V_{crit}$ flow conditions.



b. ACB configuration with blocks installed below ambient bed elevation, after 2 hours of exposure at $1.9V_{crit}$ flow conditions

Figure 3.49. ACB coverage tests.

one of the termination tests. The results of the two termination detail tests without riprap after 7 hours of $1.9V_{crit}$ tests are shown in Figure 3.51.

Filter

Studies conducted on the effectiveness of ACBs as a countermeasure to scour have determined that the use of a filter fabric or geotextile is essential to the overall effectiveness and stability of the ACB system. In the absence of a filter or geotextile, winnowing can occur and result in subsidence of all or a portion of the ACB mat. One test was performed to examine two-thirds coverage of a geotextile. After being exposed to $2.5V_{crit}$ flow velocity with sediment feed and 8- to 10-in. (200- to 254-mm) dune passage, catastrophic loss of blocks was observed.

The design intent of ACB filter tests included examination of the following:

- Standard ACB layout with grouted interface below the bed surface
- ACB used in conjunction with riprap

For the tests with ACBs and riprap, the geotextile filter extended beyond the perimeter of the blocks under the riprap.

Figure 3.52 shows two piers after testing: one pier had a geotextile filter that extended two-thirds the distance from the pier face to the periphery (failed) and the other pier had a geotextile filter that extended beyond the perimeter of the ACBs and extended two-thirds the distance of an overlying riprap layer (stable).

3.7 Gabion Mattresses

There is limited experience with the use of gabion mattress systems as a scour counter-measure for bridge piers alone. More frequently, these systems have been used for structures such as dams or dikes, or for channel slope stabilization. For this reason, the gabion testing program was derived from results of the riprap and ACB testing programs. Typically, during gabion mattress installation in the field, the units are interconnected to form a single continuous layer. Testing procedures for the gabion mattresses included tests of both unconnected and connected units.

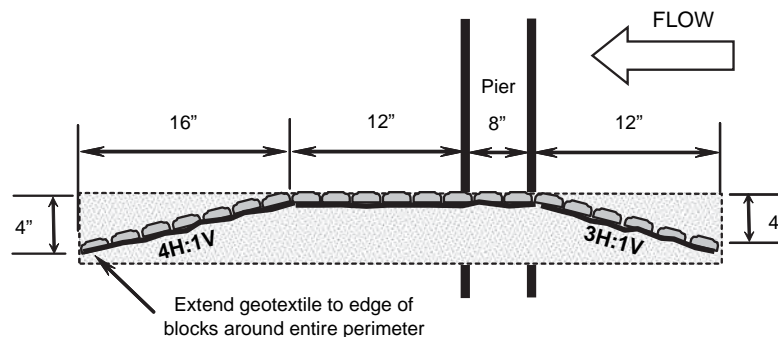
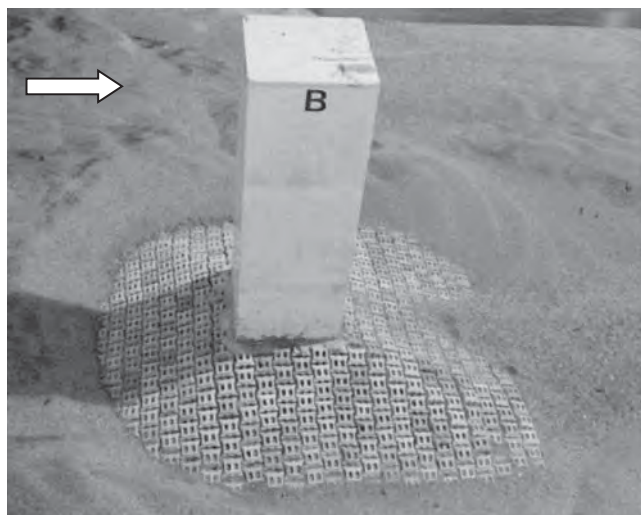
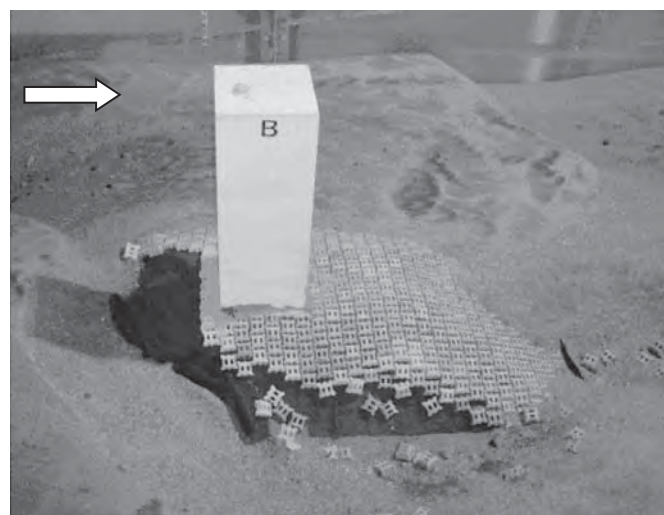


Figure 3.50. Example ACB termination test configuration.



a. ACB configuration with increased areal extent and depth after 7 hours of exposure at $1.9V_{crit}$ flow conditions.



b. ACB configuration from Figure 3.49a after 7 hours of exposure at $1.9V_{crit}$ flow conditions.

Figure 3.51. ACB termination tests.

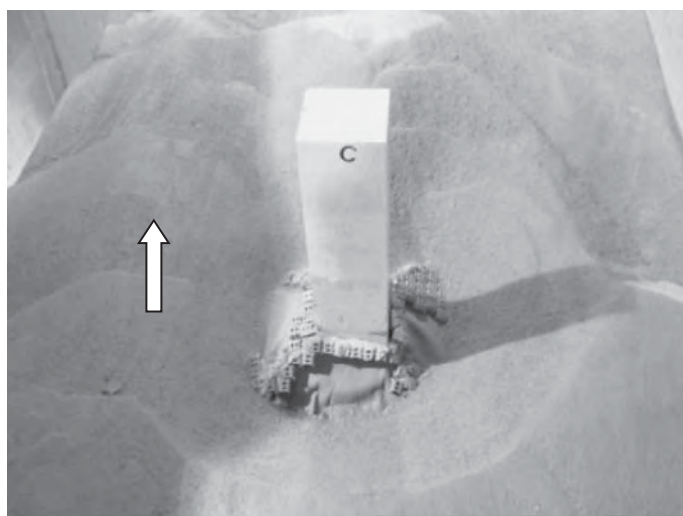
3.7.1 Materials

Gabions

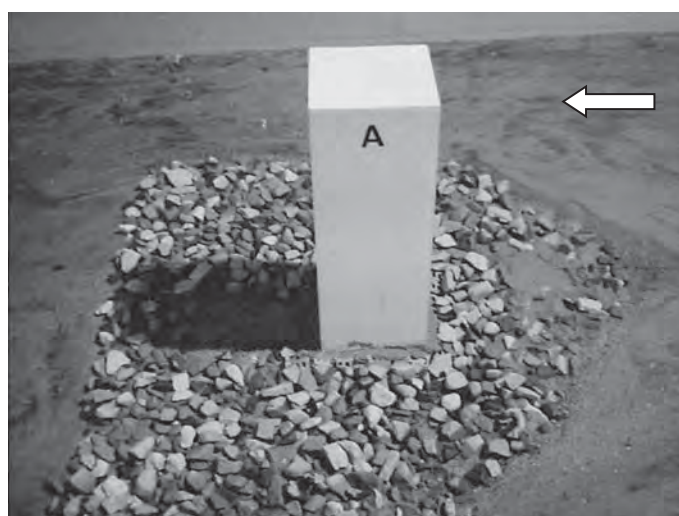
Gabion mattresses used in the laboratory tests consisted of wire mesh boxes filled with small gravel. Each gabion mattress was hand constructed with nominal dimensions of 6 in. (152 mm) long by 4 in. (102 mm) wide by 0.5 in. (12.5 mm) high. The wire mesh had a grid aperture size of 0.125 in. (3.2 mm) and was relatively flexible, but could be bent to obtain and hold a rectangular shape. Strips of plastic mesh typically used for craft projects were inserted as dividers in the gabion mattress to create three compartments in each mattress.

Gabion stone requirements were developed using the filter criteria specified in HEC-11 (Brown and Clyde 1989). The initial step establishes the compatibility of the filter (gabion stone) with the sand bed material in terms of both particle retention and permeability by defining upper and lower limits of d_{15} for the filter. This definition determines the largest size allowable to maintain particle retention and smallest size allowable to ensure the filter has greater permeability than the sand.

The fill material for the gabion mattresses was obtained from a local gravel supplier. It consisted of fine gravel having a d_{50} of 0.23 in. (5.8 mm) and a uniformity ratio d_{85}/d_{15} of 2.1. The specific gravity of the gravel ranged from 2.55 to 2.60. A



a. ACBs with two-thirds extent geotextile filter.



b. Riprap with ACBs. Riprap and geotextile filter extended beyond the ACBs.

Figure 3.52. ACB filter tests.

grain size distribution graph for the gabion stone is presented in Figure 3.53. Figure 3.54 shows a typical gabion mattress used in the testing program.

Tests to determine the critical shear stress for the gabion mattresses were performed in a smaller flume prior to placement around the test piers. The gabion mattresses did not fail under the prescribed hydraulic conditions. The maximum cross section averaged velocity observed during shear stress testing was 8.1 ft/s (2.5 m/s) and the maximum applied shear stress was 2.3 lb/ft² (110 N/m²), which occurred during a flow of 4.1 cfs (0.12 m³/s). Table 3.13 provides a summary of the physical and hydraulic characteristics of the gabion mattresses used in this study.

No attempt was made to scale the strength characteristics of the wire mesh and compartment divider material to the small size of the mattresses used in the testing program. Guidance for material strength and properties for field-scale applications are derived from relevant ASTM standards for these products and are described in detail in Appendix F.

Filter

A non-woven, needle-punched geotextile compatible with the bed material in the flume was used as the filter for all the gabion mattress tests. Geotextile selection for filter fabric was made using the method outlined in Koerner (1998), as summarized in Section 3.4.1. Table 3.14 summarizes the hydraulic and physical properties of the geotextile filter used in the gabion mattress portion of this study.

3.7.2 Testing Program

The testing program addressed stability and performance issues associated with the extent of gabion mattress placement around the pier and the termination details at the pier and around the periphery of the installation. In addition, various filter extents were investigated by varying this aspect for selected test runs (see Appendix H for details on the configurations tested). After testing coverage extent, filter extent, and termination detail, three installation designs were retested with the gabions sewn together at the edges.

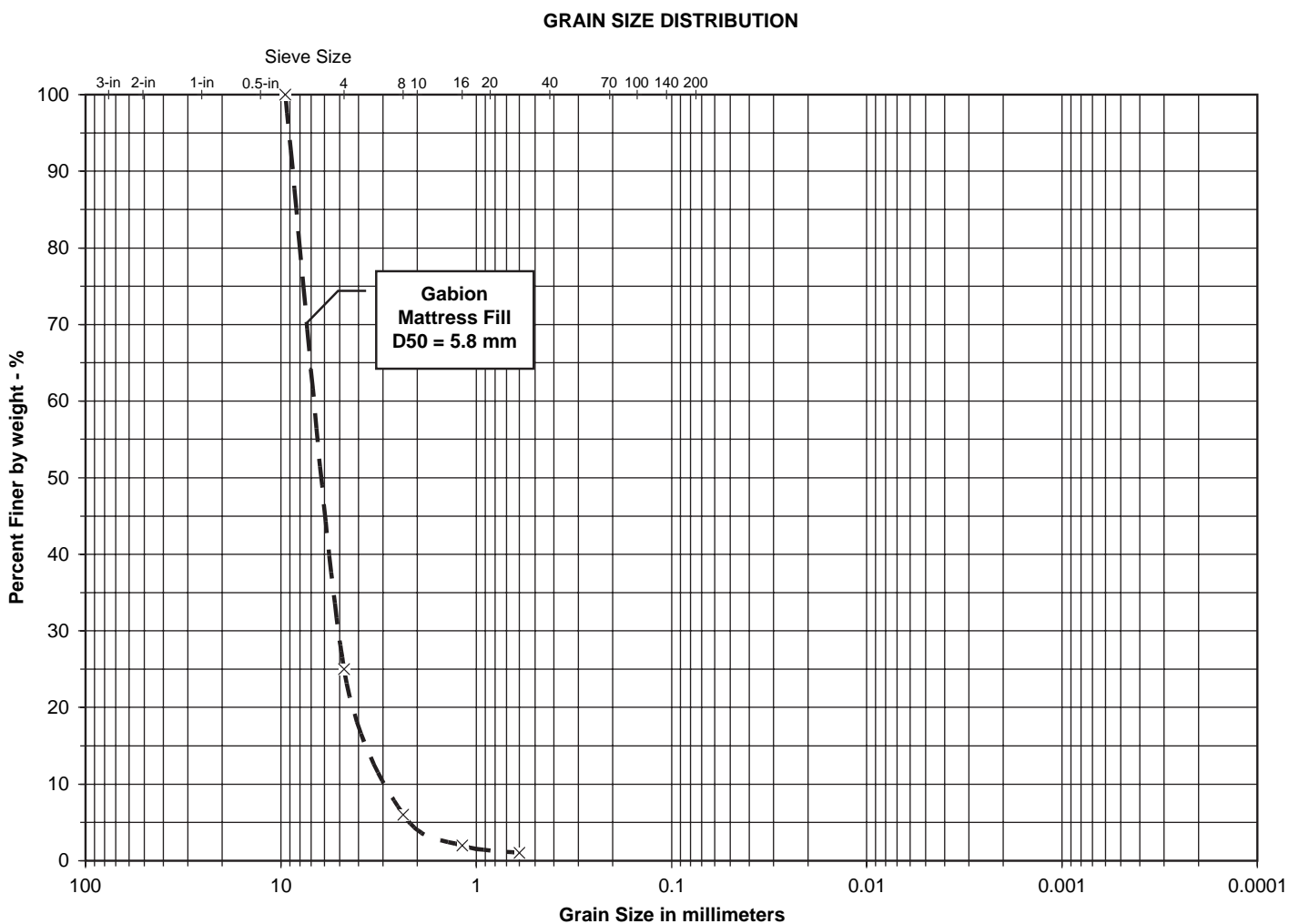


Figure 3.53. Grain size distribution for gabion mattress stone.

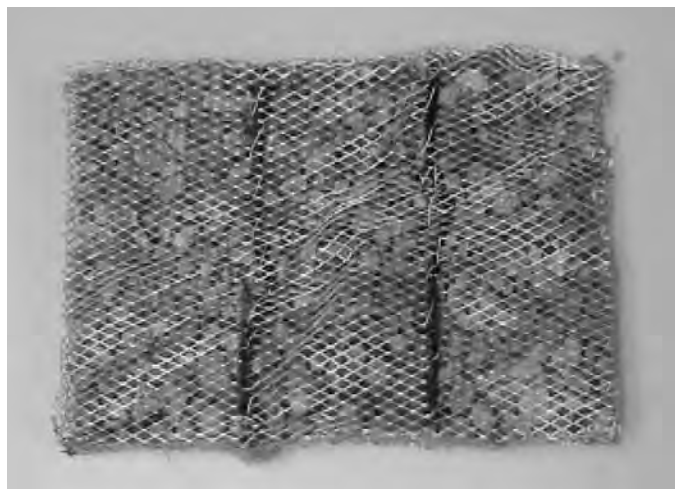


Figure 3.54. Typical gabion mattress with wire mesh and three compartments filled with stone.

Baseline

Because limited information is available on the use of gabion mattresses as a pier scour countermeasure, baseline gabion mattress installation conditions were not available. Observations from the ACB and riprap testing programs and experience with gabion mattresses in other erosion control applications were used to determine an initial starting point. First, gabion mattresses were examined for areal extent when installed flush with the ambient bed elevation, then extent of a geotextile filter and periphery turndown detail was examined.

Extent of Coverage

In the coverage tests, unconnected gabion mattresses extended a minimum horizontal distance of three pier widths normal to flow and four pier widths (1 gabion width = 0.5 a) parallel to flow; a geotextile filter extended from the pier face to the periphery of the gabion mattresses on all tests. Figure 3.55 presents the typical design layout for the gabion mattress coverage tests. All areal coverage tests of gabion mattresses included a full extent geotextile filter. Dimensions parallel and normal to the flow were varied.

Initial conditions were satisfactory when the areal extent was a minimum of four pier widths normal to flow and four pier widths parallel to flow; significant gabion displacement and loss was observed when areal coverage was less. Results of unconnected gabion mattress tests after $2V_{crit}$ flow conditions are shown in Figure 3.56.

Table 3.14. Hydraulic and physical characteristics of geotextile filter.

Filter Name	Geotextile Type	Mass/ Unit Area	AOS	Permeability	Trade Name	Manufacturer	K_g/K_s
NW2	Non-woven	250 g/m ²	~ 0.10 mm	0.4 cm/s	HaTe® B 250 K4	Huesker Synthetic GmbH	10.0

Table 3.13. Gabion mattress properties.

Property	U.S. Customary Units	Metric Units
Length	6.0 in.	152 mm
Width	4.0 in.	102 mm
Height	0.5 in.	12.5 mm
d_{50} of stone fill	0.23 in.	5.8 mm
Maximum applied velocity	8.1 ft/s	2.5 m/s
Maximum applied shear stress	2.3 lb/ft ²	110 N/m ²

Termination Detail

The design intent of the gabion mattress termination detail tests included examination of the following:

- Termination detail with full geotextile filter
- Filter extent and termination detail
- Termination detail with two-thirds geotextile filter

Appendix H, Table H.14, provides details on these tests.

Figure 3.57 shows the construction details for one of the termination tests. Termination detail tests examined conditions where the unconnected gabions were not installed horizontal but exhibited some form of turndown at the periphery.

Filter

Riprap test results confirmed that riprap performance was best when a geotextile filter extended two-thirds the distance to the periphery of the riprap. In the gabion mattress testing, filter extent was tested in conjunction with termination detail, thereby making results for performance of filter coverage difficult to isolate. Unconnected gabion mattresses tended to slide off the edge when the geotextile extended the full distance from the pier face to the periphery. When the geotextile coverage was two-thirds the distance of the gabions, the gabion mattresses were observed to displace into the troughs on the side of the installation or were lost completely. See the following section for results of connected gabion mattresses tested with two-thirds geotextile filter coverage.

Connected Gabion Mattresses

Gabion mattresses are often joined together to form a large mattress that when undermined or unstable can mold itself to the underlying subsurface, thus restoring stability to the unit. A series of tests was run with installation designs identical to previous tests, including a two-thirds geotextile filter extent,

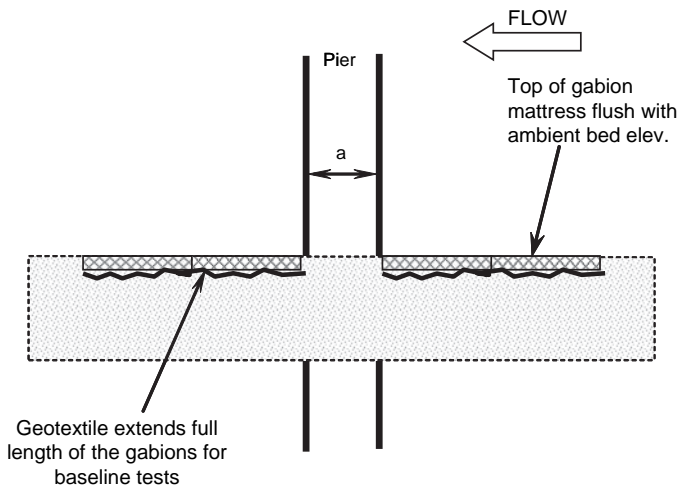


Figure 3.55. Baseline gabion mattress coverage test layout.

with the gabions connected along the edges. The gabions were observed to be stable and conforming to changes in the bed; however, these tests underscored the importance of providing an effective seal at the pier.

Figure 3.58 shows a comparison between identical installation designs with connected (Figure 3.58a) and unconnected (Figure 3.58b) gabions. Figure 3.59 shows the results of two connected gabion mattress tests.

3.8 Grout-Filled Mattresses

There is limited field experience with the use of grout-filled mattress systems as a scour countermeasure for bridge piers.

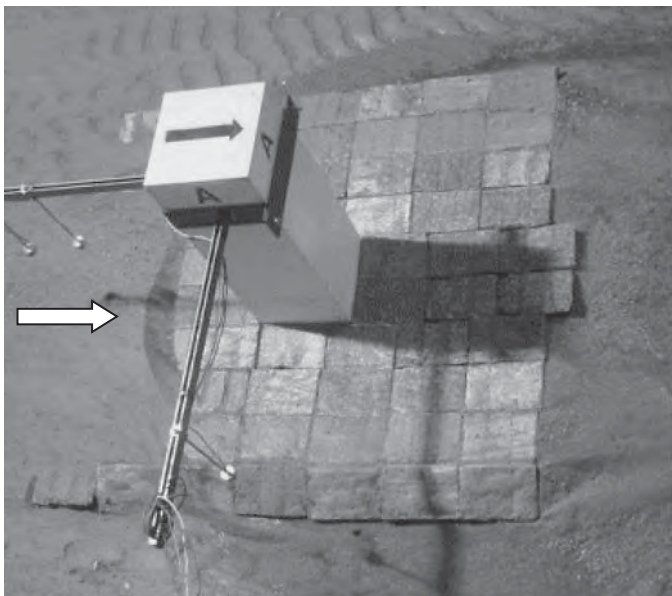
More frequently, these systems have been used for shoreline protection, underwater pipelines, and channel armoring where the mattress is placed across the entire channel width and keyed into the abutments or bank protection. For this reason, the grout-filled mattress testing program was derived from results of the gabion mattress and ACB testing programs. Both rigid and articulating configurations were tested.

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

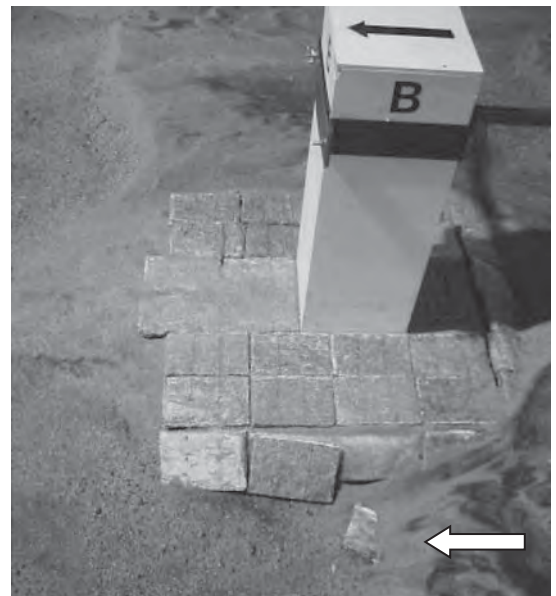
3.8.1 Materials

Mattresses

Rigid fabric-formed grout-filled mattresses were modeled by soaking a synthetic batting, typically used in quilting, in a cement-rich concrete grout. Mattresses were cut to fit the installation design, but typically a mattress was 8 in. x 12 in. (200 mm x 300 mm). The grout-filled mattresses were placed while wet on top of a geotextile filter around each pier. Relief of pore water pressure from beneath the mattress was allowed through weep holes cut into the center of each mattress and at corners where two mattresses joined. After the grout cured,



a. Greatest areal extent gabion mattress testing after $2.0V_{crit}$ test.



b. Gabion mattresses with insufficient areal extent after $2.0V_{crit}$ test.

Figure 3.56. After unconnected gabion mattress coverage testing.

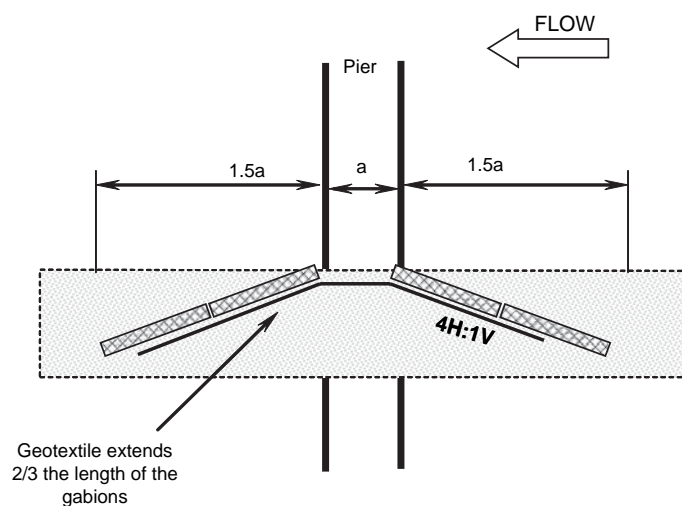


Figure 3.57. Example gabion mattress termination test configuration.

the thickness of each mattress was approximately 0.25 in. (6 mm). Figure 3.60 shows a rigid grout-filled mattress being placed while wet around a pier.

The flexible grout-filled mattresses were modeled using sheets of 1-in. (25-mm) square mosaic tile. The synthetic adhesive that connects the tiles together was scored with a razor blade to allow maximum flexibility while still maintaining block-to-block connection. The sheets were cut to fit each installation; abutting sheets were connected using several layers of cheesecloth, which also acted as a filter. Although each sheet exhibited excellent flexibility in the x- and y-directions separately, flexibility was quite limited when the sheet needed to flex in both planes simultaneously, for example, when wrapping around a corner or warped transition area. Figure 3.61 shows an articulating grout mattress being installed.

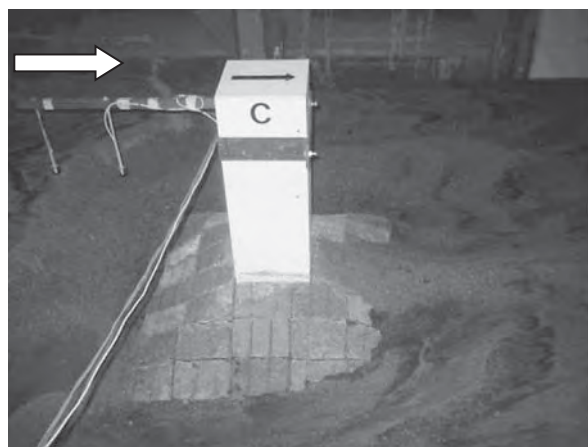


a. Connected gabion mattresses after 2 hours of exposure at $2.0V_{crit}$ flow conditions.

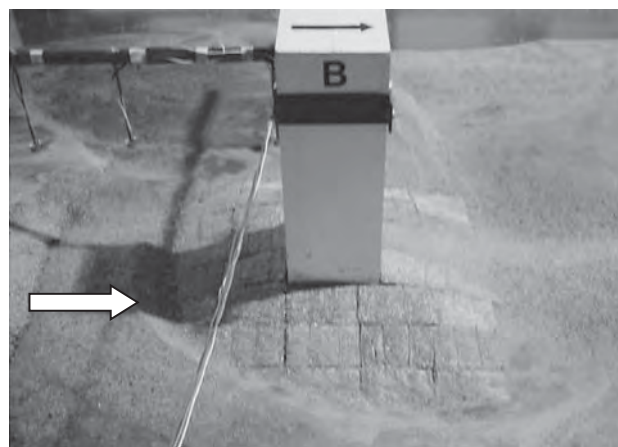


b. Unconnected gabion mattresses after 2 hours of exposure at $2.0V_{crit}$ flow conditions.

Figure 3.58. Comparison of results of connected and unconnected gabions of the same installation design.



a. Connected gabion mattresses after 2 hours of exposure at $2.0V_{crit}$ flow conditions.



b. Connected gabion mattresses after 2 hours of exposure at $2.0V_{crit}$ flow conditions.

Figure 3.59. Connected gabion mattress test results.



Figure 3.60. Placement of rigid grout-filled mattress.

Filter

Geotextile selection for filter fabric was made using the method outlined in Koerner (1998), as summarized in Section 3.4.1. Table 3.15 summarizes the hydraulic and physical properties of the geotextile filter used in the rigid grout-filled mattress portion of this study.

For the flexible grout mattress portion of the testing program, several layers of cheesecloth served as substitute for the geotextile filter. Hydraulic and physical properties were not available for this material.

3.8.2 Testing Program

The testing program addressed stability and performance issues associated with the extent of grout-filled mattress placement around the pier and the termination details at the pier and around the periphery of the installation (see Appendix H for details on the configurations tested).

Baseline

Because limited information is available on the use of grout-filled mattresses as a pier scour countermeasure, baseline grout-filled mattress installation conditions were not available. Observations from the ACB and gabion mattress testing program, as well as experience with grout-filled mattresses in other erosion control applications, were used to determine an initial

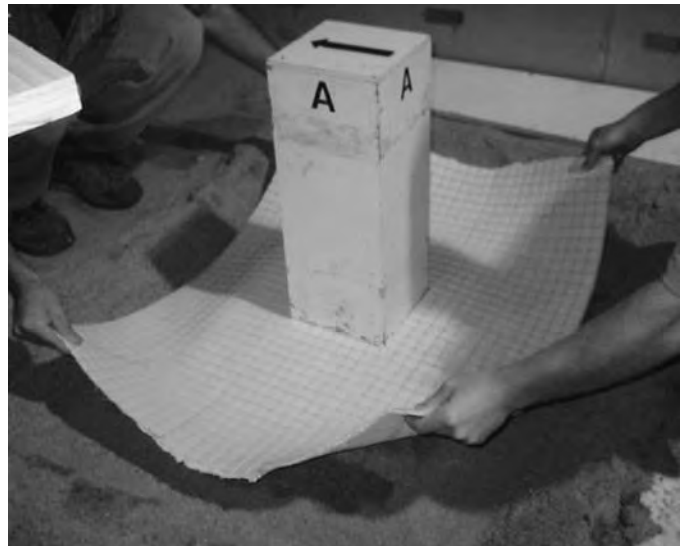


Figure 3.61. Installation of flexible grout mattress.

starting point. First grout-filled mattresses were examined for areal extent when installed flush with the ambient bed elevation and then periphery turn-down details were examined. The results of these tests are described in the following sections.

Extent of Coverage

In the coverage tests, grout-filled mattresses extended a minimum horizontal distance of three pier widths normal to flow and four pier widths parallel to flow; a geotextile filter extended from the pier face to the periphery of the mattress on all tests. Figure 3.62 presents the typical design layout for the rigid grout-filled mattress coverage tests. Figure 3.63 shows the results of two rigid grout-filled mattress areal coverage tests. The rigid grout-filled mattresses did not articulate with passing bed forms and as a result a significant amount of material was removed from beneath the mattresses in each test. Test results were unsatisfactory for all areal coverage tests.

Termination Detail

Termination detail tests examined conditions where the rigid grout-filled mattresses were not installed horizontal but exhibited some form of turn-down at the periphery. Stable conditions were observed when the grout-filled mattresses sloped away from the pier in all directions, but further investigation revealed that material had been removed from under the mattress, leaving pockets of empty space beneath the

Table 3.15. Hydraulic and physical characteristics of geotextile filter.

Filter Name	Geotextile Type	Mass/ Unit Area	AOS	Permeability	Trade Name	Manufacturer	K_g/K_s
NW2	Non-woven	250 g/m ²	~ 0.10 mm	0.4 cm/s	HaTe® B 250 K4	Huesker Synthetic GmbH	10.0

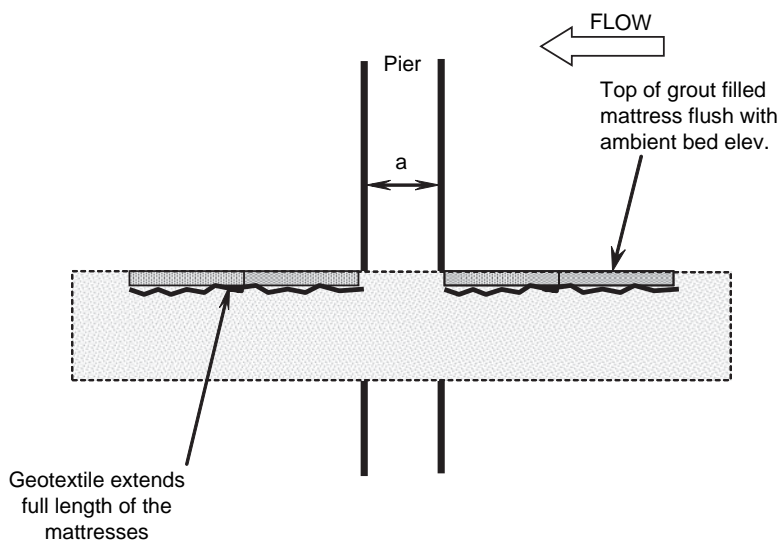


Figure 3.62. Typical grout-filled mattress coverage test layout.

countermeasure. Figure 3.64 shows the construction details for one of the termination tests.

Filter

Filter extent was not examined in this portion of the testing program. All rigid grout-filled mattress tests incorporated a geotextile that extended from the pier face to the periphery of the mattress.

Flexible Grout Mattresses

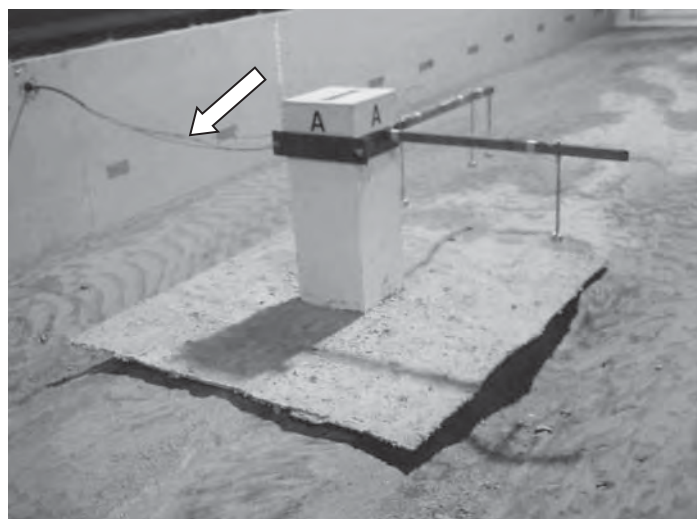
A series of tests was run with installation designs identical to previous tests except the rigid grout-filled mattresses were replaced with a flexible grout mattress. When the grout mattress was installed horizontal with a turndown on the periphery, test

results reveal a significant amount of material removed from under the grout mattress behind the pier. For the case where the grout mattresses sloped away from the pier, further investigation revealed that material had been removed from under the mattresses, leaving pockets of empty space beneath the countermeasure. Figure 3.65 shows the results of two flexible grout mattress tests.

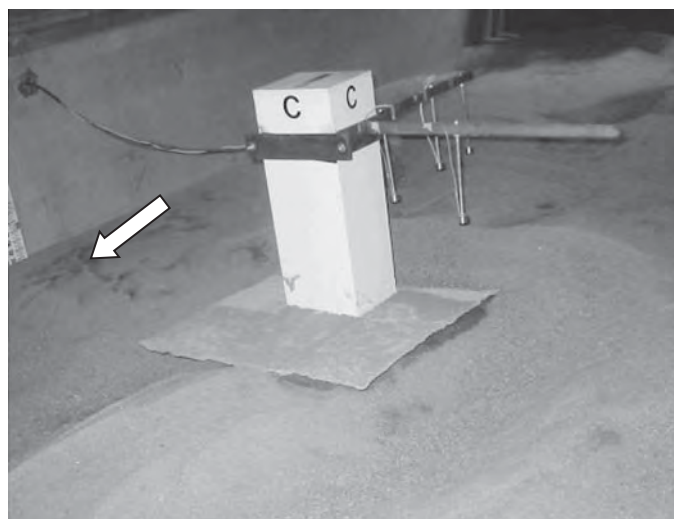
3.9 Design and Specification

3.9.1 Riprap

When properly designed and used for pier scour protection, riprap has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost,



a. Grout-filled mattress coverage test with greatest areal extent, after $2.0V_{crit}$ test.



b. Grout-filled mattress coverage test with smallest areal extent, after $2.0V_{crit}$ test.

Figure 3.63. Grout-filled mattress coverage test results.

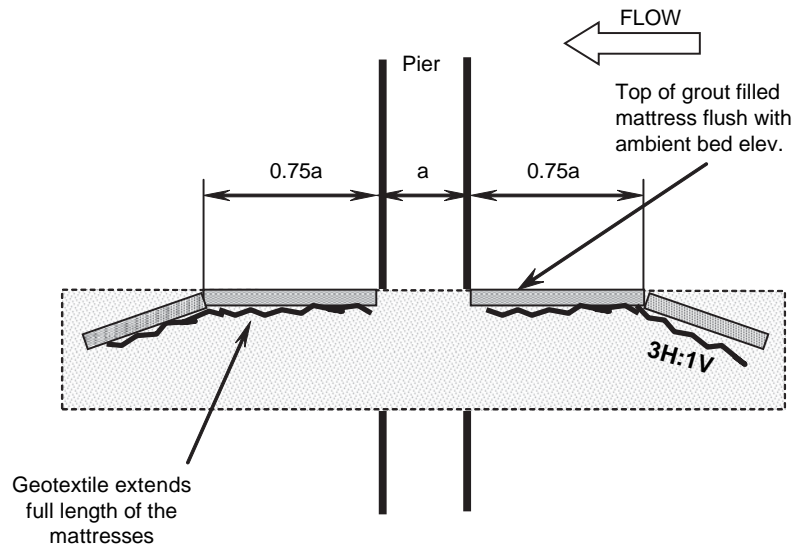


Figure 3.64. Example grout-filled mattress termination test configuration.

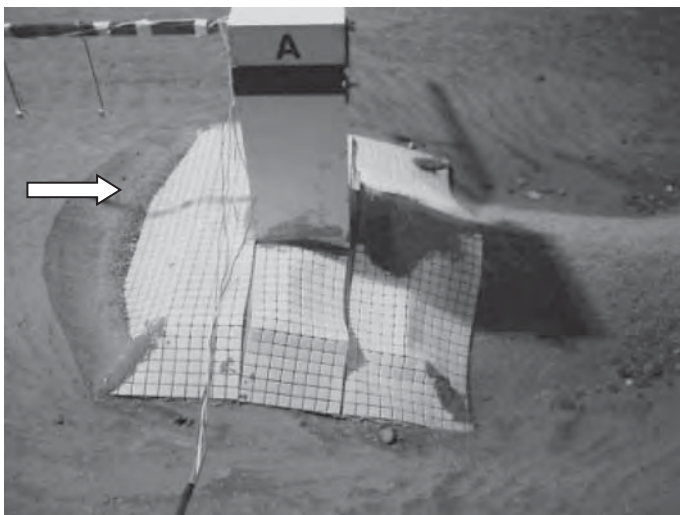
and it can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events.

Riprap tests conducted under NCHRP Project 24-07(2) included a wide variety of layout configurations at both square and rectangular (wall-type) piers, including piers skewed to the flow direction. Various filter extents and toedown methods were also investigated. An interpretation and appraisal of riprap as a pier scour countermeasure is presented in this section, drawing from the results and observations of the NCHRP Project 24-07(2) testing program, as well as from the

literature review conducted for this project and guidance from existing practice.

Design

Tests of riprap at piers were conducted using the pier riprap sizing method recommended in HEC-23 (Lagasse et al. 2001). The results of the tests confirmed that this velocity-based procedure is appropriate for sizing riprap at piers, provided that the extent and thickness of the armor layer, the gradation, and the design of the filter, also follow recommended guidelines, as discussed in the following paragraphs.



a. Flexible grout mattress after 2 hours of exposure at $2.0V_{crit}$ flow conditions. A significant amount of material was removed from under the grout mattress.



b. Flexible grout mattress with 2:1 turndown from pier face, after 2 hours of exposure at $2.0V_{crit}$ flow conditions. Further investigation revealed voids beneath the grout mattress.

Figure 3.65. Flexible grout mattress test results.

The HEC-23 pier riprap sizing procedure is recommended for use in designing pier scour countermeasures, and is presented in detail in Appendix C.

Layout

Results from riprap testing indicated that riprap areal coverage should be a minimum of two pier widths in all directions. Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation. Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create additional obstruction to the flow. The riprap layer should have a minimum thickness of three times the d_{50} size of the rock.

Poor results were observed when riprap was mounded on top of the bed. Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.

Tests confirmed that the lateral extent of riprap protection at rectangular piers must be increased when the longitudinal axis of the pier is skewed to the flow direction. At the outset of this study, no quantitative guidance existed to address this issue adequately. However, the required extent of protection at skewed piers can be inferred from the testing conducted under this project. Guidance for skewed piers is provided in the design guidelines appended to this report. The research team recommends that this topic be considered as an area needing future research.

Tests also confirmed that a filter should **not** be extended fully beneath the riprap; instead, the filter should be terminated two-thirds the distance from the pier to the edge of the riprap. When using a granular filter, the layer should have a minimum thickness of four times the d_{50} of the filter stone or 6 in. (152 mm), whichever is greater. Placing the filter and riprap under water was not investigated during the NCHRP Project 24-07(2) tests; therefore, guidance for this aspect comes from existing practice, which recommends that the layer thickness of both riprap and granular filter should be increased by 50% when placed under water. Granular filters are not recommended when dune-type bed forms are present. In addition, the riprap thickness should be increased if the depth of the bed form trough is greater than the recommended thickness of three times the d_{50} size of the riprap.

Materials

Riprap used in the laboratory tests consisted of a hard, durable sandstone having a specific gravity of 2.55 to 2.60. Other types of rock materials having different densities were not tested during this study; long-term weathering potential

also was not investigated. Recommendations for rock riprap quality, durability, and gradation are therefore derived from guidance developed using sources from both the United States and Europe. These recommendations, as well as conformance testing requirements, are provided in detail in Appendix C.

With respect to filter materials, both granular and geotextile filters were tested under clear-water and live-bed scour conditions. The effects of contraction scour and long-term degradation were not investigated in this study. Existing guidelines for the required engineering properties of these materials were found to be adequate under the conditions tested.

3.9.2 Partially Grouted Riprap

Partially grouted riprap consists of appropriately sized rocks that are placed around a pier and grouted together with grout filling 50% or less of the total void space. In contrast to fully grouted riprap, partial grouting increases the overall stability of the riprap installation unit without sacrificing flexibility or permeability. It also allows for the use of smaller rock compared to standard riprap, resulting in decreased layer thickness. The system typically includes a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention.

Tests of partially grouted riprap were conducted for NCHRP Project 24-07(2) around 8-in. (200-mm) square piers in an indoor flume using angular stone. Three different sizes of stone were investigated, with d_{50} values of 14.7, 25.4, and 30.0 mm. Partially grouted riprap was also tested at prototype scale in a large outdoor flume around a rectangular pier measuring 1.5 ft (0.5 m) wide by 4.5 ft (1.5 m) long. The riprap used in the outdoor tests had a d_{50} of 6 in. (152 mm). That stone size was somewhat smaller than the minimum recommended d_{50} of 9 in. (230 mm) used for field-scale partial grouting applications. For the prototype-scale tests, a filter composed of sand-filled geocontainers was placed under flowing water in a pre-existing scour hole around the pier. These tests confirmed the applicability of partially grouted riprap as a scour countermeasure for bridge piers.

Design

Design guidance for partially grouted riprap comes from the BAW in Germany. The intent of partial grouting is to “glue” stones together to create a conglomerate of particles. Each conglomerate is therefore significantly greater than the d_{50} stone size and typically is larger than the d_{100} size of the individual stones in the riprap matrix.

For practical placement in the field, riprap having a d_{50} smaller than 9 in. (230 mm) exhibits voids that are too small for grout to effectively penetrate to the required depth within

the rock matrix. At the other extreme, riprap having a d_{50} greater than 15 in. (380 mm) has voids that are too large to retain the grout, and does not have enough contact area between stones to effectively glue them together. With partially grouted riprap, there are no relationships *per se* for selecting the size of rock, other than the practical considerations of proper void size and adequate stone-to-stone contact area. Specific recommendations for design and specification are presented in detail in Appendix D.

Layout

The optimum performance of partially grouted riprap as a pier scour countermeasure was obtained when the riprap installment extended a minimum distance of one and a half times the pier width in all directions around the pier.

Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation. Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create any added obstruction to the flow. Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation. When used in a partially grouted application, the riprap layer should have a minimum thickness of two times the d_{50} size of the design riprap. When placement must occur under water, the thickness should be increased by 50%.

A filter layer is typically required for riprap at bridge piers. The filter should **not** be extended fully beneath the riprap; instead, it should be terminated two-thirds of the distance from the pier to the edge of the riprap.

Materials

Riprap used in the laboratory tests consisted of a hard, durable sandstone having a specific gravity of 2.55 to 2.60. Other types of rock materials having different densities were not tested during this study; long-term weathering potential also was not investigated. Recommendations for rock riprap quality, durability, and gradation are therefore derived from guidance developed using sources from both the United States and Europe. These recommendations, as well as conformance testing requirements, are provided in detail in Appendix D.

With respect to filter materials, only geotextile filters were tested with the partially grouted riprap. Existing guidelines for the required engineering properties of these materials were found to be adequate under the conditions tested. The use of sand-filled geotextiles composed of non-woven, needle-punched geotextile was confirmed to be an appropriate means of establishing a filter layer around a pier when placement must occur under water.

Standard Portland cement-based grout was used in the tests. For tests where the grout was placed under water, the recommended amount of Sicotan® admixture was included in the mix to minimize segregation and improve the “stickiness.” Specific recommendations for grout design and specification, including application quantities, are presented in detail in Appendix D.

3.9.3 Articulating Concrete Blocks

ACB systems provide a flexible armor for use as a pier scour countermeasure. These systems consist of preformed concrete units that either interlock, are held together by cables, or both. After installation is complete, the units form a continuous blanket or mat. The term “articulating” implies the ability of individual blocks of the system to conform to changes in the subgrade while remaining interconnected. Block systems are typically available in both open-cell and closed-cell varieties.

There is little field experience with the use of ACB systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for bank revetment and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. Tests conducted under NCHRP Project 24-07(2) confirm the applicability of ACB systems as a scour countermeasure for bridge piers.

Design

Tests of ACBs at piers were conducted using the factor of safety method recommended in HEC-23 (Lagasse et al. 2001). The results of the tests confirmed that this procedure is appropriate for designing ACBs for hydraulic stability at piers, provided that the extent of the armor layer, as well as the design of the filter, also follows recommended guidelines, as discussed in the following paragraphs. The HEC-23 procedure is therefore recommended for use in designing pier scour countermeasures using ACBs and is presented in detail in Appendix E. Testing also confirmed the importance of including block placement tolerance in the factor of safety calculations.

Layout

Results from the ACB testing indicated that the optimum performance of ACBs as a pier scour countermeasure was obtained when the blocks were extended a distance of at least two times the pier width in all directions around the pier. Because ACBs are essentially an erosion-resistant veneer that is one particle thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery. Blocks should not be placed on slopes greater than 2H:1V. When placed as pre-assembled mats,

they should never be placed such that a portion of one mat lies on top of another mat.

When dune-type bed forms were present, it was found that the armor must be sloped away from the pier in all directions such that the depth of the ACB system at its periphery is greater than the depth of the bed-form troughs. Although contraction scour and long-term degradation were not tested in this study, it is presumed that this same guidance applies in cases where these conditions may be present at the bridge crossing. In some cases, this requirement may result in blocks being placed further than two pier widths away from the pier. Test results confirmed that a filter should be extended fully beneath the ACBs.

Materials

Potential issues associated with long-term durability of the ACBs and variations in subgrade preparation were not investigated in this study. Therefore, guidance for concrete properties and construction techniques are derived from ASTM D 6684 and D 6884, respectively. This guidance is described in detail in Appendix E.

3.9.4 Gabion Mattresses

Gabion mattresses are containers constructed of wire mesh and filled with rocks. The length of a gabion mattress is greater than its width, and the width is greater than its thickness. Diaphragms are inserted widthwise into the mattress to create compartments. Wire is typically galvanized or coated with polyvinyl chloride to resist corrosion, and either welded or twisted into a lattice. Stones used to fill the containers can be either angular rock or rounded cobbles; however, angular rock is preferred because of the higher degree of natural interlocking of the stone fill. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous armor layer.

The wire mesh allows the gabions to deform and adapt to changes in the bed while maintaining stability. Additionally, when compared to riprap, less excavation of the bed is required and smaller, more economical stone can be used. The obvious benefit of gabion mattresses is that the size of the individual stones used to fill the mattress can be smaller than stone that would otherwise be required to withstand the hydraulic forces at a pier. Tests conducted under NCHRP Project 24-07(2) confirm the applicability of these systems as a scour countermeasure for bridge piers.

Design

There is limited field experience with the use of gabion mattresses as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for structures

such as in-channel weirs or drop structures, or for channel slope stabilization. The guidance for pier scour applications provided in this document has been developed primarily from the results of this study. Initial guidance to develop the testing program for gabion mattresses at piers was developed from the second edition of FHWA HEC-15 (Chen and Cotton 1988). The suitability of the basic design method, which is based on the concept of permissible shear stress, was subsequently confirmed for use at bridge piers by comparing the results of this testing program with the latest version of HEC-15 (Kilgore and Cotton 2005).

It should be noted that durability of the wire mesh under long-term exposure to flow conditions specific to bridge piers has not been demonstrated; therefore, the use of gabion mattresses as a bridge pier scour countermeasure has an element of uncertainty (Parker et al. 1998).

Layout

Results from the gabion mattress testing indicated that the optimum performance as a pier scour countermeasure was obtained when the mattresses were extended a distance of at least two times the pier width in all directions around the pier. Because gabion mattresses are essentially an erosion-resistant veneer that behaves as a unit that is one layer thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery. Gabion mattresses should not be placed on slopes greater than 2H:1V, nor should they be placed in a manner that causes them to lie on top of adjacent mattresses.

When dune-type bed forms were present, it was found that the armor must be sloped away from the pier in all directions such that the depth of the gabion mattress system at its periphery is greater than the depth of the bed-form troughs. Although contraction scour and long-term degradation were not tested during this study, it is presumed that this same guidance applies in cases where these conditions may be present at the bridge crossing. In some cases, this requirement may result in mattresses being placed further than two pier widths away from the pier. Similar to riprap, results confirmed that the filter should only be extended two-thirds of the distance from the pier to the periphery of the gabion mattress installation.

Testing under NCHRP Project 24-07(2) also confirmed that the gabion mattresses must be tied together using lacing wire or other types of mattress-to-mattress connectors. When mattresses were simply placed against one another without tying them together, scour around the perimeter of the installation caused individual mattresses to slide out of position. Additional scour and/or mattress displacement subsequently occurred between these gaps. From a practical standpoint, this observation indicates that field installations

must use mattress-to-mattress connection materials that are at least as strong as the wire mesh of the mattresses.

Materials

No attempt was made to scale the strength characteristics of the wire mesh and compartment divider material to the small size of the mattresses used in the testing program. Guidance for material strength and properties for field-scale applications are derived from relevant ASTM standards for these products and are described in detail in Appendix F.

3.9.5 Grout-Filled Mattresses

Grout-filled mattresses are composed of a double layer of strong synthetic fabric, typically woven nylon or polyester, sewn into a series of pillow-shaped compartments (blocks) that are connected internally by ducts. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Adjacent mattresses are typically sewn together or otherwise connected (less commonly) by special zips, straps, or ties prior to filling with grout.

The benefits of grout-filled mattresses are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the fabric forms and pumping them with concrete grout can be performed in areas where room for construction equipment is limited. When set, the grout forms a single-layer veneer made up of a grid of interconnected blocks. The blocks are interconnected by cables laced through the mattress before the grout is pumped into the fabric form, thus creating what is often called an articulating block mat (ABM). Flexibility and permeability are important functions for pier scour countermeasures. Therefore, systems that incorporate filter points or weep holes (allowing for pressure relief through the mat) combined with relatively small-diameter ducts (to allow grout breakage and articulation between blocks) are the preferred products.

There is limited field experience with the use of grout-filled mattresses as a scour countermeasure for bridge piers. More frequently, these systems have been used for shoreline protection, protective covers for underwater pipelines, and channel armoring where the mattresses are placed across the entire channel width and keyed into the abutments or banks. The guidance for pier scour applications provided in this document has been developed primarily from HEC-23 (Lagasse et al. 2001) and the results of NCHRP Project 24-07(2).

Design

Initial guidance for sizing the grout-filled mattresses for this study was developed from HEC-23, which recommends a

design method based on sliding stability. Both rigid and flexible materials were used to simulate grout-filled mattresses around 8-in. (200-mm) square piers. **The tests confirmed that grout-filled mattresses can be effective scour countermeasures for piers under clear-water conditions. However, when dune-type bed forms were present, the mattresses were subject to both undermining and uplift, even when they were toed down below the depth of the bed-form troughs. Therefore, the study cannot support the use of these products as pier scour countermeasures under live-bed conditions when dunes may be present.**

Layout

The optimum performance of grout-filled mattresses as a pier scour countermeasure was obtained when the mattresses were extended at least two times the pier width in all directions around the pier. Because these products are essentially an erosion-resistant veneer that behaves as a unit that is one layer thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery.

Although not specifically tested in this study, it is inferred that where long-term degradation and/or contraction scour is expected at a bridge crossing, grout-filled mattresses must be sloped away from the pier in all directions such that the depth of the mattress system at its periphery is greater than the depth of anticipated scour. The grout-filled mattresses should not be laid on a slope steeper than 2H:1V (50%). In some cases, this limitation may result in grout-filled mattresses being placed further than two pier widths away from the pier. Also, mattresses should not be placed such that a portion of one mattress lies on top of an adjacent mattress.

A filter layer is typically required for grout-filled mattresses at bridge piers. The filter should be extended fully beneath the system to its periphery.

Materials

Flexibility and permeability are important functions for pier scour countermeasures. Therefore, grout-filled mattress systems that incorporate filter points or weep holes (allowing for pressure relief through the mat) combined with relatively small-diameter ducts (to allow grout breakage and articulation between blocks) are the preferred products. No attempt was made to scale the strength characteristics of the fabric, grout, or block-to-block flexibility for the grout mattresses tested in this study. Guidance for material strength and properties for field-scale applications are derived from relevant ASTM standards for these products and are described in detail in Appendix G.

3.10 Construction

3.10.1 Overview

While construction techniques vary for each of the pier scour countermeasure systems tested, certain guidelines are common to all countermeasures. These are summarized in this section. More detailed, system-specific construction guidelines are provided in the appendixes. The guidelines are derived from several standard reference works including the *Manual on the Use of Rock in Hydraulic Engineering* (CUR 1995) published in the Netherlands; California Department of Transportation publications (e.g., Racine et al. 2000); the “Code of Practice: Use of Standard Construction Methods for Bank and Bottom Protection on Waterways” (MAR) (BAW 1993b) published in Germany; and manuals prepared by the U.S. Army Corps of Engineers (e.g., USACE 1987 and 1990).

3.10.2 General Guidelines

For any pier scour countermeasure system, the contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. Conformance to plans and specifications is typically ensured through the owner’s engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see USACE 1995) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for countermeasure placement are included in the project plans and specifications. Inspection and quality assurance must be carefully organized to ensure that materials delivered to the job site meet specifications. Acceptance should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject incorrect or unsuitable materials at the job site and have them removed from the project site. Material that has been improperly placed should also be rejected throughout the duration of the contract.

Construction techniques can vary tremendously because of the following factors:

- Size and scope of the overall project
- Size and weight of the riprap particles or armor units
- Placement under water or in the dry
- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions
- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general information regarding pier scour countermeasure installations that are common for all countermeasure types.

3.10.3 Filters

All pier scour countermeasures require a filter of some type. Generally, both geotextiles and granular filters can be used; however, some restrictions apply as noted in the individual design guidelines in the appendixes. **For riverine applications where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered for pier scour countermeasures.**

Geotextile Filters

Either woven or non-woven, needle-punched fabrics can be used. If a non-woven fabric is used, it should have a mass density greater than 12 oz/yd² (400 g/m²). Under no circumstances may spun-bond or slit-film fabrics be allowed. Each roll of geotextile should be labeled with the manufacturer’s name, product identification, roll dimensions, lot number, and date of manufacture. Geotextiles should not be exposed to sunlight prior to placement.

Granular Filters

Samples of granular filter material should be tested for grain size distribution to ensure compliance with the gradation specification used in design. Sampling and testing frequency should be in accordance with requirements established by the owner or owner’s authorized representative.

Subgrade Soils

When the countermeasure and filter is placed in the dry, they should be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Unsatisfactory soils include soils having excessive in-place moisture content; soils containing roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils should be removed, and the site backfilled with approved material and compacted prior to placement of the riprap. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays.

3.10.4 Installation

Subgrade Preparation

The subgrade soil conditions should meet or exceed the required material properties described in Section 3.10.3 prior to placement of the countermeasure. Soils not meeting the requirements should be removed and replaced with acceptable material.

When the countermeasure is placed in the dry, the areas receiving the countermeasure should be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the filter, and between the filter and the countermeasure. Stable and compacted subgrade soil should be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes should be compacted, shaped, and uniformly graded. The subgrade should be uniformly compacted to the geotechnical engineer's site-specific requirements.

When the countermeasure is placed under water, divers should be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create voids beneath the system. Immediately prior to placement of the filter and countermeasure system, the prepared subgrade must be inspected.

Filter Placement

Whether the filter comprises one or more layers of granular material or is made of geotextile, its placement should result in a continuous installation that maintains intimate contact with the soil beneath. Voids, gaps, tears, or other holes in the filter must be avoided to the extent practicable, and replaced or repaired when they occur.

Placement of Geotextile. The geotextile should be placed directly on the prepared area, in intimate contact with the subgrade. When a geotextile is placed, it should be rolled or spread out directly on the prepared area and be free of folds or wrinkles. The rolls should not be dragged, lifted by one end, or dropped. The geotextile should be placed in such a manner that placement of the overlying materials (countermeasure armor layer) will not excessively stretch or tear the geotextile.

After geotextile placement, the work area should not be trafficked or disturbed in a manner that might result in a loss of intimate contact between the countermeasure, the geotextile, and the subgrade. The geotextile should not be left exposed longer than the manufacturer's recommendation to minimize potential damage due to ultraviolet radiation; therefore, the overlying materials should be placed as soon as practicable.

The geotextile should be placed so that upstream strips overlap downstream strips. Overlaps should be in the direction of flow wherever possible. The longitudinal and transverse joints should be overlapped at least 1.5 ft (0.46 m) for dry installations and at least 3 ft (0.91 m) for underwater installations. If the seam of the geotextile is to be sewn, the thread to be used should consist of high-strength polypropylene or polyester and should be resistant to ultraviolet radiation. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or weights such as sandbags should be used.

Placement of Geotextiles Under Water. Placing geotextiles under water can be problematic for a number of reasons. Several techniques for placing geotextiles under water are presented in Section 3.12.5.

Placement of Granular Filter. For placing a granular filter, front-end loaders are the preferred method for dumping and spreading the material on slopes milder than approximately 4H:1V. A typical minimum thickness for granular filters is 0.5 to 1.0 ft (0.15 to 0.3 m), depending on the size of the overlying armor layer and whether a layer of bedding stone is to be used between the filter and the countermeasure. For a granular filter placed under water, the thickness should be increased by 50%. Underwater placement of granular media around a bridge pier is best accomplished using a large-diameter tremie pipe to control the placement location and thickness, while minimizing the potential for segregation.

Countermeasure Placement

Most countermeasure systems may be placed from either land-based or water-based operations and can be placed under water or in the dry. The necessary equipment and techniques are specific to each countermeasure type. These equipment and techniques are covered in the individual design guidelines in the appendixes.

3.11 Inspection, Maintenance, and Performance Evaluation

3.11.1 Inspection During Construction

Inspection during construction should be conducted by qualified personnel who are independent of the contractor. Underwater inspection of pier scour countermeasures should only be performed by divers specifically trained and certified for such work.

Subgrade

Inspection of the subgrade should be performed immediately prior to geotextile or granular filter placement. The sub-

grade should be clean and free of projections, debris, construction materials, or other foreign objects that would prevent the filter from being properly placed. Likewise, there should be no potholes, rills, or other voids that the filter material might bridge over.

The subgrade material itself should not be muddy or frozen, and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area should be noted and photographed. Observations of such should be brought to the attention of the project engineer as they may represent conditions that are different than those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product identification. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics and slit-film geotextiles should **never** be used in pier scour countermeasure applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, rocks, anchoring pins, or U-shaped soil staples may be used to hold the geotextile in position while the countermeasure is being placed. The countermeasure should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

Countermeasure

Inspection requirements for the countermeasure differ for each countermeasure type. These requirements are covered in detail in the individual design guidelines in the appendixes. In general, the subgrade preparation, geotextile placement, countermeasure system, and overall finished condition including termination trenches, if any, should be inspected before accepting the work.

3.11.2 Periodic and Post-Flood Inspection

Pier scour countermeasures will typically be inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a

specified magnitude. Underwater inspection should only be performed by divers specifically trained and certified for such work. Specific inspection requirements for each pier scour countermeasure system are provided in the design guidelines in the appendixes.

3.11.3 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. As with any armor system, progressive failure of a pier scour countermeasure from successive flows must be avoided by providing timely maintenance intervention for most countermeasure systems. Where localized areas are limited to loss of individual armor elements, there may be opportunities to repair the area by adding additional armor elements and tying them into the original armor layer.

Voids or undermining underneath the armor system should be filled with material that meets the specifications of the original design. Guidance specific to each countermeasure type is provided in the design guidelines in the appendixes.

3.11.4 Performance Evaluation

The evaluation of any countermeasure's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. For proper performance assessment of a pier scour countermeasure, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the pier scour countermeasure. Present-day channel cross-section geometry and planform should be compared to those at the time of countermeasure installation. Both lateral and vertical instability of the channel in the vicinity of the bridge can significantly alter hydraulic conditions at the piers. Approach flows may become skewed to the pier alignment, causing greater local and contraction scour.

Although the person making the performance evaluation will probably not be the inspector, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported as a percentage of the initial capital improvement cost.

To guide the performance evaluation for each pier scour countermeasure, a rating system is presented in each of the design guidelines in the appendixes. Numerical ratings from 0 (worst) to 6 (best) are established for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the current condition of the countermeasure?

Recommended actions corresponding to the current condition rating codes are also provided. For several countermeasures, a case history or example of a field performance evaluation is provided.

3.12 Filter Requirements

3.12.1 Filter Design

The importance of the filter component of a pier scour countermeasure installation should not be underestimated. Emphasis must be given to compatibility criteria between the filter (granular or geotextile) and the soil. Correct filter design reduces the effects of piping by limiting the loss of fines, while simultaneously maintaining a permeable, free-flowing interface. Figures 3.66a and 3.66b illustrate the basic difference between stable and unstable soil structures.

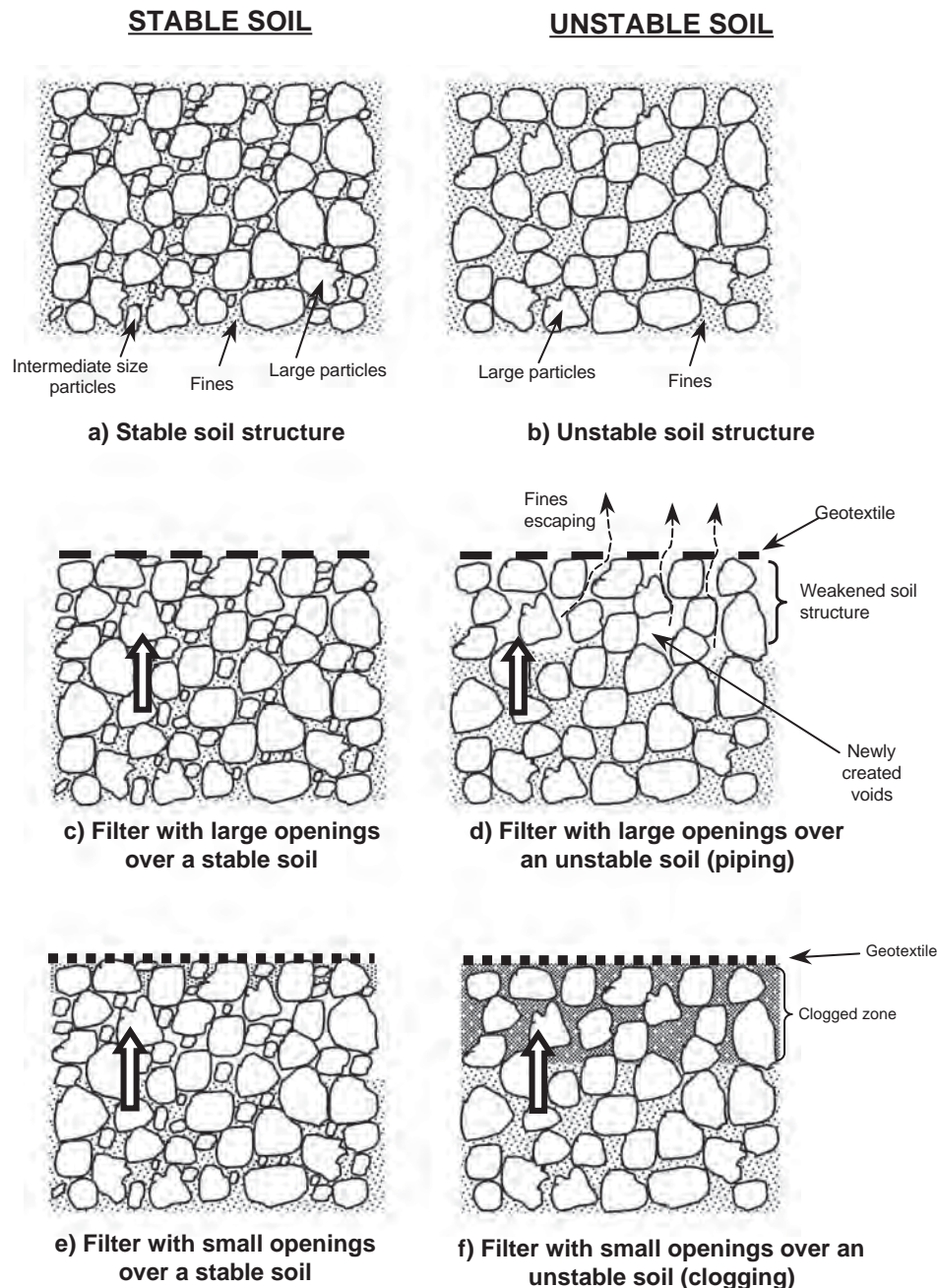


Figure 3.66. Examples of soil and filter compatibility processes.

Figures 3.66c through f illustrate several common filtering processes that can occur in stable and unstable base soils (modified from Geosyntech Consultants 1991). The large arrows indicate the direction of water flow in the base soil. In Figure 3.66c, the fine particles immediately adjacent to the filter are initially washed away (through the filter). The large and intermediate particles are retained by the filter; they in turn prevent any further loss of fines. This soil matrix will continue to remain stable over time.

In Figure 3.66d, an unstable soil is covered by a filter with large pores. Piping of the fine particles will continue unabated, because there are no particles of intermediate size to prevent their movement by the forces of seepage flow and turbulence at the interface.

In Figure 3.66e, a stable soil is covered by a filter with small pores. This filter will retain most of the fines, but the presence of intermediate-sized particles prevents the continued migration of fines from lower in the matrix. Thus a clogging layer is prevented from forming to any significant extent. This is contrasted with the condition shown in Figure 3.66f, where

no particles of intermediate size are present to mitigate the buildup of an impermeable barrier of plugged void spaces and clogging at the interface.

Filters must be sufficiently permeable to allow unimpeded flow from the base soil through the filter material for two reasons: (1) to regulate the filtration process at the base soil-filter interface, as illustrated in Figure 3.66, and (2) to minimize hydrostatic pressure buildup from local groundwater fluctuations in the vicinity of the channel bed and banks (e.g., seasonal water level changes or storm events).

The permeability of the filter should never be less than the material below it (whether base soil or another filter layer). Figures 3.67a through c illustrate the typical process that occurs during and after a flood event. Seepage forces can result in piping of the base soil through the countermeasure armor layer (e.g., riprap). If a less permeable material underlies the riprap, an increase of hydrostatic pressure can build beneath the riprap. A permeable filter material, properly designed, will alleviate problems associated with fluctuating surface water levels.

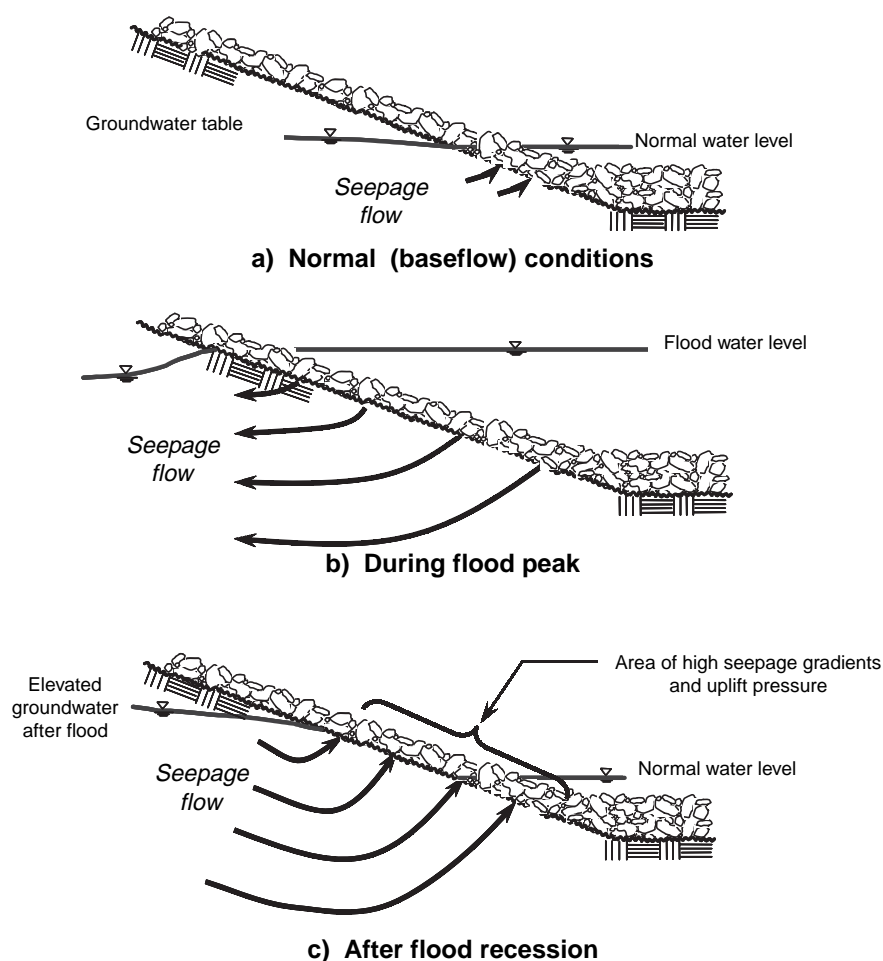


Figure 3.67. Changes in water levels and seepage patterns during a flood.

3.12.2 Base Soil Properties

Base soil is defined here as the subgrade material upon which the countermeasure and filter will be placed. Base soil can be native in-place material, or imported and recom-pacted fill. The following properties of the base soil should be obtained for proper design of the filter, when using either a geotextile or a layer of aggregate.

General Soil Classification

Soils are classified based on laboratory determinations of particle size characteristics and the physical effects of varying water content on soil consistency. Typically, soils are described as coarse grained if more than 50% by weight of the particles is larger than a #200 sieve (0.075-mm mesh) and fine grained if more than 50% by weight is smaller than this size. Sands and gravels are examples of coarse-grained soils, while silts and clays are examples of fine-grained soils.

The fine-grained fraction of a soil is further described by changes in its consistency caused by varying water content and by the percentage of organic matter present. Soil classification procedures are described in ASTM D 2487, “Standard Practice for Classification of Soils for Engineering Purposes: Unified Soil Classification System” (ASTM 2003a).

Particle Size Distribution

The single most important soil property for design purposes is the range of particle sizes in the soil. Particle size is a simple and convenient way to assess soil properties. Also, particle size tends to be an indication of other properties such as permeability. Characterizing soil particle size involves determining the relative proportions of gravel, sand, silt, and clay in the soil. This characterization is usually done by sieve analysis for coarse-grained soils or sedimentation (hydrometer) analysis for fine-grained soils. ASTM D 422, “Standard Test Method for Particle-Size Analysis of Soils,” outlines the specific procedure (ASTM 2003a).

Plasticity

Plasticity is defined as the property of a material that allows it to be deformed rapidly, without rupture, without elastic rebound, and without volume change. A standard measure of plasticity is the plasticity index (PI), which should be determined for soils with a significant percentage of clay. The results associated with plasticity testing are referred to as the Atterberg Limits. ASTM D 4318, “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils,” defines the testing procedure (ASTM 2003a).

Porosity

Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.

Permeability

Permeability, also known as hydraulic conductivity, is a measure of the ability of soil to transmit water. ASTM provides two standard laboratory test methods for determining permeability. They are ASTM D 2434, “Standard Test Method for Permeability of Granular Soils (Constant Head)” or ASTM D 5084, “Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter” (ASTM 2003b). In these tests, the amount of water passing through a saturated soil sample is measured over a specified time interval, along with the sample’s cross-sectional area and the hydraulic head at specific locations. The soil’s permeability is then calculated from these measured values. Permeability is related more to particle size distribution than to porosity, as water moves through large and interconnected voids more easily than small or isolated voids. Various equations are available to estimate permeability based on the grain size distribution. Table 3.16 lists average values of porosity and permeability for alluvial soils.

3.12.3 Geotextile Filter Properties

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percentage of open area). In addition, geotextiles must be sufficiently strong to withstand the stresses during installation. These values are available from manufacturers. The following paragraphs briefly describe the most relevant properties.

Table 3.16. Typical values of porosity and permeability of alluvial soils.

Type of Material	Porosity (vol/vol)	Permeability (cm/s)
Gravel, coarse	0.28	4×10^{-1}
Gravel, fine	0.34	
Sand, coarse	0.39	5×10^{-2}
Sand, fine	0.43	3×10^{-3}
Silt	0.46	3×10^{-5}
Clay	0.42	9×10^{-8}

Source: modified from McWhorter and Sunada (1977)

Permeability

The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimeters per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows perpendicularly through the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of (s)⁻¹. Permeability (and permittivity) is extremely important in riprap filter design. For scour countermeasure installations, the permeability of the geotextile should be at least 10 times that of the underlying material.

Transmissivity

The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second (cm²/s). This property is directly related to the drainage function and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas nonwoven, needle-punched fabrics have a much greater capacity due to their three-dimensional microstructure. Transmissivity is not particularly relevant to filter design.

Apparent Opening Size (AOS)

Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.

Porosity

Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to nonwoven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.

Percent Open Area (POA)

POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles

only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.

Thickness

As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).

Grab Strength and Elongation

Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.

Tear Strength

Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.

Puncture Strength

Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving pier scour countermeasure installation have been discussed here. Geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by the countermeasure, these stresses do not represent the environment that the geotextile will experience in the long term.

3.12.4 Granular Filter Properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters may be used alone or as a transitional layer between a predominantly fine-grained base soil and a geotextile.

Particle Size Distribution

As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material.

Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziems whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure 3.68 provides a design chart based on the Cistin–Ziems approach.

Permeability

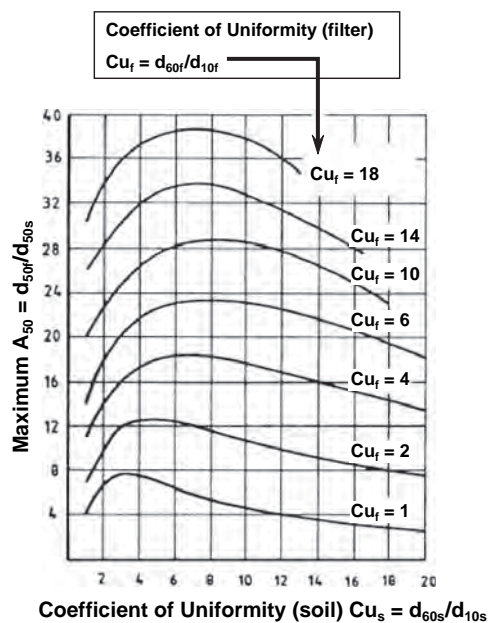
Permeability of a granular filter material is determined by laboratory test, or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For countermeasure installations, the permeability of the filter should be at least 10 times the permeability of the underlying material.

Porosity

Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.

Thickness

Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. should be specified. For placement under water, thickness should be increased by 50%.



Source: Heibaum (2004)

Figure 3.68. Filter design chart according to Cistin–Ziems.

Quality and Durability

Aggregate used for a granular filter should be hard, dense, and durable.

3.12.5 Placing Geotextiles Under Water

Placing geotextiles under water is problematic for a number of reasons. Most geotextiles that are used as filters beneath riprap are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, meaning that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the riprap (Koerner 1998). In addition, unless the work area is isolated from river currents by a cofferdam, flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric (a condition that contractors refer to as “galloping”) that are extremely difficult to control. In mild currents, geotextiles (precut to length) have been placed using a roller assembly, with sandbags to hold the fabric temporarily.

To overcome these problems, engineers in Germany have developed a product that consists of two non-woven geotextiles (or a woven and a non-woven geotextile) with sand in between. This blanket-like product, known as SandMat™, has layers that are stitch-bonded or sewn together to form a heavy, filtering geocomposite. The composite blanket exhibits an overall specific gravity ranging from approximately 1.5 to 2.0, so it sinks readily.

According to Heibaum (2002), this composite geotextile has sufficient stability to be handled even when loaded by currents up to approximately 3.3 ft/s (1 m/s). At the geotextile–subsoil interface, a non-woven fabric should be used because of the higher angle of friction compared to woven geotextiles. Figure 3.69 shows a close-up photo of the SandMat™ material. Figure 3.70 shows the SandMat™ blanket being rolled out using conventional geotextile placement equipment.

In deep water or in currents greater than 3.3 ft/s (1 m/s), German practice calls for the use of sand-filled geocontainers. For specific project conditions, geosynthetic containers can be chosen that combine the resistance against hydraulic loads with the filtration capacity demanded by the application. Geosynthetic containers have proven to give sufficient stability against erosive forces in many applications, including wave-attack environments. The size of the geocontainer must be chosen such that the expected hydraulic load will not transport the container during placement (Heibaum 2002). Once placed, the geocontainers are overlaid with the final armor material.

Figure 3.71 shows a geotextile container being filled with sand. Figure 3.72 shows the sand-filled geocontainer being



Source: Colcrete-Von Essen Inc.

Figure 3.69. Close-up photo of SandMat™ geocomposite blanket.

handled with an articulated-arm clam grapple. The filled geocontainer in the photograph is a nominal 1-tonne (1,000-kg or 2,200-lb) unit. The preferred geotextile for these applications is always a non-woven, needle-punched fabric, with a minimum mass per unit area of 500 g/m². Smaller geocontainers can be fabricated and handled by one or two people for smaller-sized applications.

As a practical minimum, a 200-lb (90.7 kg) geocontainer covering a surface area of about 6 to 8 ft² (0.56 to 0.74 m²) can be fashioned from non-woven, needle-punched geotextile having a minimum mass per unit area of 200 g/m², filled at the job site, and field-stitched with a hand-held machine. Figures 3.73 and



Source: Colcrete-Von Essen Inc.

Figure 3.71. Filling geocontainer with sand.



Source: Colcrete-Von Essen Inc.

Figure 3.70. SandMat™ geocomposite blanket being unrolled.



Source: Colcrete-Von Essen Inc.

Figure 3.72. Handling a 1-tonne sand-filled geocontainer.



a. Demonstrating puncture resistance of geocontainers



b. Placing geocontainers with small front-end loader into scour hole

Figure 3.73. Small (200-lb [90.7-kg]) sand-filled geocontainers for prototype-scale test.

3.74 illustrate the smaller geocontainers being installed at a prototype-scale test installation (for more detail see Section 3.5.3).

3.13 Pier Scour Countermeasure Selection

Selecting the most appropriate pier scour countermeasure for a particular bridge site requires knowledge of not only the bridge characteristics and riverine conditions that have combined to create a potential scour-critical situation, but also the strengths and vulnerabilities of the countermeasures being considered. In addition, the costs associated with the installation and maintenance of the countermeasure throughout the remaining life of the bridge must be considered.

A methodology was developed under NCHRP Project 24-07(2) to assist practitioners in the selection of appropriate

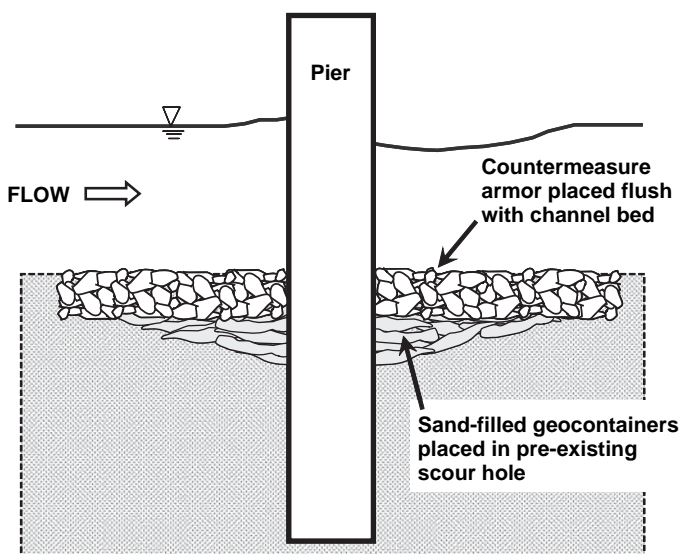


Figure 3.74. Schematic diagram of sand-filled geocontainers beneath riprap armor.

pier scour countermeasures for a given set of site-specific conditions. The method has been incorporated into a Microsoft® Excel workbook that is available on the TRB web site (http://www.trb.org/news/blurb_detail.asp?id=7998). The user-friendly Excel workbook allows the practitioner to customize the selection process to determine the relative suitability of six different pier scour countermeasure alternatives at a given site. The methodology provides a quantitative ranking of armoring countermeasure types and incorporates a fatal-flaw mechanism that identifies situations where a particular countermeasure is unequivocally unsuitable because of one or more circumstances unique to the site. The methodology is presented in detail in Appendix B of this report.

The selection methodology is intended to identify the countermeasure type best suited for application at a particular site. It is *not* intended to be used as a tool for comparing between different sites and would not be useful for prioritizing among various bridge sites where pier scour countermeasures are being considered.

Alaska, California, and Virginia DOTs participated in beta testing the selection methodology. Comments and recommendations from the beta testers, as well as review comments received from members of the NCHRP Project 24-07(2) panel, were incorporated into the methodology. Appendix B describes the final methodology that resulted from this process.

3.14 Implementation Plan

3.14.1 The Product

As described in more detail in the preceding sections, the product of this research was practical selection criteria for bridge pier scour countermeasures; guidelines and recommended specifications for design and construction; and

guidelines for inspection, maintenance, and performance evaluation. The following countermeasures were considered:

- Riprap
- Partially grouted riprap and geotextile containers
- ACB systems
- Gabion mattresses
- Grout-filled mattresses

3.14.2 The Market

The market or audience for the results of this research will be hydraulic engineers and maintenance and inspection personnel in state, federal, and local agencies with a bridge-related responsibility. These would include the following:

- State highway agencies
- Federal Highway Administration
- City/county bridge engineers
- Railroad bridge engineers
- U.S. Army Corps of Engineers
- Bureau of Land Management
- National Park Service
- Forest Service
- Bureau of Indian Affairs
- Any other governmental agency with bridges under its jurisdiction
- Consultants to the agencies above

3.14.3 Impediments to Implementation

A serious impediment to successful implementation of results of this research will be difficulties involved in reaching a diverse audience scattered among numerous agencies and institutions; however, this can be countered by a well-planned technology transfer program. Because of the complexity and geographic scope of the bridge scour problem and the diversity of bridge foundation geometries, a major challenge was to present the results in a format that can be applied by agencies with varying levels of engineering design capabilities and maintenance resources. Presenting the selection criteria and guidelines in a format familiar to bridge owners, who are the target audience, will facilitate their use of the results of this research. The standard format adopted for this study will help ensure successful implementation.

3.14.4 Leadership in Application

Through the National Highway Institute (NHI) and its training courses, FHWA has the program in place to reach a diverse and decentralized target audience. For example, recommendations from this study could be considered for the

next edition of HEC-23, “Bridge Scour and Stream Instability Countermeasures,” and NHI Course No. 135048, “Countermeasure Design for Scour and Stream Instability.”

TRB—through its annual meetings and committee activities, publications such as the *Transportation Research Record*, and periodic bridge conferences—can also play a leading role in disseminating the results of this research to the target audience.

AASHTO is the developer and sanctioning agency for standards, methods, and specifications. Thus, research results can be formally adopted through the AASHTO process. As a collective representation of individual state DOTs, AASHTO can also suggest any needed training to be developed by FHWA or others. The AASHTO Subcommittee on Bridges and Structures could provide centralized leadership through the involvement of all state DOT bridge engineers.

ASTM is a recognized leader in the development of standard specifications for the testing and documentation of material quality and performance. In 1997, Subcommittee D18-25 on Erosion and Sediment Control Technology was created. This subcommittee consists of 11 sections that are developing standards for a variety of erosion control products and applications, including articulating concrete blocks (D18-25.04), gabions (D18-25.05), and grout-filled fabric mattresses (D18-25.07). Obviously, material quality standards for manufactured products are essential for durability and longevity in their application as scour countermeasures. Similarly, performance testing is essential for the development of design procedures. ASTM standards development can provide a valuable linkage between the proposed research activities and the engineering community involved in design and specification.

Professional societies such as **ASCE** host conferences and publish peer-reviewed journals through which the latest advances in engineering research and applications reach a wide audience, including many state, federal, and local hydraulic engineers. For example, the Environmental & Water Resources Institute (EWRI)/ASCE Task Committee on Bridge Scour can play an important role in disseminating the results of this research.

Regional bridge conferences, such as the Western Bridge Engineer Conference or the International Bridge Engineering Conferences, reach a wide audience of bridge engineers, manufacturers, consultants, and contractors. The groups would have an obvious interest in pier scour countermeasures and their acceptance of the results of this research will be key to implementation by bridge owners.

3.14.5 Activities for Implementation

The activities necessary for successful implementation of the results of this research relate to technology transfer activities, as discussed in the previous section, and the activities of appropriate AASHTO and ASTM committees.

“Ownership” of the guidelines and specifications by AASHTO will be key to successful implementation. Although the guidelines and specifications that result from this research will be considered and possibly adopted by AASHTO, it is essential that the various technical committees in AASHTO accept and support these results and use the committee structure to improve them in the future.

Standards development activities within ASTM’s Erosion and Sediment Control Technology subcommittee include “Standard Guidelines for Design” and “Standard Practices for Installation” associated with various erosion and sediment control products, techniques, and areas of application. The design and installation guidance developed for selected pier scour countermeasures under this study could be formatted and published as ASTM standards documents. Such publication would unquestionably further the dissemination of information and enhance the usefulness of this work for the professional design community as well as for installation contractors and owners.

3.14.6 Criteria for Success

The best criteria for judging the success of this implementation plan will be acceptance and use of the guidelines and specifications that result from this research by state highway agency engineers and others with responsibility for design, maintenance, rehabilitation, or inspection of highway facilities. Progress can be gaged by peer reviews of technical presentations and publications and by the reaction of state DOT personnel during presentation of results at NHI courses. A supplemental critique sheet could be used during NHI courses to provide feedback on the applicability of the guidelines and suggestions for improvement.

The desirable consequences of this project, when implemented, will be more efficient design, maintenance, and inspection of highway facilities considering the threat from pier scour, and more effective use of countermeasures against that threat. The ultimate result will be a reduction in the number of bridge failures and reduction in damage to highway facilities attributable to pier scour.

CHAPTER 4

Conclusions and Suggested Research

4.1 Applicability of Results to Highway Practice

Approximately 83% of the 583,000 bridges in the NBI are built over waterways. Many, especially those on more active streams, will experience problems with scour, bank erosion, and channel instability during their useful life (Lagasse et al. 2001). The magnitude of these problems is demonstrated by the estimated average annual flood damage repair costs of approximately \$50 million for bridges on the federal aid system.

Highway bridge failures caused by scour and stream instability account for most of the bridge failures in this country. A 1973 study for the FHWA (Chang 1973) indicated that about \$75 million were expended annually up to 1973 to repair roads and bridges that were damaged by floods. Extrapolating the cost to the present makes this annual expenditure to roads and bridges on the order of \$300 to \$500 million. This cost does not include the additional indirect costs to highway users for fuel and operating costs resulting from temporary closure and detours and to the public for costs associated with higher tariffs, freight rates, additional labor costs and time. The indirect costs associated with a bridge failure have been estimated to exceed the direct cost of bridge repair by a factor of five (Rhodes and Trent 1993). Rhodes and Trent (1993) document that \$1.2 billion was expended for the restoration of flood-damaged highway facilities during the 1980s.

Although it is difficult to be precise regarding the actual cost to repair damage to the nation's highway system from problems related to pier scour, the number is obviously very large. In addition, the costs cited above do not include the extra costs that result from over-design of bridge foundations (i.e., deeper foundation depths, unnecessary or over-designed countermeasures) that result from our inability to select and design pier scour countermeasures with precision and confidence. This lack of knowledge often results in overly conservative design.

For example, current FHWA policy considers riprap placed at bridge piers to be effective in reducing the risk from pier scour, but guidance dictates that riprap placed at bridge piers must be monitored by periodic inspection or with fixed instruments. This policy derives from experience with the difficulty of adequately sizing and properly installing riprap to withstand the turbulence and hydraulic stress generated in the vicinity of a bridge pier, particularly under flood-flow conditions.

Similarly, a lack of specific design guidelines and specifications for other potentially effective pier scour countermeasures has resulted in only limited application of countermeasures such as partially grouted riprap, articulating concrete block, gabion mattresses, grout-filled mattresses, and geotextile containers. The guidelines, specifications, and recommendations from this research will provide a range of options to bridge owners for countering the effects of scour at piers and permit selecting the appropriate countermeasure for a specific problem. The end result will be a more efficient use of highway resources and a reduction in costs associated with the impacts of pier scour on highway facilities.

4.2 Conclusions and Recommendations

4.2.1 Overview

This research accomplished its basic objectives of developing guidelines and recommended specifications for design and construction, and guidelines for inspection, maintenance, and performance evaluation for a range of pier scour countermeasures including riprap, partially grouted riprap, articulating concrete blocks, gabion mattresses, grout-filled mattresses, and geotextile sand containers.

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, velocity, flow angle, pier shape and width, and other factors. If the determination is made 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, the most appropriate countermeasures must be selected, designed, constructed, and maintained.

In this study, existing design equations for sizing the armor component of the pier scour countermeasures of interest were used to develop a laboratory testing program. However, sizing the armor is only the first step in the comprehensive design, installation, inspection, and maintenance process required for a successful countermeasure. A countermeasure is an integrated system that includes the armor layer, filter, and termination details. Successful performance depends on the response of each component of the system to hydraulic and environmental stresses throughout its service life. In this context, filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also necessary. Each system typically includes a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention.

To support the selection of an appropriate pier scour countermeasure for site-specific conditions, a countermeasure selection methodology was developed. It provides an assessment of the suitability of each of five specific countermeasure types based on a variety of factors involving river environment, construction considerations, maintenance, performance, and estimated life-cycle cost of each countermeasure.

Conclusions and recommendations for each of the pier scour countermeasures investigated are summarized in the following sections. In addition, some generalized observations on pier scour protection systems are offered. **For each pier scour countermeasure type, detailed design guidelines that incorporate these conclusions and recommendations and provide additional guidance are included as stand-alone appendixes.**

4.2.2 Riprap

When properly designed and used for pier scour protection, riprap has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. **Tests conducted under NCHRP Project 24-07(2) validated and**

extended existing guidelines for using riprap as a scour countermeasure for bridge piers.

Design

Results of the tests confirmed that the HEC-23 (Lagasse et al. 2001) velocity-based procedure is appropriate for sizing riprap at piers, provided that the extent and thickness of the armor layer, the gradation, as well as the design of the filter, also follow recommended guidelines.

Layout

- Riprap areal coverage should extend a distance of at least two times the pier width in all directions around the pier.
- Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation.
- Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create additional obstruction to the flow.
- Riprap layer should have a minimum thickness of three times the d_{50} size of the rock.
- The riprap thickness should be increased if the depth of the bed-form trough or contraction scour and long-term degradation is greater than the recommended thickness of three times the d_{50} size of the riprap.
- Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.
- For wall-type piers skewed to the flow, the riprap extent should be increased by a function of the skew angle, α ; the pier width, a ; and the pier length, L .

Filter

- The filter should not be extended fully beneath the riprap, but should be terminated two-thirds the distance from the pier to the edge of the riprap.
- When a granular filter is used, the layer should have a minimum thickness of four times the d_{50} of the filter stone or 6 in., whichever is greater.
- Both riprap and granular filter thickness should be increased by 50% when placing under water.
- Granular filters are not recommended when dune-type bed forms are present.

4.2.3 Partially Grouted Riprap and Geocontainers

Partially grouted riprap consists of specifically sized rocks that are placed around a pier and grouted together with grout

filling 50% or less of the total void space. In contrast to fully grouted riprap, partial grouting increases the overall stability of the riprap installation unit without sacrificing flexibility or permeability. It also allows for the use of smaller rock compared to standard riprap, resulting in decreased layer thickness. **Tests conducted under NCHRP Project 24-07(2) confirm the applicability of partially grouted riprap as a scour countermeasure for bridge piers.**

Design

- Design guidance for partially grouted riprap comes, primarily, from the BAW in Germany.
- The intent of partial grouting is to “glue” stones together to create a conglomerate of particles. Each conglomerate is therefore significantly greater than the d_{50} stone size and typically is larger than the d_{100} size of the individual stones in the riprap matrix.
- For practical placement in the field, riprap having a d_{50} smaller than 9 in. (230 mm) exhibits voids that are too small for grout to effectively penetrate to the required depth within the rock matrix.
- At the other extreme, riprap having a d_{50} greater than 15 in. (380 mm) has voids that are too large to retain the grout and does not have enough contact area between stones to effectively glue them together.
- An appropriate riprap gradation is required to provide adequate void size.
- With partially grouted riprap, there are no relationships *per se* for selecting the size of rock, other than the practical considerations of proper void size and adequate stone-to-stone contact area, cited above.

Layout

- Optimum performance of partially grouted riprap as a pier scour countermeasure was obtained when the riprap installation extended a minimum distance of one and a half times the pier width in all directions around the pier.
- Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation.
- Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create any added obstruction to the flow.
- Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.
- When used in a partially grouted application, the riprap layer should have a minimum thickness of two times the d_{50} size of the design riprap. When placement must occur under water, the thickness should be increased by 50%.

- Where the grout must be placed under water, a recommended amount of Sicotan® admixture must be included in the mix to minimize segregation and improve the “stickiness.”

Filter

- As with standard (loose) riprap, a filter layer is typically required. The filter should **not** be extended fully beneath the riprap; instead, it should be terminated two-thirds of the distance from the pier to the edge of the riprap.
- With respect to filter materials, only geotextile filters were tested with the partially grouted riprap. The use of sand-filled geocontainers composed of non-woven, needle-punched geotextile was confirmed to be an appropriate means of establishing a filter layer around a pier when placement of either standard riprap or partially grouted riprap must occur under water.

4.2.4 Articulating Concrete Block Systems

ACB systems provide a flexible armor for use as a pier scour countermeasure. These systems consist of preformed concrete units that either interlock, are held together by cables, or both. After installation is complete, the units form a continuous blanket or mat. The term “articulating” implies the ability of individual blocks of the system to conform to changes in the subgrade while remaining interconnected. Block systems are typically available in both open-cell and closed-cell varieties.

There is little field experience with the use of ACB systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for bank revetment and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. **Tests conducted under NCHRP Project 24-07(2) confirm the applicability of these systems as a scour countermeasure for bridge piers.**

Design

Results of the tests confirmed that the factor of safety method recommended in HEC-23 (Lagasse et al. 2001) is appropriate for designing ACBs for hydraulic stability at piers, provided that the extent of the armor layer, as well as the design of the filter, also follows recommended guidelines. Testing also confirmed the importance of including block placement tolerance in the factor of safety calculations.

Layout

- The optimum performance of ACBs as a pier scour countermeasure was obtained when the blocks were extended a

distance of at least two times the pier width in all directions around the pier.

- Because ACBs are essentially an erosion-resistant veneer that is one particle thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery.
- Blocks should not be placed on slopes greater than 2H:1V; when placed as pre-assembled mats, they should never be placed such that a portion of one mat lies on top of another mat.
- When dune-type bed forms, contraction scour, and/or long-term degradation are present, the armor must be sloped away from the pier in all directions such that the depth of the ACB system at its periphery is greater than the depth of the bed-form troughs, or contraction scour and degradation. In some cases, this requirement may result in blocks being placed further than two pier widths away from the pier.

Filter

- The filter underlying the ACB system should be extended fully beneath the ACBs.
- With respect to filter materials, only geotextile filters were tested with ACB systems. In most cases, granular filters are not recommended for use with ACBs because of the large open cells in typical ACB block systems.

4.2.5 Gabion Mattresses

Gabion mattresses are containers constructed of wire mesh and filled with rocks. The length of a gabion mattress is greater than its width, and the width is greater than its thickness. Diaphragms are inserted widthwise into the mattress to create compartments. Wire is typically galvanized or coated with polyvinyl chloride to resist corrosion, and either welded or twisted into a lattice. Stones used to fill the containers can be either angular rock or rounded cobbles; however, angular rock is preferred because of the higher degree of natural interlocking of the stone fill. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous armor layer.

The wire mesh allows the gabions to deform and adapt to changes in the bed while maintaining stability. Additionally, when compared to riprap, less excavation of the bed is required and smaller, more economical stone can be used. The obvious benefit of gabion mattresses is that the size of the individual stones used to fill the mattress can be smaller than stone that would otherwise be required to withstand the hydraulic forces at a pier. There is limited field experience with the use of gabion mattress systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for structures such as in-channel weirs or drop

structures, or for channel slope stabilization. **Tests conducted under NCHRP Project 24-07(2) confirm the applicability of these systems as a scour countermeasure for bridge piers.**

Design

- The guidance for pier scour applications provided in this document was developed primarily from the results of this study (NCHRP Project 24-07(2)).
- The suitability of the basic design method, which is based on the concept of permissible shear stress, was confirmed for use at bridge piers by comparing the results of this testing program with the latest version of HEC-15 (Kilgore and Cotton 2005).
- The durability of the wire mesh under long-term exposure to flow conditions specific to bridge piers has not been demonstrated; therefore, the use of gabion mattresses as a bridge pier scour countermeasure has an element of uncertainty.

Layout

- The optimum performance of gabion mattresses as a pier scour countermeasure was obtained when the mattresses were extended a distance of at least two times the pier width in all directions around the pier.
- Because gabion mattresses are essentially an erosion-resistant veneer that behaves as a unit that is one layer thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery.
- Gabion mattresses should not be placed on slopes greater than 2H:1V, nor should they be placed in a manner that causes them to lie on top of adjacent mattresses.
- When dune-type bed forms, contraction scour, and/or long-term degradation are present, the armor must be sloped away from the pier in all directions such that the depth of the gabion mattress system at its periphery is greater than the depth of the bed-form troughs, or contraction scour and degradation. In some cases, this requirement may result in mattresses being placed further than two pier widths away from the pier.
- To be effective, the gabion mattresses must be tied together using lacing wire or other types of mattress-to-mattress connectors. Field installations must use mattress-to-mattress connection materials that are at least as strong as the wire mesh composing the mattresses.

Filter

- As with riprap, the filter should only be extended two-thirds of the distance from the pier to the periphery of the gabion mattress installation.

- When a granular filter is used, the layer should have a minimum thickness of four times the d_{50} of the filter stone or 6 in., whichever is greater.
- The granular filter thickness should be increased by 50% when placing under water.
- Granular filters are not recommended when dune-type bed forms are present.

4.2.6 Grout-Filled Mattresses

Grout-filled mattresses are composed of a double layer of strong synthetic fabric, typically woven nylon or polyester, sewn into a series of pillow-shaped compartments (blocks) that are connected internally by ducts. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Adjacent mattresses are typically sewn together prior to filling with grout.

The benefits of grout-filled mattresses are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the fabric forms and pumping them with concrete grout can be performed in areas where room for construction equipment is limited. When set, the grout forms a single-layer veneer made up of a grid of interconnected blocks. The blocks are interconnected by cables laced through the mattress before the grout is pumped into the fabric form. Flexibility and permeability are important functions for pier scour countermeasures. Therefore, systems that incorporate filter points or weep holes (allowing for pressure relief through the mattress) combined with relatively small-diameter ducts (to allow grout breakage and articulation between blocks) are the preferred products.

There is limited field experience with the use of grout-filled mattresses as a scour countermeasure for bridge piers. More frequently, these systems have been used for shoreline protection, protective covers for underwater pipelines, and channel armoring where the mattresses are placed across the entire channel width and keyed into the abutments or banks.

Tests confirm that grout-filled mattresses can be effective scour countermeasures for piers under clear-water conditions. However, when dune-type bed forms were present, the mattresses were subject to both undermining and uplift, even when they were toed down below the depth of the bed-form troughs. Therefore, this study cannot support the use of these products as pier scour countermeasures under live-bed conditions when dunes may be present.

Design

- The guidance for pier scour applications provided in this document has been developed primarily from HEC-23 (Lagasse et al. 2001) and the results of NCHRP Project 24-07(2).

- The recommended design method is based on sliding stability for both rigid and flexible grout-filled mattresses.

Layout

- The optimum performance of grout-filled mattresses as a pier scour countermeasure was obtained when the mattresses were extended at least two times the pier width in all directions around the pier.
- Because these products are essentially an erosion-resistant veneer that behaves as a unit that is one layer thick, the system edges must be toed down into a termination trench to prevent undermining and uplift around its periphery.
- Where long-term degradation and contraction scour is expected at a bridge crossing, grout-filled mattresses must be sloped away from the pier in all directions such that the depth of the mattress system at its periphery is greater than the depth of anticipated contraction scour and degradation.
- Grout-filled mattresses should not be laid on a slope steeper than 2H:1V. In some cases, this limitation may result in grout-filled mattresses being placed further than two pier widths away from the pier.
- Mattresses should not be placed such that a portion of one mattress lies on top of an adjacent mattress.

Filter

- A filter layer is typically required for grout-filled mattresses at bridge piers. The filter should be extended fully beneath the system to its periphery.
- When a granular filter is used, the layer should have a minimum thickness of four times the d_{50} of the filter stone or 6 in., whichever is greater.
- The granular filter thickness should be increased by 50% when placing under water.

4.2.7 Additional Observations on Pier Scour Protection Systems

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. Although 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 occurrence 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, some progress had been made in developing selection, design, and installation guidelines for pier scour countermeasures. For example, the publication of the first edition of HEC-23 in 1997 (Lagasse et al. 1997) was a first step toward identifying, consolidating, and disseminating information on countermeasure guidance. In addition, the first phase of this study (Parker et al. 1998 and 1999) provided the initial results of laboratory and field research to evaluate the performance of pier scour countermeasures and develop design and implementation guidance.

A wide variety of countermeasures has been used to control channel instability and scour at bridge foundations. In HEC-23 (Lagasse et al. 1997) a countermeasure matrix is provided to highlight the various groups of countermeasures and to identify their individual characteristics. In the matrix, countermeasures are organized into groups based on their functionality with respect to scour and stream stability. The three main groups of countermeasures are hydraulic countermeasures, structural countermeasures, and monitoring. Hydraulic countermeasures are those designed either to modify the flow (river training) or resist erosive forces caused by the flow (armoring). Structural countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Monitoring describes activities used to facilitate early identification of potential scour problems.

When the second edition of HEC-23 was published (Lagasse et al. 2001), only structural countermeasures or monitoring options were considered to be well-suited countermeasure systems to protect against pier scour. Hydraulic countermeasures (river training and armoring) were considered to have only a secondary benefit in preventing or controlling pier scour. For armoring countermeasures this consideration was, primarily, a result of the lack of definitive design guidance at the time. This project has focused on providing that design guidance for five pier scour armoring systems.

Within the suite of pier scour armoring systems tested, riprap and partially grouted riprap provide protection by the bulk or mass of the armor layer, through the redundancy of multiple particles in a flexible, self-healing matrix underlain by an appropriate filter. From a hydraulic stability perspective, the other three armoring systems (articulating concrete blocks, and gabion and grout-filled mattresses) provide protection through an armor layer essentially one particle (or one unit) thick. This thin veneer of armor often provides economy in both materials and installation but, if not provided with an appropriate filter and adequate transition or termination details, could be subject to rapid, potentially catastrophic failure.

A review of the conclusions and recommendations outlined for each countermeasure type in the preceding sections

reveals a range of commonalities and contrasts for these systems. For example, in most cases a filter layer is essential for successful performance of all pier scour protection. However for the countermeasures that incorporate rock particles, including gabions, the filter should extend only two-thirds of the distance from the pier to the perimeter of the armor. In contrast, articulating concrete block mats and grout-filled mattresses should have a filter underlying the full extent of the armor layer. In all cases, a granular filter should not be used when dune-type bed forms are expected in sand channels (i.e., under live-bed conditions). During testing, geotextile filters generally performed well for all countermeasure types when all components of the countermeasure system were properly designed and installed. For the ACB system, granular filters are not recommended under most conditions.

Geotextile sand containers are strongly recommended as a proven technique for placing a filter under water for riprap or partially grouted riprap, and gabion and grout-filled mattresses. For the ACB systems, a conventional geotextile filter should be used because meeting placement and grading tolerances would be difficult if geotextile containers are used as a filter.

For the pier scour countermeasures consisting of a thin veneer of armor (ACBs and the mattresses), termination details and, where necessary, anchor systems play a significant role in successful performance. It should be noted that testing of the grout-filled mattresses in both rigid and flexible configurations yielded definitive results only for clear-water conditions. More research will be required before this countermeasure can be recommended for pier scour protection under live-bed conditions. Similarly, the gabion mattress countermeasure, as tested, performed much better when the individual mattresses were physically connected to one another, compared to their performance as individual armor elements. However, laboratory testing could not provide guidance for the strength, composition, or longevity of the connecting material. For all three of the manufactured systems, the product provider should supply appropriate test results along with installation and materials guidance. This information is essential for successful performance of these products.

4.2.8 Countermeasure Selection

The countermeasure selection methodology developed as part of this study provides an assessment of the suitability of each of six specific countermeasure types based on a variety of factors. The output from the selection method provides a quantitative ranking of countermeasure types by computing a Selection Index. The Selection Index includes a fatal-flaw mechanism to identify situations where a particular countermeasure is unequivocally unsuitable due to one or more circumstances unique to the site being evaluated. The Selection

Index is intended to identify the countermeasure best suited for application at a particular site (see Appendix B). However, there is no substitute for experience and engineering judgment in countermeasure selection. The Selection Index should be considered only one indication of countermeasure suitability for site-specific conditions. The interactive Microsoft® Excel spreadsheet (available on the TRB website: http://www.trb.org/news/blurbs_detail.asp?id=7998) will be helpful in applying the selection methodology and adapting it to site-specific conditions.

4.2.9 Design Guidelines

To guide the practitioner in developing appropriate designs and ensuring successful installation and performance of pier scour armoring systems, the findings of Chapter 2 and recommendations of Chapter 3 are combined to provide a detailed set of stand-alone appendixes:

- Appendix C, Guidelines for Pier Scour Countermeasures Using Rock Riprap
- Appendix D, Guidelines for Pier Scour Countermeasures Using Partially Grouted Riprap
- Appendix E, Guidelines for Pier Scour Countermeasures Using Articulating Concrete Block (ACB) Systems
- Appendix F, Guidelines for Pier Scour Countermeasures Using Gabion Mattresses
- Appendix G, Guidelines for Pier Scour Countermeasures Using Grout-Filled Mattresses

These application guidelines are presented in a format using the FHWA's HEC-23 as a guide. As appropriate, these guidelines are recommended for consideration by AASHTO, FHWA, and state DOTs for adoption and incorporation into manuals, specifications, or other design guidance documents.

4.3 Suggested Research

The findings of Chapter 2 and the interpretation and appraisal of testing results in Chapter 3 are reflected in the recommended design methods, materials, and construction and inspection guidance presented in Appendixes C through G. In developing these guidelines, additional information, data, or field experience with various countermeasure systems would have supported more detailed guidance or specificity in several areas. The following suggestions for future research would permit extending the recommendations of this study in these areas:

- Tests of simulated grout-filled mattresses at small scale indicated that, in the presence of dune-type bed forms, both flexible and rigid systems were vulnerable to undermining and

uplift. Voids beneath these systems were noted even when the periphery was toed down below the depth of the bed-form troughs. It is suggested that these systems be investigated at near-prototype scale in the laboratory, and/or prototype scale at appropriate field sites to provide additional insight on their performance under live-bed conditions.

- Limited testing of wall-type piers skewed to the flow direction confirmed that the lateral extent of armoring countermeasures must be increased for them to perform successfully. Recommendations provided in this study have been inferred based on consideration of the additional scour potential caused by the skew. It is suggested that this subject be further investigated to either verify or modify the recommendations for skewed piers developed in this study.
- Water quality measurements were taken during the placement of partially grouted riprap in flowing water under prototype-scale conditions in the laboratory. The data indicate that transient increases in pH, turbidity, and electroconductivity occur as grout is being placed, and for a short time afterwards. This impact was limited to a local area downstream of the installation and may be within acceptable limits as established by permit agencies. Very little evidence of lateral dispersion was noted in the laboratory study. A limited number of candidate bridge sites could be identified for the installation of partially grouted riprap in the field under a wider variety of hydraulic and riverine conditions than could be investigated in the laboratory to verify the magnitude and extent of potential water quality impacts and to identify methods of mitigation and control, if such are needed.
- Establishing a seal between the countermeasure and the pier has been noted as a necessary component of any installation to prevent the winnowing of bed material through gaps in the region of high turbulence and vortex action at the pier. The placement of a thin grout seal around the pier proved effective at preventing winnowing during the laboratory tests and can be performed under water if necessary in the field. However, many other materials may be suitable for creating an effective seal, for example, small sand-filled geotextile tubes, asphaltic mastic, or geotextile collars. Additional techniques and materials should be investigated to expand the options for creating an effective seal between the pier and the scour countermeasure.
- Physically attaching a scour countermeasure (such as a cabled articulating concrete block system, gabion mattresses, or grout-filled mattresses) to a bridge pier is often suggested as a means to increase anchorage and stability of the countermeasure. Recommendations made in this study discourage physical attachment between countermeasure and pier; however, because it is a common practice, the potential for increased loading from pier scour countermeasures on the

bridge structure should be investigated, particularly in the case of countermeasure failure.

- The permeability, flexibility, and long-term durability of pier scour countermeasures are identified as beneficial characteristics. In this study, these specific properties were neither quantitatively measured nor related to countermeasure performance. Instead, inferences regarding these aspects were drawn from the literature, as well as from qualitative observations made during the testing program. The following observations point to the need for future research:

- Flexibility, abrasion resistance, and resistance to damage from impact or debris snagging on prototype-size gabion mattresses should be investigated more fully. Both welded wire and twisted wire products should be examined, along with various coatings that are currently commercially available for use with these systems.
- The ability of commercially available grout-filled mattresses to fully articulate to accommodate edge scour, undermining, or differential settlement has not been adequately demonstrated. Also, the overall permeability of the system is not well characterized, and the lack of adequate permeability may have resulted in uplift-type failures observed in the laboratory-scale tests. Further research is suggested in these areas.

- Open-cell articulated concrete blocks are generally assumed to have adequate permeability by virtue of the open cells, and to be durable, provided the concrete mix is well designed and quality controlled. The use of systems composed of solid blocks that have very little open area may be questionable because of a marked decrease in permeability. In addition, the ability of some proprietary products to fully articulate has been questioned, largely because of their “jigsaw puzzle” type of mechanical block-to-block interlock. Further research in these areas could resolve these uncertainties.
 - Improved predictive methods should be developed for quantifying dune bed-form geometry, as well as for providing practitioners a reliable method for recognizing the conditions under which the onset of dunes can be anticipated. Also, the potential interaction between bed forms and contraction scour should be investigated to determine if these processes are independent or additive at the prototype scale.
 - ACB systems, gabion mattresses, and grout-filled mattresses were all observed to act as a scour-resistant veneer that behaves as a unit that is one layer thick. The prevention of voids beneath these systems is essential to their successful performance. Further research under a wider variety of conditions than could be accomplished under this study is warranted.
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CHAPTER 5

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APPENDIX A

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APPENDIX B

Countermeasure Selection Methodology

The selection methodology provides a quantitative assessment of the suitability of six armoring-type countermeasures based on selection factors that consider river environment, construction considerations, maintenance, performance, and estimated life-cycle cost. With the exception of life-cycle costs, the methodology analyzes the design factors by stepping the user through a series of decision branches, ultimately resulting in a site-specific numerical rating for each selection factor. The following countermeasures are evaluated by this methodology:

- Standard (loose) riprap
- Partially grouted riprap
- Articulating concrete blocks
- Gabion mattresses
- Grout-filled mattresses
- Grout-filled bags

To facilitate the decision-making process, the procedure was automated using a Microsoft® Excel spreadsheet format. In the spreadsheet, the decision-making process can easily be modified to consider new situations or include additional information. Detailed directions are included in the program file, and automated features are incorporated in the program to step the user through the process.

Five factors are used to compute a Selection Index (SI) for each countermeasure:

- 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

$$SI = (S1 \times S2 \times S3 \times S4) / LCC$$

The countermeasure that has the highest value of SI is considered to be most appropriate for a given site, based not only

on its suitability to the specific riverine and project site conditions, but also in consideration of its economy. The approach is sensitive to assumptions regarding initial construction cost, remaining service life, assumed frequency of maintenance events, and extent of maintenance required. Each of these factors requires experience and engineering judgment, as well as site- or region-specific information on the cost of materials and delivery, construction practices, and prevailing labor rates. *It should be noted that the methodology can be used simply to rank the countermeasures in terms of suitability alone by assuming that the life-cycle costs are the same for all countermeasures.*

The following sections describe the five factors that compose the methodology. Flowcharts illustrating selection factors S1 through S4 are enclosed.

Bed Material

Bed material is included as a selection factor for two reasons. Abrasion caused by the transport of coarse bed sediments will cause the wire mesh on a gabion mattress to weaken and break, whereas other countermeasure types are relatively resistant to degradation by abrasion. For this reason, when bed material is greater than 2 mm, gabion mattresses are eliminated from the selection process. Bed material size also assists in distinguishing whether dune-type bed forms are anticipated. Grout-filled mattresses are susceptible to failure in the presence of bed forms because the mattresses do not articulate as well as other countermeasures. When the bed material is less than 2 mm and bed forms are not anticipated, all countermeasures included in the selection process are deemed equally viable.

Ice and/or Debris Loading

Debris in this context is considered floating material such as logs, other woody materials, man-made materials that are typically transported during floods, or ice. The intent of this selection factor is to recognize that high debris loads can be

detrimental to gabion mattresses, as indicated in the countermeasure selection matrix of HEC-23 (Lagasse et al. 2001). When a user indicates that anticipated debris loading is high, gabion mattresses receive a low rating of “1” but are not eliminated from the selection process. When debris loading is not anticipated, all countermeasures included in the selection process are deemed equally viable.

Construction Constraints

Construction constraints take into account the different needs and challenges required for placing a countermeasure in the dry versus installation under water or, in the extreme case, in flowing water. All ratings that consider construction constraints are divided into two categories: piers that have shallow footings versus piers that are more deeply embedded. This categorization is necessary because riprap-based countermeasures are typically thicker than alternative countermeasures and they require pre-excavation that may undermine the footer.

In addition, the requirements for specialized equipment are addressed. For example, the equipment requirements, placement techniques, and construction QA/QC requirements for partially grouted riprap are straightforward for working in the dry; however, placement under water requires construction equipment and placement technologies that are much more sophisticated. Subgrade preparation requirements and placement tolerances also vary among countermeasure types. For example, a relatively thin veneer of articulating concrete blocks requires finer grading techniques than an equivalent, and much thicker, riprap layer.

Working beneath a bridge deck that affords little headroom will dictate the type of equipment that can be used for countermeasure installation. Last, alternative placement techniques, particularly for rock riprap, typically dictate the strength requirements for geotextiles to meet criteria for geotextile survivability during installation.

The decision box for flow velocity is intended to reflect the relative difficulty in placing a mattress system, such as ACBs, gabion mattresses, or grout mattresses under fast-flowing water ($V > 4$ ft/s). When the countermeasure does not need to be placed under water and access for construction equipment of all types is good, all countermeasures included in the selection process are deemed equally viable.

Inspection and Maintenance

Inspection and maintenance guidelines vary greatly among countermeasure types. Underwater or buried installations require different considerations to ensure that the countermeasure can be adequately inspected, compared to surficial treatments in ephemeral or intermittent stream environments. The numerical values assigned to this selection factor

reflect the relative difficulty of repairing and/or replacing “manufactured” countermeasures, such as ACBs, gabion mattresses, and grout mattresses, versus the relative ease of adding more riprap stone.

The maintenance required for gabion mattresses as presented may be somewhat higher than for other forms of revetment because the wire mesh used to construct the gabion is susceptible to vandalism. When the countermeasure can be inspected and maintenance performed in the dry, all countermeasures included in the selection process are deemed equally viable.

Life-Cycle Costs

The Selection Index calculation is similar to the “Risk Priority Number” method suggested by Johnson and Niezgoda (2004). Johnson and Niezgoda use the concept of “risk categories” in contrast to this selection methodology concept of “suitability categories” to relate various factors. Both methods represent relatively simple techniques for selecting pier scour countermeasures. However, because of the complexity of determining costs associated with countermeasure design and implementation, Johnson and Niezgoda discussed life-cycle costs but did not include those costs in the scope of their procedure.

Without consideration of life-cycle cost, the suitability of a countermeasure is dictated solely by the environment of the river and its interaction with the bridge structure, combined with the strengths and vulnerabilities of the countermeasure. This selection methodology attempts to simplify the life-cycle cost estimation process through a series of spreadsheets that assist the user in evaluating regional availability of materials, installation expenses, and an estimation of maintenance based on experience and engineering judgment.

Life-cycle cost information can be difficult to quantify. Initial construction costs are relatively easy to develop; however, even for a specific countermeasure, these costs can vary widely depending on regional availability of materials, site conditions, and access or constructability constraints. Therefore, a particular countermeasure might be very cost effective in one locale and prohibitively expensive in another. Extending these issues to life-cycle maintenance requires an even broader set of assumptions. This portion of the assessment attempts to ease this process for the practitioner by providing templates for cost estimation.

Estimating life-cycle costs for pier scour countermeasures requires consideration of three major components:

- Initial construction materials and delivery costs
- Initial construction installation costs associated with labor and equipment
- Periodic maintenance during the life of the installation

Each of the above components comprises multiple elements, which differ among the various countermeasure types. For example, quantities and unit costs of alternative materials will vary depending on the specific project conditions, as well as local and regional factors. Experience with these factors, as well as project-specific knowledge of the bridge site, are required in order to be as accurate as practicable when using this selection methodology.

The following Issues should be considered when developing life-cycle cost estimates:

- Availability of materials of the required size and weight
- Haul distance
- Site access
- Equipment requirements for the various countermeasures being considered
- Construction under water vs. placement in the dry
- Environmental and water quality issues and permitting requirements
- Habitat and/or migration issues for threatened and endangered species
- Traffic control during construction and/or maintenance activities
- Local labor rates
- Construction using in-house resources versus outside contract
- Design life of the installation
- Anticipated frequency and extent of periodic maintenance and repair activities

Quantifying each of these factors requires experience and engineering judgment. For this reason, these variables are user inputs in the life-cycle cost worksheets. *The default values that are provided in the Excel spreadsheet program can and should be changed by the user to reflect both site-specific and state or regional conditions.*

Additional Considerations

Federal or state regulations that preclude the use of a particular countermeasure because of environmental considerations and permitting issues are beyond the scope of NCHRP Project 24-07(2). The practitioner in any particular state must be aware of circumstances that may warrant the exclusion of a countermeasure for consideration at a specific site.

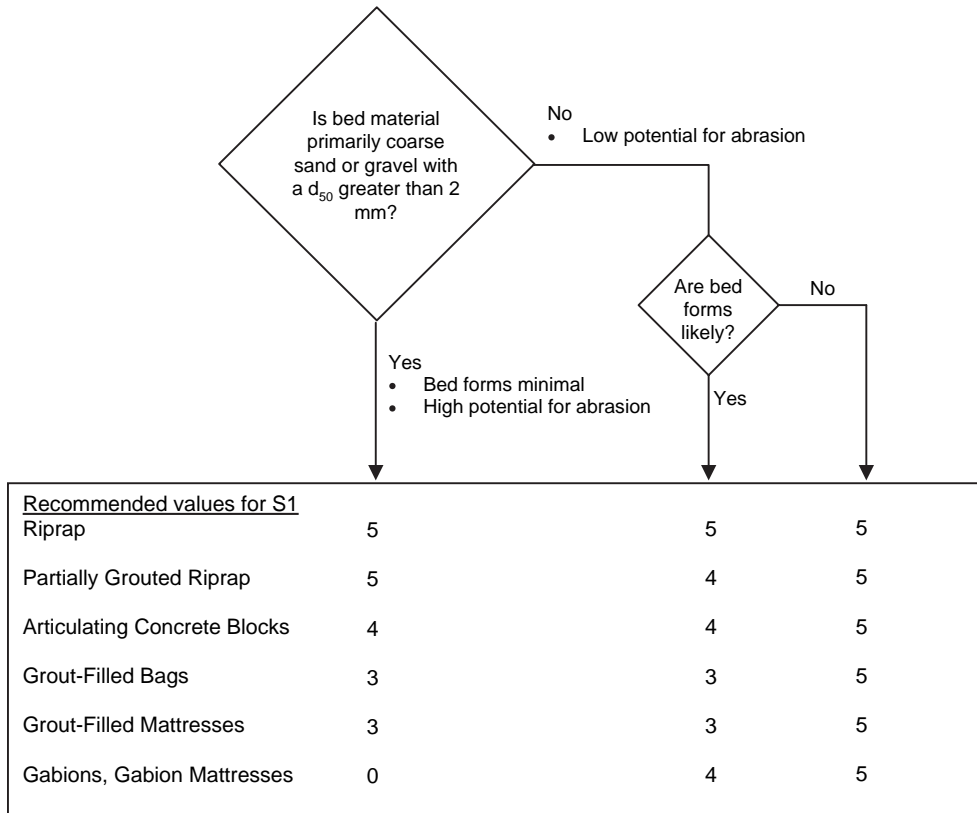
A feature allowing the user to easily include an additional design consideration, such as state-specific environmental concerns, to the computation of the Selection Index was added to the Excel-based selection methodology program. Inclusion of an additional selection criterion will require the user to assign values in the context of the selection factors for all countermeasures considered.

In addition, a feature was added to the selection methodology Excel spreadsheet capability to permit a user to introduce another countermeasure and generate selection factor values for that countermeasure. Inclusion of an additional countermeasure will require the user to assign values in the context of the design considerations and selection factors. The supplementary countermeasure feature and design consideration feature can be used independently or together, as described in the countermeasure selection Excel file available on the TRB website (http://trb.org/news/blurb_detail.asp?id=7998).

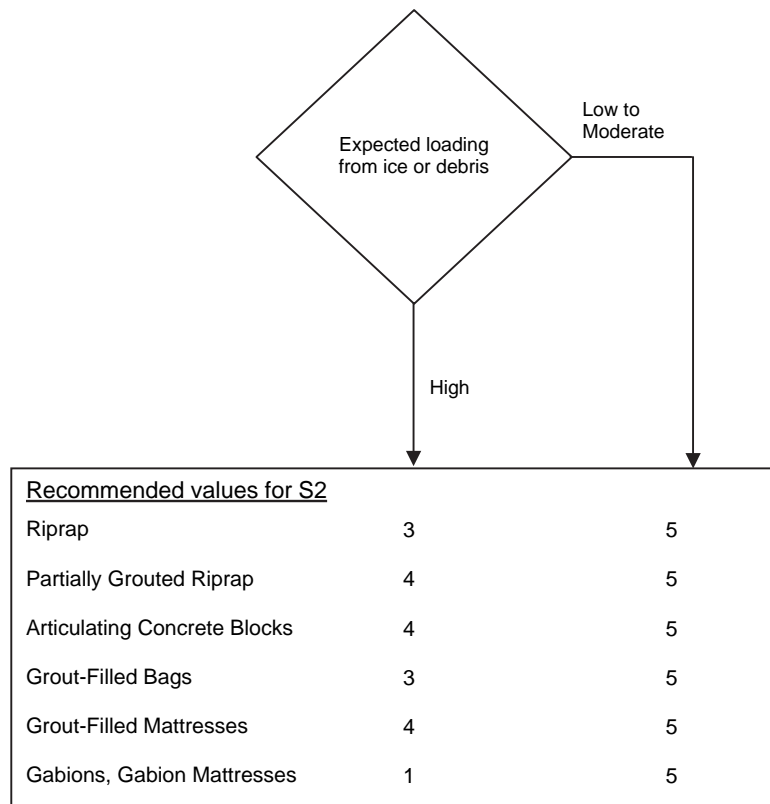
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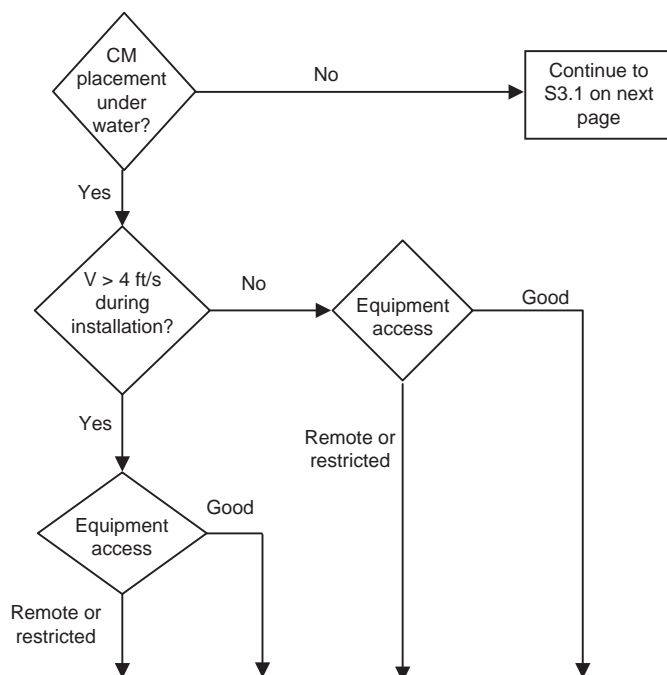
Factor S1: Bed Material



Factor S2: Ice/Debris Load



Factor S3: Construction Considerations



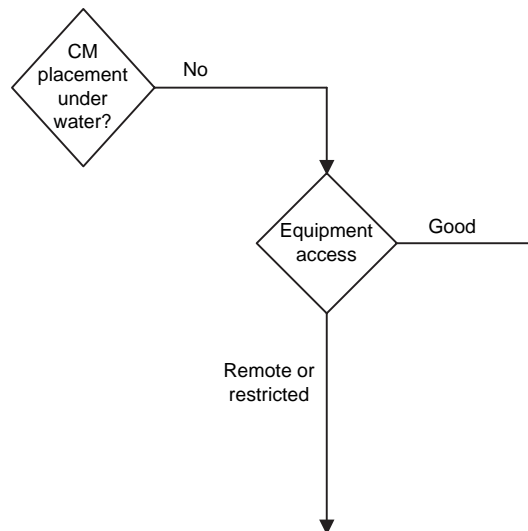
<u>Recommended values for S3</u>	<u>SF*</u>		<u>DF</u>		<u>SF</u>		<u>DF</u>		<u>SF</u>		<u>DF</u>	
	SF	DF	SF	DF	SF	DF	SF	DF	SF	DF		
Riprap	0	2	1	5	1	3	2	5				
Partially Grouted Riprap	0	0	0	0	2	4	0	5				
Articulating Concrete Blocks	0	0	1	1	2	2	0	4				
Grout-Filled Bags	0	1	1	2	1	3	1	5				
Grout-Filled Mattresses	0	0	0	0	3	3	0	4				
Gabions, Gabion Mattresses	0	0	0	1	1	1	0	3				

*Note: Armoring countermeasures not recommended for these conditions.

SF = Shallow Pier, e.g. Spread Footing DF = Deep Footing

Factor S3.1: Construction Considerations

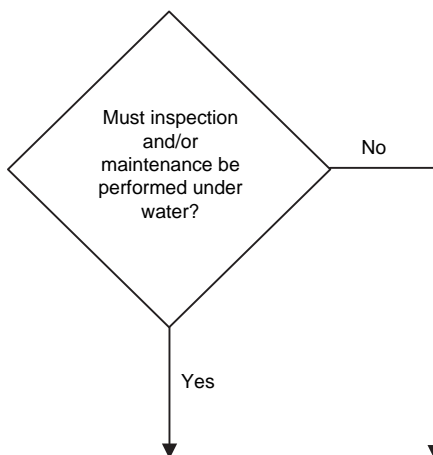
No Underwater Placement



<u>Recommended values for S3</u>	<u>SF</u>	<u>DF</u>	<u>SF</u>	<u>DF</u>
Riprap	1	3	1	5
Partially Grouted Riprap	2	4	2	5
Articulating Concrete Blocks	2	3	5	5
Grout-Filled Bags	1	4	1	5
Grout-Filled Mattresses	3	4	5	5
Gabions, Gabion Mattresses	1	3	2	5

SF= Shallow Pier, e.g. Spread Footing DF= Deep Footing

Factor S4: Inspection and Maintenance



<u>Recommended values for S4</u>		
Riprap	5	5
Partially Grouted Riprap	4	5
Articulating Concrete Blocks	3	5
Grout-Filled Bags	2	5
Grout-Filled Mattresses	2	5
Gabions, Gabion Mattresses	1	5



APPENDIX C

Guidelines for Pier Scour Countermeasures Using Rock Riprap

Introduction, C-2

1 Design and Specification, C-2

2 Construction, C-13

3 Inspection, Maintenance, and Performance Evaluation, C-18

References, C-24

Introduction

When properly designed and used for erosion protection, riprap has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. This design guideline considers the application of riprap as a pier scour countermeasure.

Design of a pier scour countermeasure system using riprap requires knowledge of river bed and foundation material; flow conditions including velocity, depth and orientation; pier size, shape, and skew with respect to flow direction; riprap characteristics of size, density, durability, and availability; and the type of interface material between the riprap and underlying foundation. The system typically includes a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention.

The guidance provided in this document for pier protection applications of riprap has been developed primarily from FHWA Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al. 2001) and the results of NCHRP Project 24-07(2) (Lagasse et al. 2007), NCHRP Project 24-23 (Lagasse et al. 2006), and NCHRP Project 24-07 (Parker et al., 1998).

This document is organized into three parts:

- Part 1 provides design and specification guidelines for riprap systems.
- Part 2 presents construction guidelines.
- Part 3 provides guidance for inspection, maintenance, and performance evaluation of riprap used as a pier scour countermeasure.

Part 1: Design and Specification

1.1 Materials

1.1.1 Size, Shape, and Density

Riprap design methods typically yield a required size of stone that will result in stable performance under the design loadings. Because stone is produced and delivered in a range of sizes and shapes, the required size of stone is often stated in terms of a minimum allowable representative size. For pier scour protection, the designer specifies a minimum allowable d_{50} for the rock composing the riprap, thus indicating the size for which 50% (by weight) of the particles are smaller. Stone sizes can also be specified in terms of weight (e.g., W_{50}) using an accepted relationship between size and volume, and the known (or assumed) density of the particle.

Shape. The shape of a stone can be generally described by designating three axes of measurement: major, intermediate, and minor, also known as the “A, B, and C” axes, as shown in Figure C1.1.

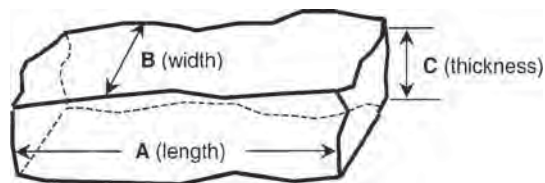


Figure C1.1. Riprap shape described by three axes.

Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying a maximum allowable value for the ratio A/C , also known as the shape factor, provides a suitable measure of particle shape, since the B axis is intermediate between the two extremes of length A and thickness C . A maximum allowable value of 3.0 is recommended:

$$\frac{A}{C} \leq 3.0 \quad (\text{C1.1})$$

For riprap applications, stones tending toward subangular to angular are preferred, due to the higher degree of interlocking, hence greater stability, compared to rounded particles of the same weight.

Density. A measure of density of natural rock is the specific gravity S_g , which is the ratio of the density of a single (solid) rock particle γ_s to the density of water γ_w :

$$S_g = \frac{\gamma_s}{\gamma_w} \quad (\text{C1.2})$$

Usually, a minimum allowable specific gravity of 2.5 is required for riprap applications. Where quarry sources uniformly produce rock with a specific gravity significantly greater than 2.5 (such as dolomite, $S_g = 2.7$ to 2.8), the equivalent stone size can be substantially reduced and still achieve the same particle weight gradation.

Size and weight. Based on field studies, the recommended relationship between size and weight is given by

$$W = 0.85(\gamma_s d^3) \quad (\text{C1.3})$$

where

W = Weight of stone, lb (kg)

γ_s = Density of stone, lb/ft³ (kg/m³)

d = Size of intermediate (“B”) axis, ft (m)

Table C1.1 provides recommended gradations for ten standard classes of riprap based on the median particle diameter d_{50} as determined by the dimension of the intermediate (“B”) axis. These gradations were developed under NCHRP Project 24-23, “Riprap Design Criteria, Specifications, and Quality Control” (Lagasse et al. 2006). The proposed gradation criteria are based on a nominal or “target” d_{50} and a uniformity ratio d_{85}/d_{15} that results in riprap that is well graded. The target uniformity ratio is 2.0 and the allowable range is from 1.5 to 2.5.

Table C1.1. Size gradations for 10 standard classes of riprap.

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Size	Min	Max	Min	Max	Min	Max	Max
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Based on Equation C1.3, which assumes the volume of the stone is 85% of a cube, Table C1.2 provides the equivalent particle weights for the same ten classes, using a specific gravity of 2.65 for the particle density.

1.1.2 Recommended Tests for Rock Quality

Standard test methods relating to material type, characteristics, and testing of rock and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are provided in this section and are recommended for specifying the quality of the riprap stone. In general, the test methods recommended in this section are intended to ensure that the stone is dense and durable, and will not degrade significantly over time.

Rocks used for riprap should break only with difficulty, have no earthy odor, not have closely spaced discontinuities (joints or bedding planes), and not absorb water easily. Rocks composed of appreciable amounts of clay, such as shales, mudstones, and claystones, are never acceptable for use as riprap. Table C1.3 summarizes the recommended tests and allowable values for rock and aggregate.

1.2 Hydraulic Stability Design Procedure

To determine the required size of stone for riprap at bridge piers, NCHRP Project 24-23 recommends using the rearranged Isbash equation from HEC-23 to solve for the median stone diameter:

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g} \quad (C1.4)$$

where

d_{50} = Particle size for which 50% is finer by weight, ft (m)

V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)

S_g = Specific gravity of riprap (usually taken as 2.65)

g = Acceleration due to gravity, 32.2 ft/s² (9.81 m/s²)

It is important to note that the design conditions in the immediate vicinity of a bridge pier are more severe than the approach conditions upstream. Therefore, the *local* velocity should be used in Equation C1.4. As recommended in HEC-23, the section-average approach velocity V_{avg} must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg} \quad (C1.5)$$

Table C1.2. Weight gradations for 10 standard classes of riprap.

Nominal Riprap Class by Median Particle Weight		W_{15}		W_{50}		W_{85}		W_{100}
Class	Weight	Min	Max	Min	Max	Min	Max	Max
I	20 lb	4	12	15	27	39	64	140
II	60 lb	13	39	51	90	130	220	470
III	150 lb	32	93	120	210	310	510	1100
IV	300 lb	62	180	240	420	600	1000	2200
V	1/4 ton	110	310	410	720	1050	1750	3800
VI	3/8 ton	170	500	650	1150	1650	2800	6000
VII	1/2 ton	260	740	950	1700	2500	4100	9000
VIII	1 ton	500	1450	1900	3300	4800	8000	17600
IX	2 ton	860	2500	3300	5800	8300	13900	30400
X	3 ton	1350	4000	5200	9200	13200	22000	48200

Table C1.3. Recommended tests for rock quality.

Test Designation	Property	Allowable value	Frequency ⁽¹⁾	Comments	
AASHTO TP 61	Percentage of Fracture	< 5%	1 per 20,000 tons	Percentage of pieces that have fewer than 50% fractured surfaces	
AASHTO T 85	Specific Gravity and Water Absorption	Average of 10 pieces: $S_g > 2.5$ Absorption < 1.0%	1 per year	If any individual piece exhibits an S_g less than 2.3 or water absorption greater than 3.0%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
AASHTO T 103	Soundness by Freezing and Thawing	Maximum of 10 pieces after 25 cycles: < 0.5%	1 per 2 years	Recommended only if water absorption is greater than 0.5% and the freeze-thaw severity index is greater than 15 per ASTM D 5312.	
AASHTO T 104	Soundness by Use of Sodium Sulfate or Magnesium Sulfate	Average of 10 pieces: < 17.5%	1 per year	If any individual piece exhibits a value greater than 25%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
AASHTO TP 58	Durability Index Using the Micro-Deval Apparatus	Value > 90 > 80 > 70	Application Severe Moderate Mild	1 per year	Severity of application per Section 5.4, CEN (2002). Most riverine applications are considered mild or moderate.
ASTM D 3967	Splitting Tensile Strength of Intact Rock Core Specimens	Average of 10 pieces: > 6 MPa	1 per year	If any individual piece exhibits a value less than 4 MPa, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
ASTM D 5873	Rock Hardness by Rebound Hammer	See Note (2)	1 per 20,000 tons	See Note (2)	
Shape	Length to Thickness Ratio A/C	< 10%, $d_{50} < 24$ in < 5%, $d_{50} > 24$ in	1 per 20,000 tons	Percentage of pieces that exhibit A/C ratio greater than 3.0 using the Wolman count method (Lagasse et al., 2006)	
ASTM D 5519	Particle Size Analysis of Natural and Man-Made Riprap Materials		1 per year	See Note (3)	
Gradation	Particle Size Distribution Curve		1 per 20,000 tons	Determined by the Wolman count method (Lagasse et al., 2006), where particle size, d , is based on the intermediate (B) axis.	

(1) Testing frequency for acceptance of riprap from certified quarries, unless otherwise noted. Project-specific tests exceeding quarry certification requirements, either in performance value or frequency of testing, must be specified by the Engineer.

(2) Test results from D 5873 should be calibrated to D 3967 results before specifying quarry-specific minimum allowable values.

(3) Test results from D 5519 should be calibrated to Wolman count (Lagasse et al., 2006) results before developing quarry-specific relationships between size and weight; otherwise, assume $W = 85\%$ that of a cube of dimension d having a specific gravity of S_g .

where

V_{des} = Design velocity for local conditions at the pier (ft/s) (m/s)

K_1 = Shape factor equal to 1.5 for round-nose piers and 1.7 for square-edged piers

K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for pier near the bank in a straight reach to 1.7 for pier located in the main current of flow around a sharp bend)

V_{avg} = Section average approach velocity (Q/A) upstream of bridge (ft/s)

If the local velocity V_{local} is available from stream tube or flow distribution output from a 1-D model, or directly computed from a 2-D model, then only the pier shape coefficient should be used to determine the design velocity. The maximum local velocity is recommended since the channel could shift and the maximum velocity could impact any pier:

$$V_{des} = K_1 V_{local} \quad (C1.6)$$

Once a design size d_{50} for the riprap is established, a standard gradation class can be selected, if design criteria and economic considerations permit. Using standard sizes, the appropriate gradation can be achieved by selecting the next size larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one.

1.3 Layout Dimensions

Based on information derived primarily from NCHRP Project 24-07(2) (Lagasse et al. 2007), the optimum performance of riprap as a pier scour countermeasure was obtained when the riprap extended a distance of 2 times the pier width in all directions around the pier.

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α , which is a function of the width, a , and length, L , of the pier (or pile bents) and the skew angle, α , as given below (after Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cos \alpha + L \sin \alpha}{a} \right)^{0.65} \quad (\text{C1.7})$$

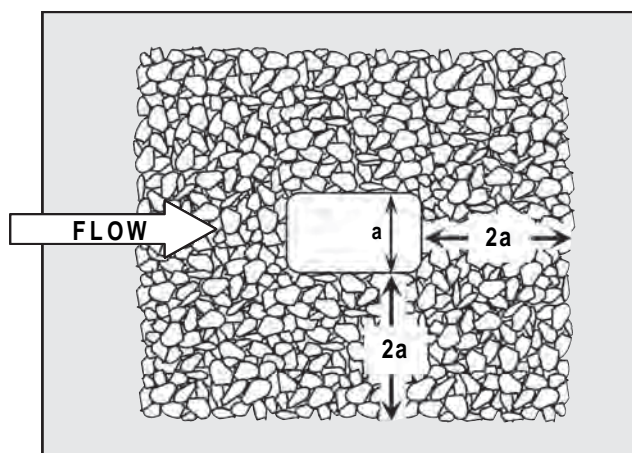
Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation. Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create any added obstruction to the flow. Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.

The riprap layer should have a minimum thickness of 3 times the d_{50} size of the rock. However, when contraction scour through the bridge opening exceeds $3d_{50}$, the thickness of the riprap must be increased to the full depth of the contraction scour plus any long-term degradation. In river systems where dune-type bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of Karim (1999) and/or van Rijn (1984). In general, an upper limit on the crest-to-trough height, Δ , is provided by Bennett (1997) as $\Delta < 0.4y$ where y is the depth of flow. This limit suggests that the maximum depth of the bed-form trough below ambient bed elevation will not exceed 0.2 times the depth of flow. Additional riprap thickness due to any of these conditions may warrant an increase in the extent of riprap away from the pier faces, such that riprap launching at a 2H:1V slope under water can be accommodated. When placement of the riprap must occur under water, the thickness should be increased by 50%. Recommended layout dimensions are provided in Figure C1.2.

A filter layer is typically required for riprap at bridge piers. The filter should *not* be extended fully beneath the riprap; instead, it should be terminated two-thirds of the distance from the pier to the edge of the riprap. When a granular stone filter is used, the layer should have a minimum thickness of 4 times the d_{50} of the filter stone or 6 in. (15 cm), whichever is greater. As with riprap, the layer thickness should be increased by 50% when placing under water. Sand-filled geotextiles made of properly selected materials provide a convenient method for controlled placement of a filter in flowing water. This method can also be used to partially fill an existing scour hole when placement must occur under water, as illustrated in Figure C1.3. For more detail, see Lagasse et al. (2007).

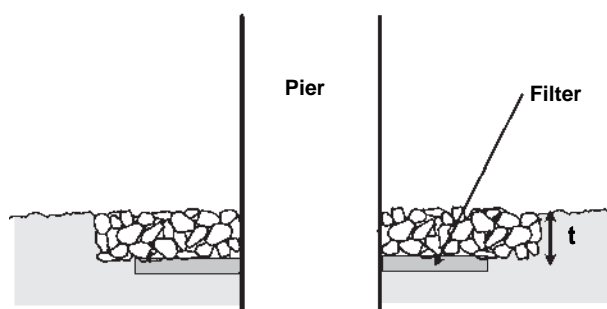
1.4 Filter Requirements

The importance of the filter component of a riprap installation should not be underestimated. Two kinds of filters are used in conjunction with riprap: granular filters and geotextile filters. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations, of the filter layer. *In cases where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.*



Pier width = "a" (normal to flow)

Riprap placement = 2 (a) from pier (all around)



Minimum riprap thickness $t = 3d_{50}$, depth of contraction scour, or depth of bedform trough, whichever is greatest

Filter placement = $4/3(a)$ from pier (all around)

Figure C1.2. Riprap layout diagram for pier scour protection.

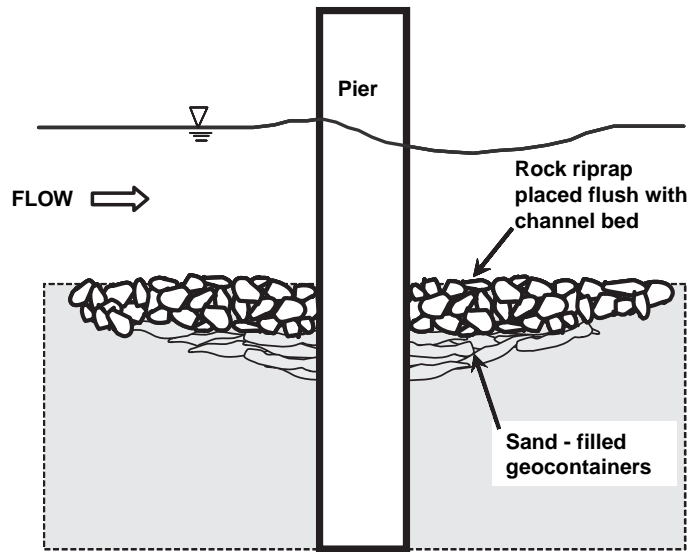
The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain *all* the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind.

1.4.1 Geotextile Filter Properties

Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.***

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand stresses during installation. These properties are readily available from manufacturers. The following list briefly describes the most relevant properties:

- **Permeability.** The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimeters per second (cm/s). This property is directly related to the filtration function that a



Minimum riprap thickness $t = 3d_{50}$, depth of contraction scour, or depth of bedform trough, whichever is greatest

Filter placement = $4/3(a)$ from pier (all around)

Figure C1.3. Schematic diagram of sand-filled geocontainers beneath pier riprap.

geotextile must perform, where water flows across the plane of the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of $(s)^{-1}$. Permeability (and permittivity) is extremely important in filter design. For pier riprap installations, the permeability of the geotextile should be at least 10 times greater than that of the underlying material:

$$K_g > 10K_s \quad (C1.8)$$

where

K_g = Permeability of geotextile (cm/s)

K_s = Permeability of subgrade soil (cm/s)

- **Transmissivity.** The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second (cm^2/s). This property is directly related to the drainage function and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas non-woven, needle-punched fabrics have a much greater capacity due to their three-dimensional (3-D) microstructure. Transmissivity is not particularly relevant to filter design.
- **Apparent opening size (AOS).** Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.
- **Porosity.** Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.

- **Percent open area (POA).** POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Thickness.** As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).
- **Grab strength and elongation.** Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.
- **Tear strength.** Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.
- **Puncture strength.** Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

Table C1.4 provides the recommended characteristics for geotextile filters. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving pier riprap have been discussed here. Geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term

Table C1.4. Recommended requirements for geotextile properties.

Test Designation	Property	Allowable value ⁽¹⁾		Comments
		Elongation < 50% ⁽²⁾	Elongation > 50% ⁽²⁾	
ASTM D 4632	Grab Strength	> 315 lbs (Class 1) > 250 lbs (Class 2) > 180 lbs (Class 3)	> 200 lbs (Class 1) > 160 lbs (Class 2) > 110 lbs (Class 3)	From AASHTO M 288
ASTM D 4632	Sewn Seam Strength ⁽³⁾	> 270 lbs (Class 1) > 220 lbs (Class 2) > 160 lbs (Class 3)	> 180 lbs (Class 1) > 140 lbs (Class 2) > 100 lbs (Class 3)	From AASHTO M 288
ASTM D 4533	Tear Strength ⁽⁴⁾	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4833	Puncture Strength	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4751	Apparent Opening Size	Per design criteria (Section 1.4 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 1.4 of this design guide)		Minimum allowable value
ASTM D 4355	Degradation by Ultraviolet Light	> 50% strength retained after 500 hours of exposure		Minimum allowable value
ASTM D 4873	Guide for Identification, Storage, and Handling			Provides information on identification, storage, and handling of geotextiles.
ASTM D 4759	Practice for the Specification Conformance of Geosynthetics			Provides information on procedures for ensuring that geotextiles at the jobsite meet the design specifications.

(1) Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class. The following descriptions have been modified from AASHTO M 288:

- *Class 1* is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including when placement of riprap must occur in multiple lifts, when drop heights may exceed 1 ft (0.3 m) or when repeated vehicular traffic on the installation is anticipated.
- *Class 2* is recommended for installation conditions where placement in regular, single lifts are expected and little or no vehicular traffic on the installation will occur, or when placing individual rocks by clamshell, orange-peel grapple or specially equipped hydraulic excavator with drop heights less than 1 ft.
- *Class 3* is specified for the least severe installation environments, with drop heights less than 1 ft onto a bedding layer of select sand, gravel or other select imported material.

(2) As measured in accordance with ASTM D 4632.

(3) When seams are required.

(4) The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 55 lbs. The MARV corresponds to a statistical measure whereby 2.5% of the tested values are less than the mean value minus two standard deviations (Koerner 1998).

endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by the riprap, these stresses do not represent the environment that the geotextile will experience in the long term.

1.4.2 Geotextile Filter Design Procedure

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. A decision tree is provided as Figure C1.4 to assist in determining the appropriate soil retention criterion for the geotextile. The figure includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater than 30% clay or is predominantly fine-grained soil (more than 50% passing the #200 sieve).

If a granular transition layer is required, the geotextile should be designed to be compatible with the properties of the granular layer. If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required. However, this requirement can be waived if the base soil exhibits the following conditions for hydraulic conductivity, K ; plasticity index, PI ; and undrained shear strength, c :

$$K < 1 \times 10^{-7} \text{ cm/s}$$

$$PI > 15$$

$$c > 10 \text{ kPa}$$

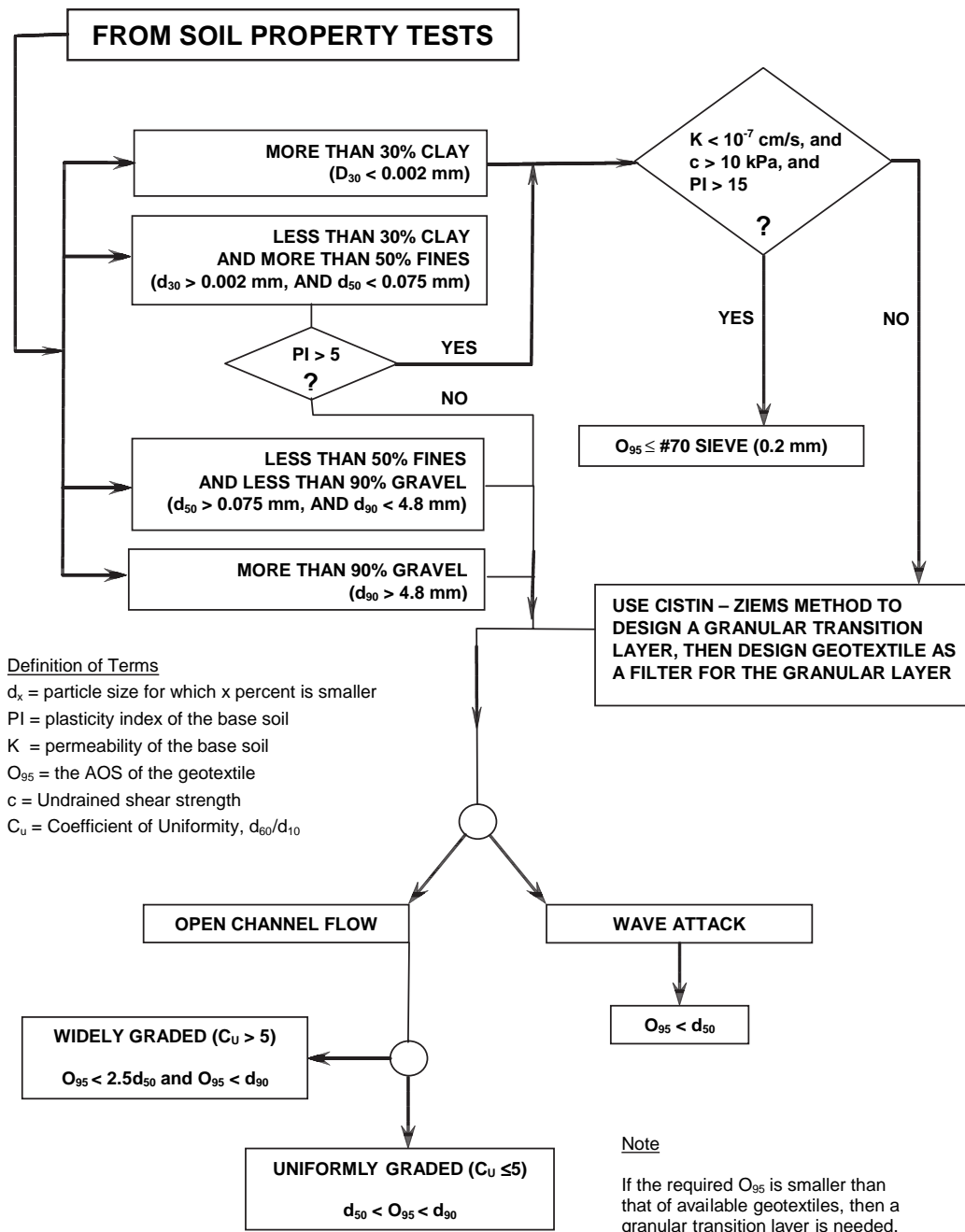
Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS less than a #70 sieve (approximately 0.2 mm) can be used with soils meeting these conditions and essentially functions more as a separation layer than a filter.

Step 3. Determine Permeability Criterion. The permeability criterion requires that the filter exhibit a permeability at least 4 times greater than that of the base soil (Koerner 1998) and for critical or severe applications, at least 10 times greater (Holtz et al. 1995). Generally speaking, if the permeability of the base soil or granular filter has been determined from laboratory testing, that value should be used. If laboratory testing was not conducted, then an estimate of permeability based on the particle size distribution should be used.

To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s^{-1} . Typically, the designer will need to contact the geotextile manufacturer to obtain values of permeability, permittivity, and thickness.

Step 4. Select a Geotextile that Meets the Required Strength Criteria. Strength and durability requirements depend on the installation environment and the construction equipment that is being used. See Table C1.4 for recommended values based on AASHTO M 288, "Geotextile Specification for Highway Construction," which provides guidance on allowable strength and elongation values for three categories of installation severity. For additional guidelines regarding the selection of durability test methods, refer to ASTM D 5819, "Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability."

Step 5. Minimize Long-Term Clogging Potential. When a woven geotextile is used, its POA should be greater than 4% by area. If a non-woven geotextile is used, its porosity should be greater than 30% by volume. A good rule of thumb suggests that the geotextile having the largest AOS that satisfies the particle retention criteria should be used (provided of course that all other minimum allowable values described in this section are met as well).



Source: modified from Koerner (1998)

Figure C1.4. Geotextile selection based on soil retention.

1.4.3 Granular Filter Properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters can be used alone or can serve as a transitional layer between a predominantly fine-grained base soil and a geotextile. The following list briefly describes the most relevant properties:

- **Particle Size Distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibbaum (2004) presents

a summary of a procedure originally developed by Cistin and Ziems whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure C1.5 provides a design chart based on the Cistin–Ziems approach.

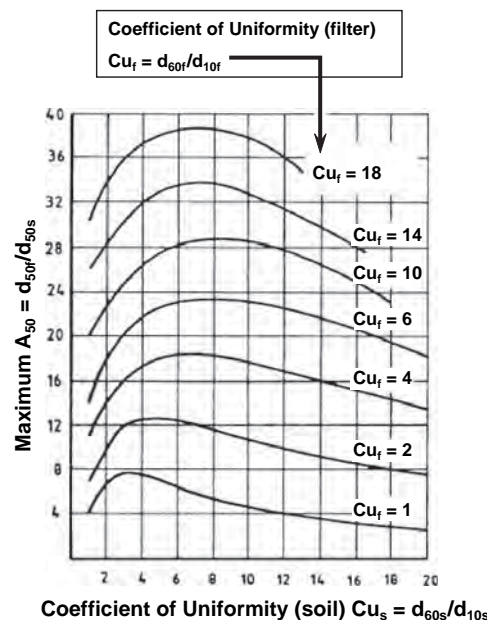
- **Permeability.** Permeability of a granular filter material is determined by laboratory test or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For pier riprap, the permeability of the granular filter should be at least 10 times greater than that of the underlying material.
- **Porosity.** Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.
- **Thickness.** Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. is specified. For placement under water, thickness should be increased by 50%.
- **Quality and durability.** Aggregate used for a granular filter should be hard, dense, and durable.

1.4.4 Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter. That approach was developed for subsoils consisting of well-graded sands and may not be widely applicable to other soil types. An alternative approach that is considered more robust in this regard is the Cistin–Ziems method.

The suggested steps for proper design of a granular filter using this method are outlined below. Note that the subscript “s” is used to represent the base (finer) soil, and “f” is used to represent the filter (coarser) layer.

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).



Source: Heibaum (2004)

Figure C1.5. Granular filter design chart according to Cistin and Ziems.

Step 2. Determine Key Indices for Base Soil. From the grain size information, determine the median grain size, d_{50} , and the coefficient of uniformity, $Cu_s = d_{60}/d_{10}$, of the base soil.

Step 3. Determine Key Indices for Granular Filter. One or more locally available aggregates should be identified as potential candidates for use as a filter material. The d_{50} and coefficient of uniformity, $Cu_f = d_{60}/d_{10}$, should be determined for each candidate filter material.

Step 4. Determine Maximum Allowable d_{50} for Filter. Enter the Cistin–Ziems design chart (Figure C1.5) with the coefficient of uniformity, Cu_s , for the base soil on the x-axis. Find the curve that corresponds to the coefficient of uniformity, Cu_f , for the filter in the body of the chart and, from that point, determine the maximum allowable A_{50} from the y-axis. Compute the maximum allowable d_{50f} of the filter using d_{50fmax} equals A_{50max} times d_{50s} . Check to see if the candidate filter material conforms to this requirement. If it does not, continue checking alternative candidates until a suitable material is identified.

Step 5. Check for Permeability. From laboratory permeameter tests or the grain size distribution of the candidate filter material, determine whether the hydraulic conductivity of the filter is at least 10 times greater than that of the subsoil.

Step 6. Check for Compatibility with Riprap Rock. Repeat steps 1 through 4 above, considering that the filter material is now the “finer” soil and the riprap is the “coarser” material. If the Cistin–Ziems criterion is not met, then multiple layers of granular filter materials should be considered.

Step 7. Filter Layer Thickness. For practicality of placement, the nominal thickness of a single filter layer should not be less than 6 in. (15 cm). Single-layer thicknesses up to 15 in. (38 cm) may be warranted where large rock fill particle sizes are used. When multiple filter layers are required, each individual layer should range from 4 to 8 in. (10 to 20 cm) in thickness (Brown and Clyde 1989)).

NOTE: In cases where dune-type bed forms may be present or of underwater installation, it is strongly recommended that only a geotextile filter be considered.

Part 2: Construction

Riprap is placed in a riverine or coastal environment to prevent scour or erosion of the bed, banks, shoreline, or near structures such as bridge piers and abutments. Riprap construction involves placement of rock and stone in layers on top of a bedding or filter layer composed of sand, gravel, and/or geotechnical fabric. The basis of the protection afforded by the riprap is the mass and interlocking of the individual rocks.

Factors to consider when designing riprap structures begin with the source for the rock; the method to obtain or manufacture the rock; competence of the rock; and the methods and equipment to collect, transport, and place the riprap. Rock for riprap may be obtained from quarries, by screening oversized rock from earth borrow pits, by collecting rock from fields, or from talus deposits. Screening borrow pit material and collecting field rocks present problems such as rocks that are too large or that have unsatisfactory length-to-width ratios for riprap.

Quarries are generally the best source for obtaining large rock specified for riprap. However, not all quarries can produce large rock because of the characteristics or limited volume of the rock formation. Because quarrying generally uses blasting to fracture the rock formation into material suitable for riprap, cracking of the large rocks may only become evident after loading, transporting, and dumping the material at the quarry or after moving the material from quarry to stockpile at the job site or from the stockpile to the final placement location.

In most cases, the production of the rock material will occur at a source that is relatively remote from the construction area. Therefore, this discussion assumes that the rock is hauled to the site of the installation, where it is either dumped directly, stockpiled, or loaded onto waterborne equipment.

The objectives of construction of a good riprap structure are (1) to obtain a rock mixture from the source that meets the design specifications and (2) to place that mixture in a well-knit, compact, and uniform layer without segregation of the mixture. The guidance in this section has been developed to facilitate the proper installation of riprap systems to achieve suitable hydraulic performance and maintain stability against hydraulic loading. The proper installation of riprap systems is essential to the adequate functioning and performance of the system during the design hydrologic event. Guidelines are provided herein for maximizing the correspondence between the design intent and the actual field-finished conditions of the project. This section addresses the preparation of the subgrade, placement of the filter, riprap placement, and measurement and payment.

2.1 General Guidelines

The contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. This responsibility is typically performed by the owner's engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see ER 1180-1-6 [U.S. Army Corps of Engineers 1995]) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for riprap placement are included in the project plans and specifications. Standard riprap specifications and layout guidance are found in Part 1 of this appendix. Recommended requirements for the stone, including the tests necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications, are provided. Field tests can be performed at the quarry and/or on the job site, or representative samples can be obtained for laboratory testing.

Typically, one or more standard riprap gradations are specified and plan sheets show locations, grades, and dimensions of rock layers for the countermeasure. The stone shape is important and riprap should be blocky rather than elongated, platy, or round. In addition, the stone should have sharp, angular, clean edges at the intersections of relatively flat surfaces.

Segregation of material during transportation, dumping, or off-loading is not acceptable. Inspection of riprap placement consists of visual inspection of the operation and the finished surface. Inspection must ensure that a dense, rough surface of well-keyed graded rock of the specified quality and sizes is obtained, that the layers are placed such that voids are minimized, and that the layers are the specified thickness.

Inspection and quality assurance must be carefully organized and conducted in case potential problems or questions arise over acceptance of stone material. Acceptance should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject stone at the quarry, at the job site or stockpile, and in place in the structures throughout the duration of the contract. Stone rejected at the job site should be removed from the project site. Stone rejected at the quarry should be disposed of or otherwise prevented from mixing with satisfactory stone.

Construction techniques can vary tremendously because of the following factors:

- Size and scope of the overall project
- Size and weight of the riprap particles
- Placement under water or in the dry

- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions
- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general information regarding construction of riprap installations and some basic information and description of techniques and processes involved.

2.2 Materials

2.2.1 Stone

The best time to control the gradation of the riprap mixture is during the quarrying operation. Generally, sorting and mixing later in stockpiles or at the construction site is not recommended. Inspection of the riprap gradation at the job site is usually carried out visually. Therefore, it is helpful to have a pile of rocks with the required gradation at a convenient location where inspectors can see and develop a reference to judge by eye the suitability of the rock being placed. On-site inspection of riprap is necessary both at the quarry and at the job site to ensure proper gradation and material that does not contain excessive amounts of fines. Breakage during handling and transportation should be taken into account.

The Wolman count method (Wolman 1954) as described in *NCHRP Report 568* (Lagasse et al. 2006) may be used as a field test to determine a size distribution based on a random sampling of individual stones within a matrix. This method relies on samples taken from the surface of the matrix to make the method practical for use in the field. The procedure determines frequency by size of a surface material rather than using a bulk sample. The middle dimension (B axis) is measured for 100 randomly selected particles on the surface.

The Wolman count method can be done by stretching a survey tape over the material and measuring each particle located at equal intervals along the tape. The interval should be at least 1 ft for small riprap and increased for larger riprap. The longer and shorter axes (A and C) can also be measured to determine particle shape. One rule that must be followed is that if a single particle is large enough to fall under two interval points along the tape, then it should be included in the count twice. It is best to select an interval large enough that this does not occur frequently.

2.2.2 Filter

Geotextile. Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). Under no circumstances may spun-bond or slit-film fabrics be allowed. Each roll of geotextile shall be labeled with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. Geotextiles shall not be exposed to sunlight prior to placement.

Granular filters. Samples of granular filter material shall be tested for grain size distribution to ensure compliance with the gradation specification used in design. Sampling and testing frequency shall be in accordance with the owner or owner's authorized representative.

2.2.3 Subgrade Soils

When the riprap and filter are placed in the dry, they shall be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Unsatisfactory soils shall be considered those soils having excessive in-place moisture content; soils containing

roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils shall be removed, and the excavation backfilled with approved material that is compacted prior to placement of the riprap. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays, per the geotechnical engineer's recommendations.

2.3 Installation

2.3.1 Subgrade Preparation

The subgrade soil conditions shall meet or exceed the required material properties described in Section 2.2.3 prior to placement of the riprap. Soils not meeting the requirements shall be removed and replaced with acceptable material.

When riprap is placed in the dry, the areas to receive the riprap shall be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the filter, and between the filter and the riprap. Stable and compacted subgrade soil shall be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes shall be compacted, shaped, and uniformly graded. The subgrade should be uniformly compacted to the geotechnical engineer's site-specific requirements.

When riprap is placed under water, divers shall be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create voids beneath the system. Immediately prior to placement of the filter and riprap system, the prepared subgrade must be inspected.

2.3.2 Placing the Filter

Whether the filter comprises one or more layers of granular material or is made of geotextile, its placement should result in a continuous installation that maintains intimate contact with the soil beneath. Voids, gaps, tears, or other holes in the filter must be avoided to the extent practicable, and repaired or the filter replaced when they occur.

Placement of Geotextile. The geotextile shall be placed directly on the prepared area, in intimate contact with the subgrade. When a geotextile is placed, it should be rolled or spread out directly on the prepared area and be free of folds or wrinkles. The rolls shall not be dragged, lifted by one end, or dropped. The geotextile should be placed in such a manner that placement of the overlying materials (riprap and/or bedding stone) will not excessively stretch or tear the geotextile.

After geotextile placement, the work area shall not be trafficked or disturbed in a manner that might result in a loss of intimate contact between the riprap stone, the geotextile, and the subgrade. The geotextile shall not be left exposed longer than the manufacturer's recommendation to minimize potential damage due to ultraviolet radiation; therefore, the overlying materials should be placed as soon as practicable.

The geotextile shall be placed so that upstream strips overlap downstream strips. Overlaps shall be in the direction of flow wherever possible. The longitudinal and transverse joints shall be overlapped at least 1.5 ft (46 cm) for dry installations and at least 3 ft (91 cm) for below-water installations. If a sewn seam is to be used for the seaming of the geotextile, the thread to be used shall consist of high-strength polypropylene or polyester and shall be resistant to ultraviolet radiation. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or weights such as sandbags shall be used.

Placing Geotextiles Under Water. Placing geotextiles under water can be problematic for a number of reasons. Most geotextiles that are used as filters beneath riprap are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, mean-

ing that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the riprap (Koerner 1998).

Flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric (a condition that contractors refer to as “galloping”) that are extremely difficult to control. The preferred method of controlling geotextile placement is to isolate the work area from river currents by a temporary cofferdam. In mild currents, geotextiles precut to length can be placed by divers, using sandbags to hold the filter temporarily.

For riprap at piers, sand-filled geocontainers made of non-woven, needle-punched fabric are particularly effective for placement under water as shown in Figure C1.3. The geotextile fabric and sand fill that compose the geocontainers should be selected in accordance with appropriate filter design criteria presented in Part 1 and placed such that they overlap to cover the required area. Geocontainers can be fabricated in a variety of dimensions and weights. Each geocontainer should be filled with sand to no more than 80% its total volume so that it remains flexible and “floppy.” The geocontainers can also serve to fill a pre-existing scour hole around a pier prior to riprap placement, as shown in Figure C1.3. For more information, see Lagasse et al. (2006, 2007).

Placement of Granular Filter. For placing a granular filter, front-end loaders are the preferred method for dumping and spreading the material on slopes milder than approximately 4H:1V. A typical minimum thickness for granular filters is 0.5 to 1.0 ft (0.15 to 0.3 m), depending on the size of the overlying riprap and whether a layer of bedding stone is to be used between the filter and the riprap. When a granular filter is placed under water, the thickness should be increased by 50%. Placing granular media under water around a bridge pier is best accomplished using a large-diameter tremie pipe to control the placement location and thickness, while minimizing the potential for segregation. *NOTE: For riverine applications where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.*

2.3.3 Placing the Riprap

Riprap may be placed from either land-based or water-based operations and can be placed under water or in the dry. Special-purpose equipment such as clamshells, orange-peel grapples, or hydraulic excavators (often equipped with a “thumb”) is preferred for placing riprap. Unless the riprap can be placed to the required thickness in one lift using dump trucks or front-end loaders, tracked or wheeled vehicles are discouraged from use because they can destroy the interlocking integrity of the rocks when driven over previously placed riprap.

Water-based operations may require specialized equipment for deep-water placement or can use land-based equipment loaded onto barges for near-shore placement. In all cases, riprap should be placed from the bottom working toward the top of the slope so that rolling and/or segregation does not occur.

Riprap Placement on Geotextiles. Riprap should be placed over the geotextile by methods that do not stretch, tear, puncture, or reposition the fabric. Equipment should be operated to minimize the drop height of the stone without the equipment contacting and damaging the geotextile. Generally, this will be about 1 ft of drop from the bucket to the placement surface (ASTM D 6825). Further guidance on recommended strength properties of geotextiles as related to the severity of stresses during installation are provided in Part 1 of this appendix. When the preferred equipment cannot be utilized, a bedding layer of coarse granular material on top of the geotextile can serve as a cushion to protect the geotextile. Material composing the bedding layer must be more permeable than the geotextile to prevent uplift pressures from developing.

Riprap Placement Under Water. Riprap placed in water requires close observation and increased quality control to ensure a continuous well-graded uniform rock layer of the required thickness (ASTM D 6825). A systematic process for placing and continuous monitoring to ver-

ify the quantity and layer thickness is important. Typically, riprap thickness is increased by 50% when placement must occur under water.

Excavation, grading, and placement of riprap and filter under water require additional measures. For installations of a relatively small scale, the stream around the work area can be diverted during the low-flow season. For installations on larger rivers or in deeper water, the area can be temporarily enclosed by a cofferdam, which allows for construction dewatering if necessary. Alternatively, a silt curtain made of plastic sheeting may be suspended by buoys around the work area to minimize environmental degradation during construction.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remote operated vehicles (ROVs) can provide some information about the riprap placement under water.

2.3.4 Inspection

The subgrade preparation, geotextile placement, riprap system installation, and overall finished condition including termination trenches shall be inspected before work acceptance. Inspection guidelines for the riprap installation are presented in detail in Part 3 of this appendix.

2.4 Measurement and Payment

Riprap satisfactorily placed can be paid for based on either volume or weight. When a weight basis is used, commercial truck scales capable of printing a weight ticket including time, date, truck number, and weight should be used. When a volumetric basis is used, the in-place volume should be determined by multiplying the area, as measured in the field, of the surface on which the riprap was placed by the thickness of the riprap measured perpendicular as dimensioned on the contract drawings.

In either case, the finished surface of the riprap should be surveyed to ensure that the as-built lines and grades meet the design plans within the specified tolerance. Survey cross sections perpendicular to the axis of the structure are usually taken at specified intervals. All stone outside the limits and tolerances of the cross sections of the structure, except variations so minor as not to be measurable, is deducted from the quantity of new stone for which payment is to be made. In certain cases, excess stone may be hazardous or otherwise detrimental; in this circumstance, the contractor must remove the excess stone at its own expense. Payment will be full compensation for all material, labor, and equipment to complete the work.

Part 3: Inspection, Maintenance, and Performance Evaluation

3.1 Inspection During Construction

Inspection during construction shall be conducted by qualified personnel who are independent of the contractor. Underwater inspection of riprap scour countermeasures at piers shall be performed only by divers specifically trained and certified for such work.

3.1.1 Subgrade

Inspection of the subgrade shall be performed immediately prior to geotextile placement. The subgrade should be clean and free of projections, debris, construction materials, or other foreign objects that would prevent the filter from being properly placed. Likewise, there should be no potholes, rills, or other voids that the filter material might bridge over.

The subgrade material itself should not be muddy or frozen and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area shall be noted and photographed; observations of such should be brought to the attention

of the project engineer as they may represent conditions that are different than those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

3.1.2 Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product designation. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics and slit-film geotextiles should **never** be used in riprap applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, extra concrete blocks, or U-shaped soil staples may be used to hold the geotextile in position while the blocks are being placed. The riprap should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

3.1.3 Riprap

Inspection of riprap placement typically consists of visual inspection of the operation and the finished surface. Inspection must ensure that a dense, rough surface of well-keyed graded rock of the specified quality and sizes is obtained, that the layers are placed such that voids are minimized, and that the layers are the specified thickness.

3.2 Periodic and Post-Flood Inspection

Pier riprap is typically inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a specified magnitude.

Underwater inspection of a riprap system shall be performed only by divers specifically trained and certified for such work. The following guidance for inspecting riprap is presented in the National Highway Institute (NHI) training course 135047, "Stream Stability and Scour at Highway Bridges for Bridge Inspectors":

1. Riprap should be **angular and interlocking**. (Old bowling balls would not make good riprap. Flat sections of broken concrete paving do not make good riprap.)
2. Riprap should have a **granular or synthetic geotextile filter** between the riprap and the sub-grade material.
3. Riprap should be **well graded** (a wide range of rock sizes). The maximum rock size should be no greater than about twice the median (d_{50}) size.
4. For bridge piers, riprap should generally extend up to the bed elevation so that the top of the riprap is visible to the inspector during and after floods.
5. When riprap is inspected, affirmative answers to the following questions are strong indicators of problems:
 - Has riprap been **displaced** downstream?
 - Has angular riprap blanket **slumped** down slope?
 - Has angular riprap material been **replaced** over time by smoother river run material?
 - Has riprap material physically **deteriorated, disintegrated**, or been **abraded** over time?
 - Are there **holes** in the riprap blanket where the filter has been exposed or breached?

3.3 Inspection Coding Guide

To guide the inspection of a riprap installation, a coding system was developed under NCHRP Project 24-23 (Lagasse et al. 2006). Similar to the National Bridge Inspection Standards (NBIS)

(U.S.DOT 2004) Item 113, it establishes numerical ratings from 0 (worst) to 9 (best). Recommended action items based on the numerical rating are also provided. This coding system is applicable to any riprap installation, including bridge pier protection.

3.4 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. As with any armor system, progressive failure from successive flows must be avoided by providing timely maintenance intervention.

3.5 Performance Evaluation

The evaluation of any riprap system's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. To properly assess the performance of pier riprap, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the riprap. Present-day channel cross-section geometry and planform should be compared to those at the time of installation. Both lateral and vertical instability of the channel can significantly alter hydraulic conditions at the site. Approach flows may exhibit an increasingly severe angle of attack (impinging flow) over time, increasing the hydraulic loading on the riprap.

It is recognized that the person making the performance evaluation will probably not be the inspector; however, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported as a percentage of the initial capital improvement cost.

3.5.1 Performance Rating Guide

To guide the performance evaluation for riprap as a pier scour countermeasure, a rating system is presented in this section. It establishes numerical ratings from 0 (worst) to 6 (best) for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the present physical condition of the countermeasure?

Tables C3.1 through C3.3 present a rating system for riprap used as a pier scour countermeasure. A single numerical score is not intended; rather, an independent rating (0-6 or U) is given

Table C3.1. Rating system for riprap: hydraulic history.

Code	Hydraulic History	Code	Hydraulic History
U	N/A	3	Moderate: The countermeasure has experienced one or more flows greater than the 10-year event.
6	Extreme: The countermeasure has experienced one or more flows greater than the 100-year event.	2	Low: The countermeasure has experienced one or more flows greater than the 5-year event.
5	Severe: The countermeasure has experienced one or more flows greater than the 50-year event.	1	Very Low: The countermeasure has experienced one or more flows greater than the 2-year event.
4	High: The countermeasure has experienced one or more flows greater than the 25-year event.	0	Negligible: The countermeasure has not experienced any flows greater than a 2-year event.

Table C3.2. Rating system for riprap: maintenance history.

Code	Maintenance History	Code	Maintenance History
U	N/A	3	Moderate: The system has required occasional maintenance since installation.
6	None Required: No maintenance has been needed since installation.	2	High: Frequent maintenance has been required.
5	Very Low: The system has required maintenance for very small, local areas once or twice.	1	Very High: Significant maintenance is usually required after flood events.
4	Low: The system has required minor maintenance.	0	Excessive: The system typically requires maintenance every year.

Table C3.3. Rating system for riprap: current condition.

Code	Description of Current Condition	Code	Description of Current Condition
U	The system is uninspectable, due to burial by sediment, debris, or other circumstance.	3	Fair: Some missing particles as evidenced by irregular armor surface; localized areas exhibit decreased layer thickness.
6	Excellent: The system is in excellent condition, with no displacement of particles and no undermining. System is well abutted to pier with no gaps.	2	Poor: Obvious deterioration of the system has occurred. Gaps or holes are present that have exposed the underlying filter.
5	Very Good: The system exhibits only minimal evidence of settlement or particle movement around the periphery.	1	Badly Damaged: The system has experienced substantial deterioration in terms of particle displacement. The armor layer has separated from the pier, leaving gaps.
4	Good: Some minor settlement and/or particle displacement observed.	0	Severe: The system has suffered damage such that it is no longer providing scour protection. The only recourse is to remove the remains of the installation and replace it with a redesigned countermeasure.

Recommended actions based on current condition rating:

Code U: The riprap cannot be inspected. A plan of action should be developed to determine the condition of the installation. Possible remedies may include removal of debris, excavation during low flow, probing, or non-destructive testing using ground penetrating radar or seismic methods.

Codes 6 or 5: Continue periodic inspection program at the specified interval.

Codes 4 or 3: Increase inspection frequency. The rating history of the installation should be tracked to determine if a downward trend in the rating is evident. Depending on the nature of the riprap application, the installation of monitoring instruments might be considered.

Code 2: The maintenance engineer's office should be notified and maintenance should be scheduled. The cause of the low rating should be determined, and consideration given to redesign and replacement. Larger stone size or alternative scour countermeasure systems should be considered as a replacement.

Codes 1 or 0: The maintenance engineer's office should be notified immediately. Depending upon the nature of the riprap application, other local officials and/or law enforcement agencies identified in the Plan of Action for the bridge may also need to be notified.

for each of the three topical areas. Recommended actions corresponding to the current condition rating are also provided.

3.5.2 Pier Riprap Failure Modes

Schoharie Creek Case Study. FHWA's HEC-18 (Richardson and Davis 2001) and HEC-23 (Lagasse et al. 2001) document the catastrophic bridge failure at Schoharie Creek attributed to inadequate pier riprap.

The failure of the I-90 bridge over Schoharie Creek near Albany, New York, on April 5, 1987, which cost 10 lives, was investigated by the National Transportation Safety Board (NTSB). The peak flow was 64,900 cfs (1,838 m³/s) with a 70- to 100-year return period. The foundations of the four bridge piers were large spread footings 82 ft (25 m) long, 18 ft (5.5 m) wide, and 5 ft (1.5 m) deep without piles. The footings were set 5 ft (1.5 m) into the stream bed in very dense ice contact

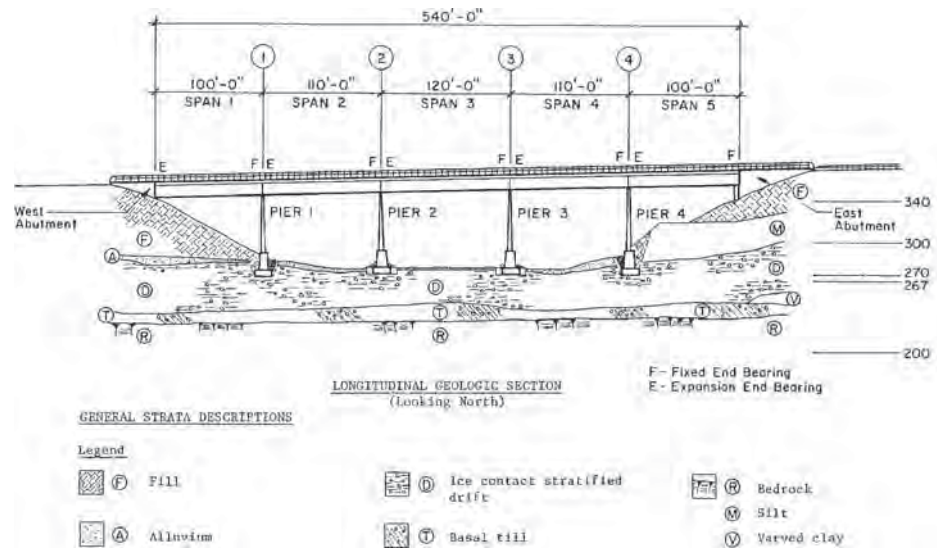


Figure C3.1. South elevation of Schoharie Creek bridge showing key structural features and a schematic geological section.

stratified glacial drift, which was considered non-erodible by the designers (Figure C3.1). However, flume studies of samples of the stratified drift showed that some material would be eroded at a velocity of 4 ft/s (1.5 m/s), and, at a velocity of 8 ft/s (2.4 m/s), the erosion rates were high.

A 1:50-scale, 3-D model study established a flow velocity of 10.8 ft/s (3.3 m/s) at the pier that failed. Also, the 1:50-scale, 3-D model and a 1:15-scale, 2-D model study gave 15 ft (4.6 m) of maximum scour depth. The scour depth of the prototype pier (pier 3) at failure was 14 ft (4.3 m) (Figure C3.2).

Design plans called for the footings to be protected with riprap. Over time (1953 to 1987), much of the riprap was removed by high flows. NTSB gave as the probable cause “. . . the failure of the New York State Thruway authority [NYSTA] to maintain adequate riprap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the severity of the accident was the lack of structural redundancy in the bridge.”



Note: Pier 2 is in the foreground with Pier 3 in the background.

Figure C3.2. Pier scour holes at Schoharie Creek bridge in 1987.



Figure C3.3. Photograph of riprap at pier 2, October 1956.

The NYSTA inspected the bridge annually or biennially with the last inspection on April 1, 1986. A 1979 inspection by a consultant hired by the New York State Department of Transportation indicated that most of the riprap around the piers was missing (Figures C3.3 and C3.4); however, the 1986 inspection failed to detect any problems with the condition of the riprap at the piers. Based on the NTSB findings, the conclusions from this failure are that inspectors and their supervisors must recognize that riprap does not necessarily make a bridge safe from scour, and inspectors must be trained to recognize when riprap is missing and the significance of this condition.

Summary. Examples of the most common modes of riprap failure at piers provide guidance for post-flood and post-construction performance evaluation. Inspectors need to be aware of, and understand, the causes of riprap inadequacies that they see in the field. While the specific mechanism causing failure of the riprap is difficult to determine, and a number of factors, acting either individually or combined, may be involved, the reasons for riprap failures at bridge piers can be summarized as follows:

- Particle size was too small because
 - Shear stress was underestimated



Note: Flow is from right to left.

Figure C3.4. Photograph of riprap at pier 2, August 1977.

- Velocity was underestimated
- Inadequate allowance was made for channel curvature
- Design channel capacity was too low
- Design discharge was too low
- Inadequate assessment was made of abrasive forces
- Inadequate allowance was made for effect of obstructions (such as debris)
- Channel changes caused
 - Increased angle of attack (skew)
 - Decreased channel capacity or increased depth
 - Scour
- Riprap material had improper gradation
- Material was placed improperly
- No filter blanket was installed or blanket was inadequate or damaged

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APPENDIX D

Guidelines for Pier Scour Countermeasures Using Partially Grouted Riprap

Introduction, D-2

1 Design and Specification, D-3

2 Construction, D-15

3 Inspection, Maintenance, and Performance Evaluation, D-22

References, D-26

Introduction

Partially grouted riprap, when properly designed and used for erosion protection, has an advantage over rigid structures because it is flexible when under attack by river currents, it can remain functional even if some individual stones may be lost, and it can be repaired relatively easily. Properly constructed, partially grouted riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. This design guideline considers the application of partially grouted riprap as a pier scour countermeasure.

Partially grouted riprap consists of appropriately sized rocks that are placed around a pier and grouted together with grout filling 50% or less of the total void space (Figure D1.1). In contrast to fully grouted riprap, partial grouting increases the overall stability of the riprap installation unit without sacrificing flexibility or permeability. It also allows for the use of smaller rock compared to standard riprap, resulting in decreased layer thickness. Because riprap is a natural material and is readily available in many areas, it has been used extensively in erosion protection works.

Design of a pier scour countermeasure system using partially grouted riprap requires knowledge of river bed and foundation material; flow conditions including velocity, depth, and orientation; pier size, shape, and skew with respect to flow direction; riprap characteristics of size, density, durability, and availability; and the type of interface material between the partially grouted riprap and underlying foundation. The system typically includes a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention.

The guidance for pier scour applications provided in this document has been developed primarily from the results of NCHRP Project 24-07(2) (Lagasse et al. 2007) and publications from the Federal Waterway Engineering and Research Institute (Bundesanstalt für Wasserbau, or BAW) in Germany (e.g., BAW 1990).

This document is organized into three parts:

- Part 1 provides design and specification guidelines for partially grouted riprap systems.
- Part 2 presents construction guidelines.



Figure D1.1. Close-up view of partially grouted riprap.

- Part 3 provides guidance for inspection, maintenance, and performance evaluation of partially grouted riprap used as a pier scour countermeasure.

Part 1: Design and Specification

1.1 Materials

1.1.1 Rock Riprap

Riprap design methods typically yield a required size of stone that will result in stable performance under the design loadings. Because stone is produced and delivered in a range of sizes and shapes, the required size of stone is often stated in terms of a minimum allowable representative size. For pier scour protection, the designer specifies a minimum allowable d_{50} for the rock composing the riprap, thus indicating the size for which 50% (by weight) of the particles are smaller. Stone sizes can also be specified in terms of weight (e.g., W_{50}) using an accepted relationship between size and volume, and the known (or assumed) density of the particle.

Shape: The shape of a stone can be generally described by designating three axes of measurement: major, intermediate, and minor, also known as the “A, B, and C” axes, as shown in Figure D1.2.

Riprap stones should not be thin and platy, nor should they be long and needle like. Therefore, specifying a maximum allowable value for the ratio A/C , also known as the shape factor, provides a suitable measure of particle shape, since the B axis is intermediate between the two extremes of length A and thickness C. A maximum allowable value of 3.0 is recommended:

$$\frac{A}{C} \leq 3.0 \quad (\text{D1.1})$$

For riprap applications, stones tending toward subangular to angular are preferred, because of the higher degree of interlocking, hence greater stability, compared to rounded particles of the same weight.

Density: A measure of density of natural rock is the specific gravity, S_g , which is the ratio of the density of a single (solid) rock particle, γ_s , to the density of water, γ_w :

$$S_g = \frac{\gamma_s}{\gamma_w} \quad (\text{D1.2})$$

Usually, a minimum allowable specific gravity of 2.5 is required for riprap applications. Where quarry sources uniformly produce rock with a specific gravity significantly greater than 2.5 (such as dolomite, $S_g = 2.7$ to 2.8), the equivalent stone size can be substantially reduced and still achieve the same particle weight gradation.

Size and weight: Based on field studies, the recommended relationship between size and weight is given by

$$W = 0.85(\gamma_s d^3) \quad (\text{D1.3})$$

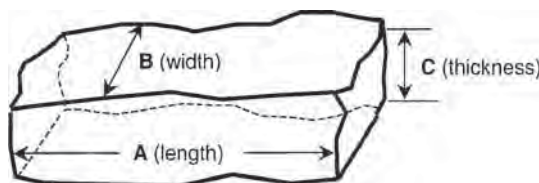


Figure D1.2. Riprap shape described by three axes.

where:

W = Weight of stone, lb (kg)

γ_s = Density of stone, lb/ft³ (kg/m³)

d = Size of intermediate (“B”) axis, ft (m)

Table D1.1 provides recommended gradations for 10 standard classes of riprap based on the median particle diameter, d_{50} , as determined by the dimension of the intermediate (“B”) axis. These gradations were developed under NCHRP Project 24-23, “Riprap Design Criteria, Specifications, and Quality Control” (Lagasse et al. 2006). The proposed gradation criteria are based on a nominal or “target” d_{50} and a uniformity ratio, d_{85}/d_{15} , which result in riprap that is well graded. The target uniformity ratio is 2.0 and the allowable range is from 1.5 to 2.5.

The intent of partial grouting is to “glue” stones together to create a conglomerate of particles. Each conglomerate is therefore significantly greater than the d_{50} stone size and typically is larger than the d_{100} size of the individual stones in the riprap matrix. **Only three standard classes may be used with the partial grouting technique: Classes II, III, and IV.** Riprap smaller than Class II exhibits voids that are too small for grout to effectively penetrate to the required depth within the rock matrix, while riprap that is larger than Class IV has voids that are too large to retain the grout, and does not have enough contact area between stones to effectively glue them together.

Permeability of the completed installation is maintained because less than 50% of the void space is filled with grout. Flexibility of the installation occurs because the matrix will fracture into the conglomerate-sized pieces under hydraulic loading and/or differential settlement. The surface of each conglomerate particle is highly rough and irregular, and so maintains excellent interlocking between particles after fracturing occurs.

Based on Equation D1.3, which assumes the volume of the stone is 85% of a cube, Table D1.2 provides the equivalent particle weights for the same 10 classes, using a specific gravity of 2.65 for the particle density.

1.1.2 Recommended Tests for Rock Quality

Standard test methods relating to material type, characteristics, and testing of rock and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are provided in this section and are recommended for specifying the quality of the riprap stone. In general, the test methods recommended in this section are intended to ensure that the stone is dense and durable, and will not degrade significantly over time.

Table D1.1. Size gradations for 10 standard classes of riprap.

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Size	Min	Max	Min	Max	Min	Max	Max
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Note: Only Classes II, III, and IV are suitable for use in partial grouting applications

Table D1.2. Weight gradations for 10 standard classes of riprap.

Nominal Riprap Class by Median Particle Weight		W ₁₅		W ₅₀		W ₈₅		W ₁₀₀
Class	Weight	Min	Max	Min	Max	Min	Max	Max
I	20 lb	4	12	15	27	39	64	140
II	60 lb	13	39	51	90	130	220	470
III	150 lb	32	93	120	210	310	510	1100
IV	300 lb	62	180	240	420	600	1000	2200
V	1/4 ton	110	310	410	720	1050	1750	3800
VI	3/8 ton	170	500	650	1150	1650	2800	6000
VII	1/2 ton	260	740	950	1700	2500	4100	9000
VIII	1 ton	500	1450	1900	3300	4800	8000	17600
IX	2 ton	860	2500	3300	5800	8300	13900	30400
X	3 ton	1350	4000	5200	9200	13200	22000	48200

Note: Only Classes II, III, and IV are suitable for use in partial grouting applications

Rocks used for riprap should break only with difficulty, have no earthy odor, not have closely spaced discontinuities (joints or bedding planes), and should not absorb water easily. Rocks composed of appreciable amounts of clay—such as shales, mudstones, and claystones—are never acceptable for use as partially grouted riprap. Table D1.3 summarizes the recommended tests and allowable values for rock and aggregate.

Table D1.3. Recommended tests for rock quality.

Test Designation	Property	Allowable value		Frequency ⁽¹⁾	Comments
AASHTO TP 61	Percentage of Fracture	< 5%		1 per 20,000 tons	Percentage of pieces that have fewer than 50% fractured surfaces
AASHTO T 85	Specific Gravity and Water Absorption	Average of 10 pieces: S _g > 2.5 Absorption < 1.0%		1 per year	If any individual piece exhibits an S _g less than 2.3 or water absorption greater than 3.0%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO T 103	Soundness by Freezing and Thawing	Maximum of 10 pieces after 25 cycles: < 0.5%		1 per 2 years	Recommended only if water absorption is greater than 0.5% and the freeze-thaw severity index is greater than 15 per ASTM D 5312.
AASHTO T 104	Soundness by Use of Sodium Sulfate or Magnesium Sulfate	Average of 10 pieces: < 17.5%		1 per year	If any individual piece exhibits a value greater than 25%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO TP 58	Durability Index Using the Micro-Deval Apparatus	Value > 90 > 80 > 70	Application Severe Moderate Mild	1 per year	Severity of application per Section 5.4, CEN (2002). Most riverine applications are considered mild or moderate.
ASTM D 3967	Splitting Tensile Strength of Intact Rock Core Specimens	Average of 10 pieces: > 6 MPa		1 per year	If any individual piece exhibits a value less than 4 MPa, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
ASTM D 5873	Rock Hardness by Rebound Hammer	See Note (2)		1 per 20,000 tons	See Note (2)
Shape	Length to Thickness Ratio A/C	< 10%, d ₅₀ < 24 in < 5%, d ₅₀ > 24 in		1 per 20,000 tons	Percentage of pieces that exhibit A/C ratio greater than 3.0 using the Wolman count method (Lagasse et al., 2006)
ASTM D 5519	Particle Size Analysis of Natural and Man-Made Riprap Materials			1 per year	See Note (3)
Gradation	Particle Size Distribution Curve			1 per 20,000 tons	Determined by the Wolman count method (Lagasse et al., 2006), where particle size, d, is based on the intermediate (B) axis.

(1) Testing frequency for acceptance of riprap from certified quarries, unless otherwise noted. Project-specific tests exceeding quarry certification requirements, either in performance value or frequency of testing, must be specified by the Engineer.

(2) Test results from D 5873 should be calibrated to D 3967 results before specifying quarry-specific minimum allowable values.

(3) Test results from D 5519 should be calibrated to Wolman count (Lagasse et al., 2006) results before developing quarry-specific relationships between size and weight; otherwise, assume W = 85% that of a cube of dimension d having a specific gravity of S_g.

Table D1.4. Mixture for 1 yd³ of grout.

Material	Quantity by weight
Ordinary Portland cement	740 to 760 lb
Fine concrete aggregate (sand), dry	1,180 to 1,200 lb
¼" crusher chips (very fine gravel), dry	1,180 to 1,200 lb
Water	420 to 450 lb
Air entrained	5% to 7%
Anti-washout additive (used only for placement under water)	6 to 8 lb

1.1.3 Grout

For partially grouted riprap applications, only Portland cement–based grout is appropriate. General requirements for grouting materials are based on guidance developed by the BAW in Germany (BAW 1990). Table D1.4 provides guidance on the basic grout mix for 1 yd³ (0.76 m³) of grout.

The mix should result in a wet grout density ranging from 120 to 140 lb/ft³ (2.0 to 2.3 kg/dm³). Wet densities outside this range should be rejected and the mix re-evaluated for material properties of the individual constituents.

1.1.4 Recommended Tests for Grout Quality

A variety of tests have been developed by the BAW in Germany. The two most relevant tests are described below. The full document entitled, “Guidelines for Testing of Cement and Bitumen Bonded Materials for the Grouting of Armor Stones on Waterways” has been translated into English as part of NCHRP Project 24-07(2) and can be found in the Reference Document (available on the TRB website: http://www.trb.org/news/blurp_detail.asp?id=7998).

Consistency Test. The consistency of Portland cement–based grouting material is determined using a slump test. A standardized slump cone and portable test table have been developed for this purpose. Figure D1.3 provides photographs illustrating the method. The diameter of the slumped grout is measured after pulling the cone without tapping and again after 15 taps of the test table.

For placement in the dry: 34 to 38 cm without tapping
50 to 54 cm after 15 taps

For placement under water: 30 to 34 cm without tapping
34 to 38 cm after 15 taps



a. Slump cone and test table



b. Measuring grout slump

Figure D1.3. Consistency test for Portland cement grout.

Washout Test. The washout test provides a measure of resistance to erosion by measuring the loss of grout material when immersed in water. A screened basket 13 cm in diameter with a 3-mm mesh size is filled with 2.0 kg of fresh grout. The grout is lightly tamped and the grout-filled basket is weighed. The basket is then dropped three times into a water tank of 1 m height. Afterwards the grout and basket are weighed again, and the loss of mass is determined. The maximum permissible loss of mass is 6.0%.

1.2 Hydraulic Stability Design Procedure

With partially grouted riprap, there are no relationships *per se* for selecting the size of rock, other than the practical considerations of proper void size, gradation, and adequate stone-to-stone contact area as discussed in Section 1.1.

Prototype-scale tests of partially grouted riprap at a pier were performed for NCHRP Project 24-07(2) by Colorado State University (CSU) in 2005. The CSU tests were conducted in a 20-ft (6 m) wide outdoor flume (see Lagasse et al. 2007). In the laboratory setting, Class I riprap with a d_{50} of 6 in. (15 cm) was partially grouted on one side of the pier and standard (loose) rock having the same gradation was placed on the other side. Discharges were steadily increased until an approach velocity of 6.6 ft/s (2.0 m/s) was achieved upstream of the pier, at which point the maximum discharge capacity of the flume was reached. Using a velocity multiplier of 1.7 to account for the square-nose pier shape, local velocity at the pier was estimated to be approximately 11 ft/s (3.4 m/s). The partially grouted riprap was undamaged after several hours of testing, whereas the loose riprap experienced damage by particle displacement.

Tests of partially grouted riprap at Braunschweig University, Germany, demonstrated the ability of partially grouted riprap to remain stable and undamaged in high-velocity flow of 26 ft/s (8 m/s) (Heibaum 2000). However, those tests were not conducted at a pier.

It is recommended that for field applications, the class of riprap (II, III, or IV) used for a partially grouted pier scour countermeasure be selected based on the economics of locally available riprap material that satisfies the gradation requirements of Section 1.1.

1.3 Layout Dimensions

Based on laboratory studies performed for NCHRP Project 24-07(2), the optimum performance of partially grouted riprap as a pier scour countermeasure was obtained when the armor extended a distance of at least 1.5 times the pier width in all directions around the pier.

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α , which is a function of the width, a , and length, L , of the pier (or pile bents) and the skew angle, α , as given below (after Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cos \alpha + L \sin \alpha}{a} \right)^{0.65} \quad (\text{D1.4})$$

Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation. Placing the top of the riprap flush with the bed is ideal for inspection purposes and does not create any added obstruction to the flow. Mounding riprap around a pier is not acceptable for design in most cases, because it obstructs flow, captures debris, and increases scour at the periphery of the installation.

The riprap layer should have a thickness of at least 2 times the d_{50} size of the rock, as shown in Figure D1.4. When placement must occur under water, the thickness of the riprap should be

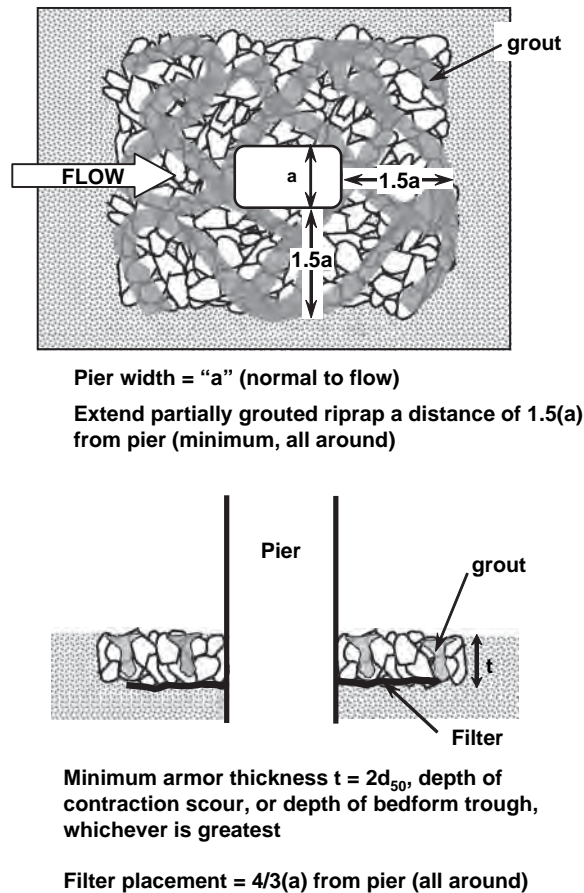
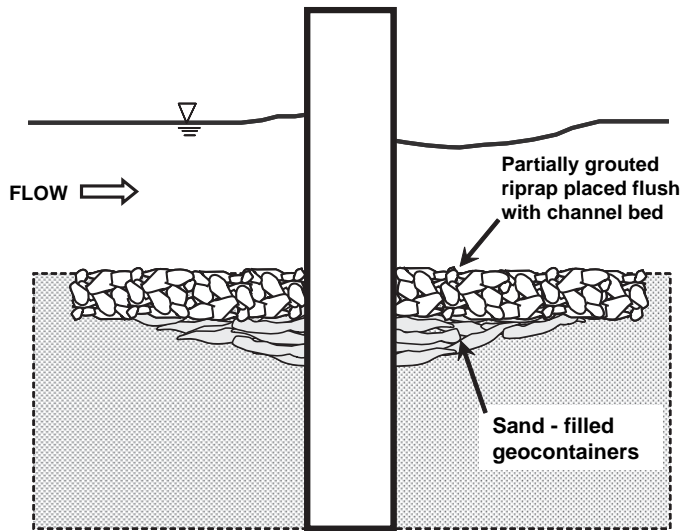


Figure D1.4. Partially grouted riprap layout diagram for pier scour countermeasures.

increased by 50% to account for irregularities in subgrade excavation; **however, in this case the recommended grout application quantity should not be increased in kind.**

When contraction scour through the bridge opening exceeds $2d_{50}$, the thickness of the armor must be increased to the full depth of the contraction scour plus any long-term degradation. In river systems where dune-type bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of Karim (1999) and/or van Rijn (1984). In general, an upper limit on the crest-to-trough height, Δ , is provided by Bennett (1997) as $\Delta < 0.4y$ where y is the depth of flow. This limit suggests that the maximum depth of the bed-form trough below ambient bed elevation will not exceed 0.2 times the depth of flow. Additional armor thickness due to any of these conditions may warrant an increase in the extent of the partially grouted riprap away from the pier faces.

A filter layer is typically required for partially grouted riprap at bridge piers. The filter should **not** be extended fully beneath the armor; instead, it should be terminated two-thirds of the distance from the pier to the edge of the armor layer. When using a granular stone filter, the layer should have a minimum thickness of 4 times the d_{50} of the filter stone or 6 in. (15 cm), whichever is greater. As with riprap, the layer thickness should be increased by 50% when placing under water. Sand-filled geocontainers made of properly selected materials provide a convenient method for controlled placement of a filter in flowing water. This method can also be used to partially fill an existing scour hole when placement must occur under water, as illustrated in Figure D1.5. For more detail, see Lagasse et al. (2001, 2007).



Minimum armor thickness $t = 2d_{50}$, depth of contraction scour, or depth of bedform trough, whichever is greatest

Filter placement = $4/3(a)$ from pier (all around)

Figure D1.5. Schematic diagram showing sand-filled geocontainer filter beneath partially grouted riprap.

1.4 Filter Requirements

The importance of the filter component of a partially grouted riprap installation should not be underestimated. Two kinds of filters are used in conjunction with partially grouted riprap: granular filters and geotextile filters. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of the filter layer. ***In cases where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.***

The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain *all* the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind.

1.4.1 Geotextile Filter Properties

Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.***

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand stresses during installation. These properties are readily available from manufacturers. The following list briefly describes the most relevant properties:

- **Permeability.** The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of cen-

timeters per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows across the plane of the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of $(s)^{-1}$. Permeability (and permittivity) is extremely important in filter design. For partially grouted riprap installations at piers, the permeability of the geotextile should be at least 10 times greater than that of the underlying material:

$$K_g > 10K_s \quad (D1.5)$$

where

K_g = Permeability of geotextile (cm/s)

K_s = Permeability of subgrade soil (cm/s)

- **Transmissivity.** The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second. This property is directly related to the drainage function and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas non-woven, needle-punched fabrics have a much greater capacity due to their three-dimensional (3-D) microstructure. Transmissivity is not particularly relevant to filter design.
- **Apparent opening size (AOS).** Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.
- **Porosity.** Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Percent open area (POA).** POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Thickness.** As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).
- **Grab strength and elongation.** Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.
- **Tear strength.** Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.
- **Puncture strength.** Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

Table D1.5 provides the recommended characteristics for geotextile filters. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving pier riprap have been discussed here. Geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by the partially grouted

Table D1.5. Recommended requirements for geotextile properties.

Test Designation	Property	Allowable value ⁽¹⁾		Comments
		Elongation < 50% ⁽²⁾	Elongation > 50% ⁽²⁾	
ASTM D 4632	Grab Strength	> 315 lbs (Class 1) > 250 lbs (Class 2) > 180 lbs (Class 3)	> 200 lbs (Class 1) > 160 lbs (Class 2) > 110 lbs (Class 3)	From AASHTO M 288
ASTM D 4632	Sewn Seam Strength ⁽³⁾	> 270 lbs (Class 1) > 220 lbs (Class 2) > 160 lbs (Class 3)	> 180 lbs (Class 1) > 140 lbs (Class 2) > 100 lbs (Class 3)	From AASHTO M 288
ASTM D 4533	Tear Strength ⁽⁴⁾	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4833	Puncture Strength	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4751	Apparent Opening Size	Per design criteria (Section 1.4 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 1.4 of this design guide)		Minimum allowable value
ASTM D 4355	Degradation by Ultraviolet Light	> 50% strength retained after 500 hours of exposure		Minimum allowable value
ASTM D 4873	Guide for Identification, Storage, and Handling			Provides information on identification, storage, and handling of geotextiles.
ASTM D 4759	Practice for the Specification Conformance of Geosynthetics			Provides information on procedures for ensuring that geotextiles at the jobsite meet the design specifications.

(1) Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class. The following descriptions have been modified from AASHTO M 288:

- *Class 1* is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including when placement of riprap must occur in multiple lifts, when drop heights may exceed 1 ft (0.3 m) or when repeated vehicular traffic on the installation is anticipated.
- *Class 2* is recommended for installation conditions where placement in regular, single lifts are expected and little or no vehicular traffic on the installation will occur, or when placing individual rocks by clamshell, orange-peel grapple or specially equipped hydraulic excavator with drop heights less than 1 ft.
- *Class 3* is specified for the least severe installation environments, with drop heights less than 1 ft onto a bedding layer of select sand, gravel or other select imported material.

(2) As measured in accordance with ASTM D 4632.

(3) When seams are required.

(4) The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 55 lbs. The MARV corresponds to a statistical measure whereby 2.5% of the tested values are less than the mean value minus two standard deviations (Koerner 1998).

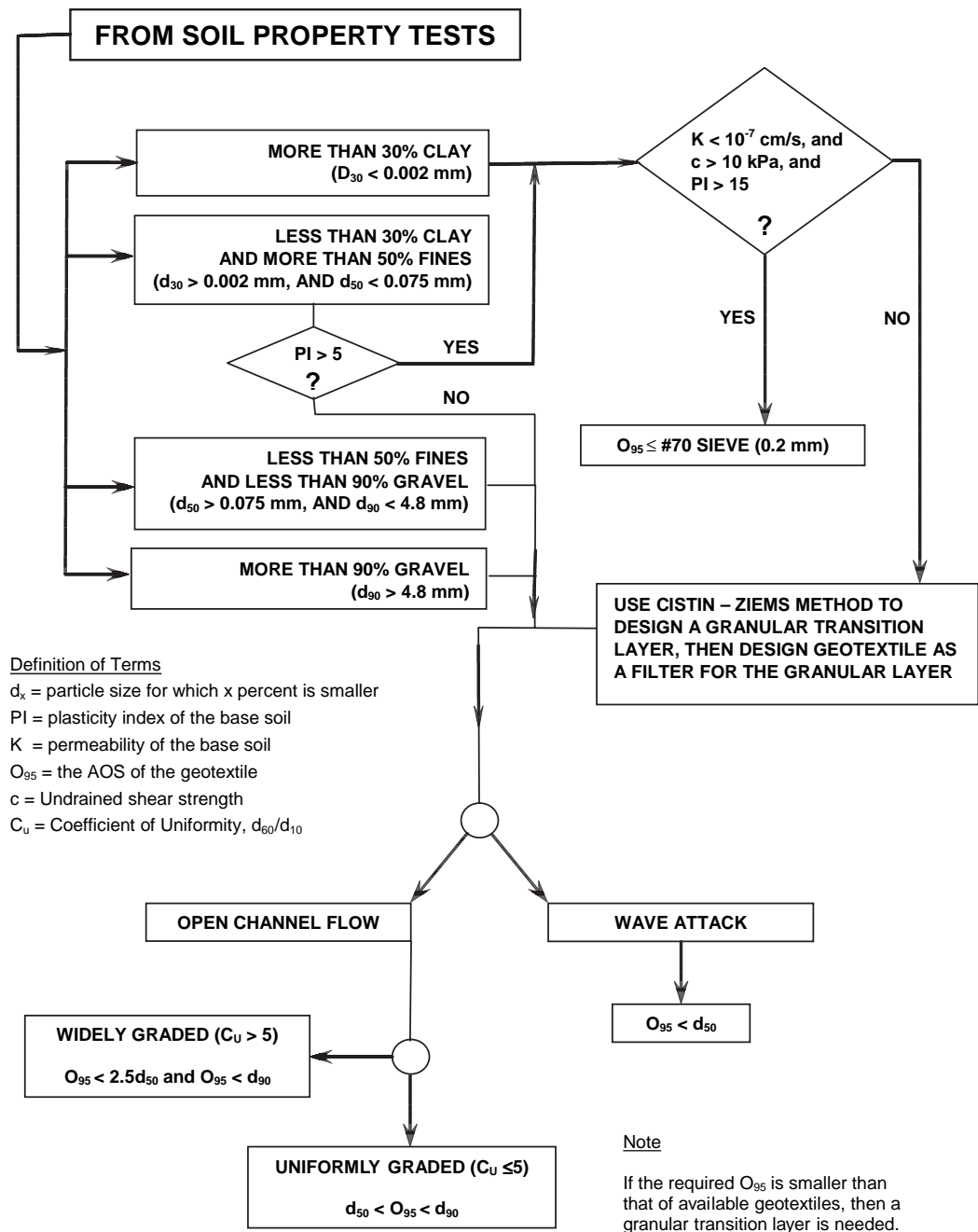
riprap, these stresses do not represent the environment that the geotextile will experience in the long term.

1.4.2 Geotextile Filter Design Procedure

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. A decision tree is provided as Figure D1.6 to assist in determining the appropriate soil retention criterion for the geotextile. The figure includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater than 30% clay or is predominantly fine-grained soil (more than 50% passing the #200 sieve).

If a granular transition layer is required, the geotextile should be designed to be compatible with the properties of the granular layer. If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required. However, this requirement can be waived if



Source: modified from Koerner (1998)

Figure D1.6. Geotextile selection based on soil retention.

the base soil exhibits the following conditions for hydraulic conductivity, K; plasticity index, PI; and undrained shear strength, c:

$$K < 1 \times 10^{-7} \text{ cm/s}$$

$$PI > 15$$

$$c > 10 \text{ kPa}$$

Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS less than a #70 sieve (approximately 0.2 mm) can be used with soils meeting these conditions and essentially functions more as a separation layer than a filter.

Step 3. Determine Permeability Criterion. The permeability criterion requires that the filter exhibit a permeability at least 4 times greater than that of the base soil (Koerner 1998) and for critical or severe applications, at least 10 times greater (Holtz et al. 1995). Generally speaking, if the permeability of the base soil or granular filter has been determined from laboratory testing, that value should be used. If laboratory testing was not conducted, then an estimate of permeability based on the particle size distribution should be used.

To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s^{-1} . Typically, the designer will need to contact the geotextile manufacturer to obtain values of permeability, permittivity, and thickness.

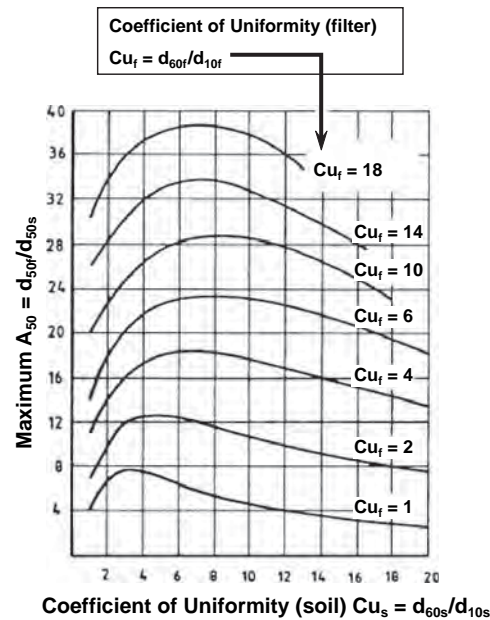
Step 4. Select a Geotextile that Meets the Required Strength Criteria. Strength and durability requirements depend on the installation environment and the construction equipment that is being used. See Table D1.5 for recommended values based on AASHTO M 288, “Geotextile Specification for Highway Construction,” which provides guidance on allowable strength and elongation values for three categories of installation severity. For additional guidelines regarding the selection of durability test methods, refer to ASTM D 5819, “Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability.”

Step 5. Minimize Long-Term Clogging Potential. When a woven geotextile is used, its POA should be greater than 4% by area. If a non-woven geotextile is used, its porosity should be greater than 30% by volume. A good rule of thumb suggests that the geotextile having the largest AOS that satisfies the particle retention criteria should be used (provided of course that all other minimum allowable values described in this section are met as well).

1.4.3 Granular Filter Properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters can be used alone or can serve as a transitional layer between a predominantly fine-grained base soil and a geotextile. The following list briefly describes the most relevant properties:

- **Particle Size Distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziems whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure D1.7 provides a design chart based on the Cistin–Ziems approach.
- **Permeability.** Permeability of a granular filter material is determined by laboratory test or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For partially grouted riprap at piers, the permeability of the granular filter should be at least 10 times greater than that of the underlying material.
- **Porosity.** Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.
- **Thickness.** Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. is specified. For placement under water, thickness should be increased by 50%.
- **Quality and durability.** Aggregate used for a granular filter should be hard, dense, and durable.



Source: Heibaum (2004)

Figure D1.7. Granular filter design chart according to Cistin and Ziems.

1.4.4 Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter. That approach was developed for subsoils consisting of well-graded sands and may not be widely applicable to other soil types. An alternative approach that is considered more robust in this regard is the Cistin–Ziems method.

The suggested steps for proper design of a granular filter using this method are outlined below. Note that the subscript “s” is used to represent the base (finer) soil, and “f” is used to represent the filter (coarser) layer.

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Key Indices for Base Soil. From the grain size information, determine the median grain size, d_{50} , and the coefficient of uniformity, $Cu_s = d_{60}/d_{10}$, of the base soil.

Step 3. Determine Key Indices for Granular Filter. One or more locally available aggregates should be identified as potential candidates for use as a filter material. The d_{50} and coefficient of uniformity, $Cu_f = d_{60}/d_{10}$, should be determined for each candidate filter material.

Step 4. Determine Maximum Allowable d_{50} for Filter. Enter the Cistin–Ziems design chart (Figure D1.7) with the coefficient of uniformity, Cu_s , for the base soil on the x-axis. Find the curve that corresponds to the coefficient of uniformity, Cu_f , for the filter in the body of the chart and, from that point, determine the maximum allowable A_{50} from the y-axis. Compute the maximum allowable d_{50f} of the filter using d_{50fmax} equals A_{50max} times d_{50s} . Check to see if the candidate filter material conforms to this requirement. If it does not, continue checking alternative candidates until a suitable material is identified.

Step 5. Check for Permeability. From laboratory permeameter tests or the grain size distribution of the candidate filter material, determine whether the hydraulic conductivity of the filter is at least 10 times greater than that of the subsoil.

Step 6. Check for Compatibility with Riprap Rock. Repeat steps 1 through 4 above, considering that the filter material is now the “finer” soil and the partially grouted riprap is the “coarser” material. If the Cistin–Ziems criterion is not met, then multiple layers of granular filter materials should be considered.

Step 7. Filter Layer Thickness. For practicality of placement, the nominal thickness of a single filter layer should not be less than 6 in. (15 cm). Single-layer thicknesses up to 15 in. (38 cm) may be warranted where large rock fill particle sizes are used. When multiple filter layers are required, each individual layer should range from 4 to 8 in. (10 to 20 cm) in thickness (Brown and Clyde 1989).

NOTE: *In cases where dune-type bed forms may be present or of underwater installation, it is strongly recommended that only a geotextile filter be considered.*

Part 2: Construction

Partially grouted riprap is placed in a riverine or coastal environment to prevent scour or erosion of the bed, banks, shoreline, or near structures such as bridge piers and abutments. Partially grouted riprap construction involves placement of rock and stone in layers on top of a bedding or filter layer composed of sand, gravel, and/or geotechnical fabric. The voids of the riprap matrix are then partially filled with a Portland cement–based grout by hose or tremie, or by automated mechanical means. The final configuration results in an armor layer that retains approximately 50% to 65% of the void space of the original riprap. Hydraulic stability of the armor is increased significantly over that of loose (ungrouted) riprap by virtue of the much larger mass and high degree of interlocking of the conglomerate particles created by the grouting process.

Factors to consider when designing partially grouted riprap countermeasures begin with the source for the rock; the method to obtain or manufacture the rock; competence of the rock; and the methods and equipment to collect, transport, and place the riprap. Rock for riprap may be obtained from quarries, by screening oversized rock from earth borrow pits, by collecting rock from fields, or from talus deposits. Screening borrow pit material and collecting field rocks present problems such as rocks that are too large or that have unsatisfactory length-to-width ratios for riprap. Quarries are generally the best source for obtaining rock for riprap. Because the partial grouting process effectively creates larger particles from smaller ones, potential concerns regarding quarrying practices needed to produce large, competent, and unfractured riprap sizes are essentially eliminated.

In most cases, the production of the rock material will occur at a quarry that is relatively remote from the construction area. Therefore, this discussion assumes that the rock is hauled to the site of the installation, where it is dumped either directly, stockpiled, or loaded onto waterborne equipment.

Riprap should be fully grouted along vertical surfaces such as piers, where void space is higher and settling would result in larger gaps. Flowability of the grout should be tested prior to placement. Grout placed under water requires special additives to prevent segregation of the aggregates and washout of the Portland cement during placement. “Stickiness” of the grout in underwater applications is important; therefore, an anti-washout additive is recommended for this reason (see Section 1.1.3) based on extensive testing and field application by the BAW in Germany.

The construction objectives for a properly partially grouted riprap armor layer follow:

1. To obtain a rock mixture from the quarry that meets the design specifications
2. To place that mixture in a well-knit, compact, and uniform layer
3. To ensure proper grout coverage and penetration to the desired depth

The guidance in this section has been developed to facilitate the proper installation of partially grouted riprap armor to achieve suitable hydraulic performance and maintain stability against

hydraulic loading to protect against scour at bridge piers. The proper installation of partially grouted riprap systems is essential to the adequate functioning and performance of the system during the design hydrologic event. Guidelines are provided herein for maximizing the correspondence between the design intent and the actual field-finished conditions of the project. This section addresses the preparation of the subgrade, geotextile placement, riprap and grout placement, backfilling and finishing, and measurement and payment.

2.1 General Guidelines

The contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. This responsibility is typically performed by the engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see ER 1180-1-6 [U.S. Army Corps of Engineers 1995]) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for riprap placement are included in the project plans and specifications. Recommended riprap specifications and layout guidance are found in Part 1 of this appendix. Recommended requirements for the stone, including the tests necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications, are provided. Field tests can be performed at the quarry and/or on the job site, or representative samples can be obtained for laboratory testing.

Gradations are specified and plan sheets show locations, grades, and dimensions of rock layers for the countermeasure. The stone shape is important and riprap should be blocky rather than elongated, platy, or round. In addition, the stone should have sharp, angular, clean edges at the intersections of relatively flat surfaces.

Segregation of rock material during transportation, dumping, or off-loading is not acceptable. Inspection of riprap placement consists of visual inspection of the operation and the finished surface. Inspection must ensure that a dense, rough surface of well-keyed graded rock of the specified quality and sizes is obtained, that the layers are placed such that voids are minimized, and that the layers are the specified thickness.

Inspection and quality assurance must be carefully organized and conducted in case potential problems or questions arise over acceptance of stone material. Acceptance of the work should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject stone at the quarry, at the job site or stockpile, and in place in the structures throughout the duration of the contract. Stone rejected at the job site should be removed from the project site. Stone rejected at the quarry should be disposed of or otherwise prevented from mixing with satisfactory stone.

Various degrees of grouting are possible, but the optimal performance is achieved when the grout is effective at “gluing” individual stones to neighboring stones at their contact points, while leaving relatively large voids between the stones.

Construction techniques can vary tremendously because of the following factors:

- Size and scope of the overall project
- Size and weight of the riprap particles
- Placement under water or in the dry
- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions

- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general information regarding construction of partially grouted riprap installations and some basic information and description of techniques and processes involved in the construction of partially grouted riprap armor as a pier scour countermeasure.

2.2 Materials

2.2.1 Stone

The best time to control the gradation of the riprap mixture is during the quarrying operation. Generally, sorting and mixing later in stockpiles or at the construction site is not recommended. Inspection of the riprap gradation at the job site is usually carried out visually. Therefore, it is helpful to have a pile of rocks with the required gradation at a convenient location where inspectors can see and develop a reference to judge by eye the suitability of the rock being placed. On-site inspection of riprap is necessary both at the quarry and at the job site to ensure proper gradation and material that does not contain excessive amounts of fines. Breakage during handling and transportation should be taken into account

The Wolman count method (Wolman 1954) as described in *NCHRP Report 568* (Lagasse et al. 2006) may be used as a field test to determine a size distribution based on a random sampling of individual stones within a matrix. This method relies on samples taken from the surface of the matrix to make the method practical for use in the field. The procedure determines frequency by size of a surface material rather than using a bulk sample. The intermediate dimension (B axis) is measured for 100 randomly selected particles on the surface.

The Wolman count method can be done by stretching a survey tape over the material and measuring each particle located at equal intervals along the tape. The interval should be at least 1 ft for small riprap and increased for larger riprap. The longer and shorter axes (A and C) can also be measured to determine particle shape. One rule that must be followed is that if a single particle is large enough to fall under two interval points along the tape, then it should be included in the count twice. It is best to select an interval large enough that this does not occur frequently.

2.2.2 Grout

The grout should not segregate when being applied to the riprap. When grout is placed under water, segregation and dispersion of fine particles is prevented by use of a chemical additive (Sicotan^(r)) as described in Section 1.1.3. The target distribution of grout within the riprap matrix is such that about two-thirds of the grout should reside in the upper half of the riprap layer, with one-third of the grout penetrating into the lower half.

The grout must not be allowed to pool on the surface of the riprap, nor puddle onto the filter at the base of the riprap. Therefore, prior to actual placement, rates of grout application should be established on test sections and adjusted based on the size of the grout nozzle and consistency of the grout. Construction methods should be closely monitored to ensure that the appropriate voids and surface openings are achieved.

2.2.3 Filter

Geotextile. Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). **Under no circumstances may spun-bond or slit-film fabrics be allowed.** Each roll of geotextile shall be labeled

with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. Geotextiles shall not be exposed to sunlight prior to placement.

Granular Filters. Samples of granular filter material shall be tested for grain size distribution to ensure compliance with the gradation specification used in design. Sampling and testing frequency shall be in accordance with the owner or owner's authorized representative.

2.2.4 Subgrade Soils

When placement is in the dry, the riprap and filter shall be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Unsatisfactory soils shall be considered those soils having excessive in-place moisture content; soils containing roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils shall be removed, and the excavation backfilled with approved material that is compacted prior to placement of the riprap. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays, per the geotechnical engineer's recommendations.

2.3 Installation

2.3.1 Subgrade Preparation

The subgrade soil conditions shall meet or exceed the required material properties described in Section 2.2.4 prior to placement of the riprap. Soils not meeting the requirements shall be removed and replaced with acceptable material.

When placement is in the dry, the areas to receive the riprap shall be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the filter, and between the filter and the riprap. Stable and compacted subgrade soil shall be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes shall be compacted, shaped, and uniformly graded. The subgrade should be uniformly compacted to the geotechnical engineer's site-specific requirements.

When placement is under water, divers shall be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create voids beneath the system. Immediately prior to placement of the filter and riprap system, the prepared subgrade must be inspected.

2.3.2 Placing the Filter

Whether the filter comprises one or more layers of granular material or is made of geotextile, its placement should result in a continuous installation that maintains intimate contact with the soil beneath. Voids, gaps, tears, or other holes in the filter must be avoided to the extent practicable, and the filter must be replaced or repaired when they occur.

Placement of Geotextile. The geotextile shall be placed directly on the prepared area, in intimate contact with the subgrade. When a geotextile is placed, it should be rolled or spread out directly on the prepared area and be free of folds or wrinkles. The rolls shall not be dragged, lifted by one end, or dropped. The geotextile should be placed in such a manner that placement of the overlying materials (riprap and/or bedding stone) will not excessively stretch or tear the geotextile.

After geotextile placement, the work area shall not be trafficked or disturbed in a manner that might result in a loss of intimate contact between the riprap stone, the geotextile, and the subgrade. The geotextile shall not be left exposed longer than the manufacturer's recommendation

to minimize potential damage due to ultraviolet radiation; therefore, the overlying materials should be placed as soon as practicable.

The geotextile shall be placed so that upstream strips overlap downstream strips. Overlaps shall be in the direction of flow wherever possible. The longitudinal and transverse joints shall be overlapped at least 1.5 ft (46 cm) for dry installations and at least 3 ft (91 cm) for below-water installations. If a sewn seam is to be used for the seaming of the geotextile, the thread to be used shall consist of high strength polypropylene or polyester and shall be resistant to ultraviolet radiation. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or weights such as sandbags shall be used.

Placing Geotextiles Under Water. Placing geotextiles under water can be problematic for a number of reasons. Most geotextiles that are used as filters beneath riprap are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, meaning that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the riprap (Koerner 1998).

Flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric (a condition that contractors refer to as “galloping”) that are extremely difficult to control. The preferred method of controlling geotextile placement is to isolate the work area from river currents by a temporary cofferdam. In mild currents, geotextiles pre-cut to length can be placed by divers, with sandbags to hold the filter temporarily.

For partially grouted riprap at piers, sand-filled geocontainers made of non-woven, needle-punched fabric are particularly effective for placement under water as shown in Figure D1.5. The geotextile fabric and sand fill that compose the geocontainers should be selected in accordance with appropriate filter design criteria presented in Part 1 and placed such that they overlap to cover the required area. Geocontainers can be fabricated in a variety of dimensions and weights. Each geocontainer should be filled with sand to no more than 80% of its total volume so that it remains flexible and “floppy.” The geocontainers can also serve to fill a pre-existing scour hole around a pier prior to placement of the partially grouted riprap, as shown in Figure D1.5. For more information, see Lagasse et al. (2006, 2007).

Placement of Granular Filter. For placing a granular filter, front-end loaders are the preferred method for dumping and spreading the material on slopes milder than approximately 4H:1V. A typical minimum thickness for granular filters is 0.5 to 1.0 ft (0.15 to 0.3 m), depending on the size of the overlying riprap and whether a layer of bedding stone is to be used between the filter and the riprap. When a granular filter is placed under water, the thickness should be increased by 50%. Placing granular media under water around a bridge pier is best accomplished using a large-diameter tremie pipe to control the placement location and thickness, while minimizing the potential for segregation. **NOTE: For riverine applications where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.**

2.3.3 Placing the Riprap

Riprap may be placed from either land-based or water-based operations and can be placed under water or in the dry. Special-purpose equipment such as clamshells, orange-peel grapples, or hydraulic excavators (often equipped with a “thumb”) is preferred for placing riprap. Unless the riprap can be placed to the required thickness in one lift using dump trucks or front-end loaders, tracked or wheeled vehicles are discouraged from use because they can destroy the interlocking integrity of the rocks when driven over previously placed riprap.

Water-based operations may require specialized equipment for deep-water placement or can use land-based equipment loaded onto barges for near-shore placement. In all cases, riprap

should be placed from the bottom working toward the top of the slope so that rolling and/or segregation does not occur.

Riprap Placement on Geotextiles. Riprap should be placed over the geotextile by methods that do not stretch, tear, puncture, or reposition the fabric. Equipment should be operated to minimize the drop height of the stone without the equipment contacting and damaging the geotextile. Generally, this will be about 1 ft of drop from the bucket to the placement surface (ASTM D 6825). Further guidance on recommended strength properties of geotextiles as related to the severity of stresses during installation are provided in Part 1 of this appendix. When the preferred equipment cannot be utilized, a bedding layer of coarse granular material on top of the geotextile can serve as a cushion to protect the geotextile. Material composing the bedding layer must be more permeable than the geotextile to prevent uplift pressures from developing.

Riprap Placement Under Water. Riprap placed in water requires close observation and increased quality control to ensure a continuous well-graded uniform rock layer of the required thickness (ASTM D6825). A systematic process for placing and continuous monitoring to verify the quantity and layer thickness is important. Typically, riprap thickness is increased by 50% when placement must occur under water.

Excavation, grading, and placement of riprap and filter under water require additional measures. For installations of a relatively small scale, the stream around the work area can be diverted during the low-flow season. For installations on larger rivers or in deeper water, the area can be temporarily enclosed by a cofferdam, which allows for construction dewatering if necessary. Alternatively, a silt curtain made of plastic sheeting may be suspended by buoys around the work area to minimize potential environmental degradation during construction.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remote operated vehicles (ROVs) can provide some information about the riprap placement under water.

2.3.4 Placing the Grout

Table D2.1 presents the recommended values for quantity of grouting material as a function of the class (size) of the riprap. The quantities are valid for mechanically grouted, medium-dense armor layers with a thickness of 2 times the d_{50} size of the riprap stones. The application quantities should not be exceeded because too much grout can create an impermeable layer on the surface of the armor layer, or on the filter at the bottom of the riprap. In addition, the flexibility of an installation is reduced when application quantities greater than the recommended amount are used.

Two types of grouting procedures, line-by-line and spot-by-spot, produce the desired conglomerate-like elements in the riprap as shown in Figure D2.1. Spot grouting produces better results than line grouting. With a proper grout mixture and appropriate placement rate, par-

Table D2.1. Grouting material quantities.

Class of riprap	Application quantity	
	ft ³ /yd ²	L/m ²
Class II	2.0 – 2.2	70 – 85
Class III	2.7 – 3.2	90 – 110
Class IV	3.4 – 4.1	115 – 140

Notes:

- When riprap is positioned loosely (e.g., dumped stone), the application quantity should be increased by 15% to 25%.
- When stones are tightly packed (e.g., compacted or plated riprap), the application quantity should be decreased by 10%.

Source: derived from BAW (1990)



Figure D2.1. Conglomerate produced during spot grouting.

tial grouting can be reliably accomplished under water as well as in the dry. Grout placement can be done by hand only in water less than 3 ft (1 m) deep. Special devices are required for placement in deeper water. Various countries in Europe have developed special grout mixes and construction methods for underwater installation of partially grouted riprap (Lagasse et al. 2001).

Grout application and penetration will behave differently in dry conditions compared to underwater placement. Usually test boxes having a surface area of at least 10 ft² (1 m²) and a depth equal to the armor layer thickness are placed on the bed when placing partially grouted riprap under water, as shown in Figure D2.2 (Heibaum 2000). The underwater boxes are filled in the water with riprap, and then removed after being grouted to confirm that the proper areal coverage and penetration depths have been achieved.

2.3.5 Inspection

The subgrade preparation, geotextile placement and partially grouted riprap system and overall finished condition including termination trenches, if any, shall be inspected before accepting the work. Inspection guidelines for the partially grouted riprap installation are presented in detail in Part 3 of this document.

2.4 Measurement and Payment

Partially grouted riprap satisfactorily placed can be paid for based on either volume or weight. When a weight basis is used, commercial truck scales capable of printing a weight ticket including time, date, truck number, and weight should be used. When a volumetric basis is used, the in-place volume should be determined by multiplying the area, as measured in the field, of the surface on which the riprap was placed by the thickness of the riprap measured perpendicular as dimensioned on the contract drawings.

In either case, the finished surface of the riprap should be surveyed to ensure that the as-built lines and grades meet the design plans within the specified tolerance. Survey cross sections perpendicular to the axis of the structure are usually taken at specified intervals. All stone outside the limits and tolerances of the cross sections of the structure, except variations so minor as not



Figure D2.2. Test box used during underwater grout placement.

to be measurable, is deducted from the quantity of new stone for which payment is to be made. In certain cases, excess stone may be hazardous or otherwise detrimental; in this circumstance, the contractor must remove the excess stone at its own expense. Payment will be full compensation for all material, labor, and equipment to complete the work.

Part 3: Inspection, Maintenance, and Performance Evaluation

3.1 Inspection During Construction

Inspection during construction shall be conducted by qualified personnel who are independent of the contractor. Underwater inspection of partially grouted riprap scour countermeasures at piers shall be performed only by divers specifically trained and certified for such work.

3.1.1 Subgrade

Inspection of the subgrade shall be performed immediately prior to geotextile placement. The subgrade should be clean and free of projections, debris, construction materials, or other foreign objects that would prevent the filter from being properly placed. Likewise, there should be no potholes, rills, or other voids that the filter material might bridge over.

The subgrade material itself should not be muddy or frozen and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area shall be noted and photographed; observations of such should be brought to the attention of the Project Engineer as they may represent conditions that are different than those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

3.1.2 Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product designation. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics and slit-film geotextiles should **never** be used in riprap applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, rocks, or U-shaped soil staples may be used to hold the geotextile in position while the riprap is being placed. The riprap should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

3.1.3 Riprap

Inspection of riprap placement typically consists of visual inspection of the operation and the finished surface. Inspection must ensure that a dense, rough surface of well-keyed angular stones of the specified quality and gradation is obtained, that the layers are placed such that voids are minimized, and that the overall finished thickness meets specifications.

3.1.4 Grout

Each batch of grout should be tested for consistency and uniformity of the mix using the recommended test methods as described in Section 1.1.4. No dry clumps of Portland cement or aggregates shall be present in the mix. The rate of placement should be monitored to ensure that the application quantities are in conformance with the requirements of Section 2.3.4.

3.2 Periodic and Post-Flood Inspection

As a pier scour countermeasure, a partially grouted riprap system would typically be inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a specified magnitude.

Underwater inspection of a partially grouted riprap system shall be performed only by divers specifically trained and certified for such work. The following guidance for inspecting riprap is presented in the National Highway Institute (NHI) training course 135047, "Stream Stability and Scour at Highway Bridges for Bridge Inspectors," and has been modified for applicability to partially grouted riprap:

1. Riprap should be **angular and interlocking**. (Old bowling balls would not make good riprap. Flat sections of broken concrete slabs do not make good riprap.)
2. The partially grouted riprap countermeasure should have a **granular or synthetic geotextile filter** between the armor layer and the subgrade material.
3. Riprap stones should be **well graded** (a wide range of rock sizes). The maximum rock size should be no greater than about twice the median (d_{50}) size.
4. For bridge piers, partially grouted riprap should generally extend up to the bed elevation so that the top of the riprap is visible to the inspector during and after floods.
5. When partially grouted riprap at piers is inspected, affirmative answers to the following questions are strong indicators of problems:
 - Has the armor been fractured and broken to the extent that riprap stones or conglomerate particles have been **displaced** downstream?
 - Has angular riprap material been **replaced** over time by smoother river run material?
 - Has the riprap material physically **deteriorated, disintegrated**, or been **abraded** over time?

- Are there **holes or gaps** in the armor layer where the filter has been exposed or breached?
- Are there **voids** underneath the armor, or has the armor been **undermined** at its periphery?

3.3 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. As with any armor system, progressive failure from successive flows must be avoided by providing timely maintenance intervention. Where localized areas are limited to loss of stones or conglomerate particles, the area can be easily repaired by adding more riprap and re-grouting the new riprap area and making sure it in turn is grouted to the original armor adjacent to the repair.

Voids or undermining underneath the system are unlikely with partially grouted riprap, because it will fracture and settle. If such areas are detected, too much grout was used in the original installation, causing the partially grouted riprap to act as a rigid armor layer. Voids or undermined areas are best treated by obliterating them by mechanically breaking the older grouted rock into conglomerate particles. Any resulting depressions or gaps in the armor can then be brought back to grade by placing and re-grouting additional riprap with a more appropriate grout application quantity.

3.4 Performance Evaluation

The evaluation of any countermeasure's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. To properly assess the performance of a pier scour countermeasure, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the pier scour countermeasure. Present-day channel cross-section geometry and planform should be compared to those at the time of countermeasure installation. Both lateral and vertical instability of the channel in the vicinity of the bridge can significantly alter hydraulic conditions at the piers. Approach flows may become skewed to the pier alignment, causing greater local and contraction scour.

It is recognized that the person making the performance evaluation will probably not be the inspector; however, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported as a percentage of the initial capital improvement cost.

To guide the performance evaluation for partially grouted riprap as a pier scour countermeasure, a rating system is presented in this section. It establishes numerical ratings from 0 (worst) to 6 (best) for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the current condition of the countermeasure?

Tables D3.1 through D3.3 present a rating system for partially grouted riprap pier scour countermeasures. A single numerical score is not intended; rather, an independent rating (0-6 or U) is given for each of the three topical areas. Recommended actions corresponding to the rating codes are also provided.

Table D3.1. Rating system for partially grouted riprap: hydraulic history.

Code	Hydraulic History	Code	Hydraulic History
U	N/A	3	Moderate: The countermeasure has experienced one or more flows greater than the 10-year event.
6	Extreme: The countermeasure has experienced one or more flows greater than the 100-year event.	2	Low: The countermeasure has experienced one or more flows greater than the 5-year event.
5	Severe: The countermeasure has experienced one or more flows greater than the 50-year event.	1	Very Low: The countermeasure has experienced one or more flows greater than the 2-year event.
4	High: The countermeasure has experienced one or more flows greater than the 25-year event.	0	Negligible: The countermeasure has not experienced any flows greater than a 2-year event.

Table D3.2. Rating system for partially grouted riprap: maintenance history.

Code	Maintenance History	Code	Maintenance History
U	N/A	3	Moderate: The system has required occasional maintenance since installation.
6	None Required: No maintenance has been needed since installation.	2	High: Frequent maintenance has been required.
5	Very Low: The system has required maintenance for very small, local areas once or twice.	1	Very High: Significant maintenance is usually required after flood events.
4	Low: The system has required minor maintenance.	0	Excessive: The system typically requires maintenance every year.

Table D3.3. Rating system for partially grouted riprap: current condition.

Code	Description of Current Condition	Code	Description of Current Condition
U	The system is uninspectable, due to burial by sediment, debris, or other circumstance.	3	Fair: The system exhibits some missing particles as evidenced by irregular armor surface; localized voids and/or undermining observed.
6	Excellent: The system is in excellent condition, with no displacement of particles and no undermining. System is well abutted to pier with no gaps.	2	Poor: Obvious deterioration of the system has occurred. Gaps or holes are present that have exposed the underlying filter. Voids or undermining are observed under large areas of the system.
5	Very Good: The system exhibits only minor fracturing and there is no evidence of settlement or particle movement.	1	Badly Damaged: The system has experienced substantial deterioration in terms of broken and dislodged particles. The armor layer has separated from the pier, leaving gaps.
4	Good: The system exhibits some fracturing, with minor settlement and/or particle displacement observed.	0	Severe: The system has suffered damage such that it is no longer providing scour protection. The only recourse is to remove the remains of the installation and replace it with a redesigned countermeasure.

Recommended actions based on current condition rating:

Code U: The partially grouted riprap cannot be inspected. A plan of action should be developed to determine the condition of the installation. Possible remedies may include removal of debris, excavation during low flow, probing, or non-destructive testing using ground-penetrating radar or seismic methods.

Codes 6 or 5: Continue periodic inspection program at the specified interval.

Codes 4 or 3: Increase inspection frequency. The rating history of the installation should be tracked to determine if a downward trend in the rating is evident. Depending on the nature of the partially grouted riprap application, the installation of monitoring instruments might be considered.

Code 2: The maintenance engineer's office should be notified and maintenance should be scheduled. The cause of the low rating should be determined, and consideration given to redesign and replacement. Materials other than partially grouted riprap might be considered as a replacement.

Codes 1 or 0: The maintenance engineer's office should be notified immediately. Depending upon the nature of the partially grouted riprap application, other local officials and/or law enforcement agencies identified in the Plan of Action for the bridge may also need to be notified.

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APPENDIX E

Guidelines for Pier Scour Countermeasures Using Articulating Concrete Block (ACB) Systems

Introduction, E-2

1 Design and Specification, E-3

2 Construction, E-16

3 Inspection, Maintenance, and Performance Evaluation, E-21

References, E-30

Introduction

Articulating concrete block (ACBs) systems provide a flexible armor for use as a pier scour countermeasure. These systems consist of preformed concrete units that either interlock, are held together by cables, or both (Figure E1.1). After installation is complete, the units form a continuous blanket or mat. This design guideline considers the application of ACB systems as a pier scour countermeasure.

The term “articulating,” as used in this document, implies the ability of individual blocks of the system to conform to changes in the subgrade while remaining interconnected by virtue of block interlock and/or additional system components such as cables, ropes, geotextiles, or geogrids. ACB systems include interlocking and non-interlocking block geometries; cable-tied and non-cable-tied systems; and vegetated and non-vegetated systems. Block systems are typically available in both open-cell and closed-cell varieties.

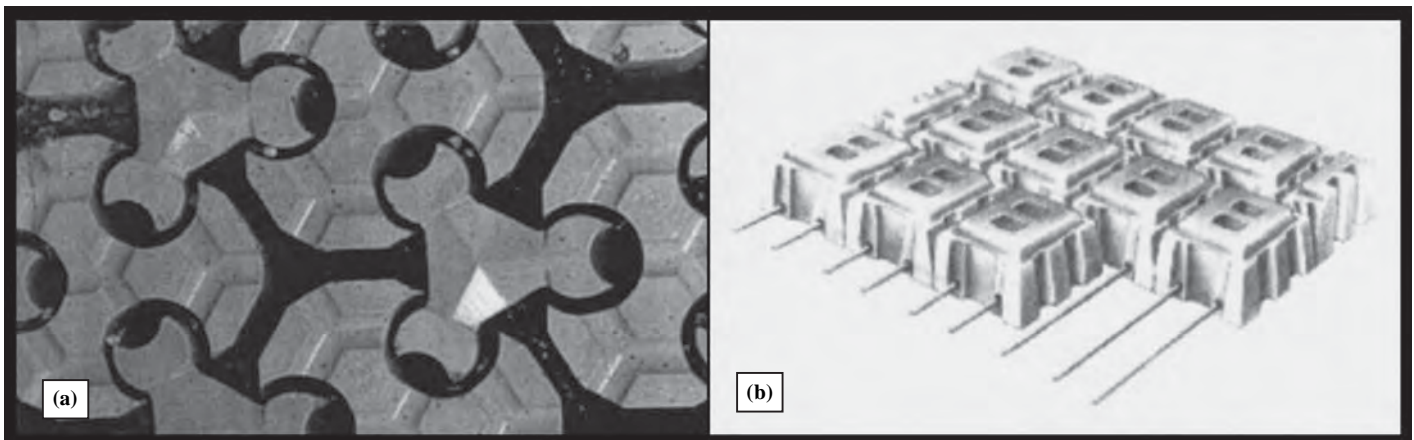
There is little field experience with the use of ACB systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for bank revetment and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. The guidance for pier scour applications provided in this document has been developed primarily from the results of NCHRP Project 24-07(2) (Lagasse et al. 2007).

It should be noted that manufacturers of ACB systems must test their products and develop performance data from the test results. Since ACB systems vary in shape and performance from one proprietary system to the next, each system will have unique performance characteristics.

In all cases, successful performance of ACBs depends on maintaining intimate contact between the block system and the subgrade under the hydraulic loading associated with the design event.

This document is organized into three parts:

- Part 1 provides design and specification guidelines for ACBs, given the appropriate performance data for any particular block system.
- Part 2 presents construction guidelines.
- Part 3 provides guidance for inspection, maintenance, and performance evaluation of ACB systems used as a pier scour countermeasure.



Source: (a) American Excelsior Company, (b) Armortec

Figure E1.1. Examples of interlocking block (a) and cable-tied block (b) systems.

Part 1: Design and Specification

1.1 Materials

An ACB system consists of a matrix of individual concrete blocks placed together to form an erosion-resistant armor layer with specific hydraulic performance characteristics. The system includes a filter layer, typically a geotextile, specifically selected for compatibility with the subsoil. The filter allows infiltration and exfiltration to occur while providing particle retention. The individual blocks must be dense and durable, and the matrix must be flexible and porous.

ASTM International has published D 6684 (2005) specifically for ACB systems. Table E1.1 lists some concrete properties required by this standard.

ASTM D 6684 also specifies minimum strength properties of geotextiles according to the severity of the conditions during installation. Harsh installation conditions (vehicular traffic, repeated lifting, realignment, and replacement of mattress sections, etc.) require stronger geotextiles.

1.2 Hydraulic Stability Design Procedure

The hydraulic stability of ACB systems is analyzed using a “discrete particle” approach. The design approach is similar to that introduced by Stevens and Simons (1971) as modified by Julien (1995) in the derivation of the factor of safety method for sizing rock riprap. In that method, a calculated factor of safety of 1.0 or greater indicates that the particles will be stable under the given hydraulic conditions. For ACBs, the factor of safety force balance has been recomputed considering the weight and geometry of the blocks, and the Shields relationship for estimating the particle’s critical shear stress is replaced with actual test results (Clopper 1989, 1992).

Considerations are also incorporated into the design procedure to account for the additional forces generated on a block that protrudes above the surrounding matrix because of subgrade irregularities or imprecise placement. The analysis methodology purposely omits any restraining forces due to cables, because any possible benefit that cables might provide are reflected in the performance testing of the block. Cables may prevent blocks from being lost entirely but they do not prevent a block system from failing through loss of intimate contact with the subgrade.

Similarly, the additional stability afforded by vegetative root anchorage or mechanical anchoring devices, while recognized as potentially significant, is ignored in the analysis procedures for the sake of conservatism in block selection and design.

1.2.1 Selecting a Target Factor of Safety

The designer must determine what factor of safety should be used for a particular application. Typically, a minimum allowable factor of safety of 1.2 is used for revetment (bank protection) when the project hydraulic conditions are well known and the installation can be conducted under well-controlled conditions. Higher factors of safety are typically used for protection at bridge piers, abutments, and channel bends because of the complexity in computing hydraulic conditions at these locations.

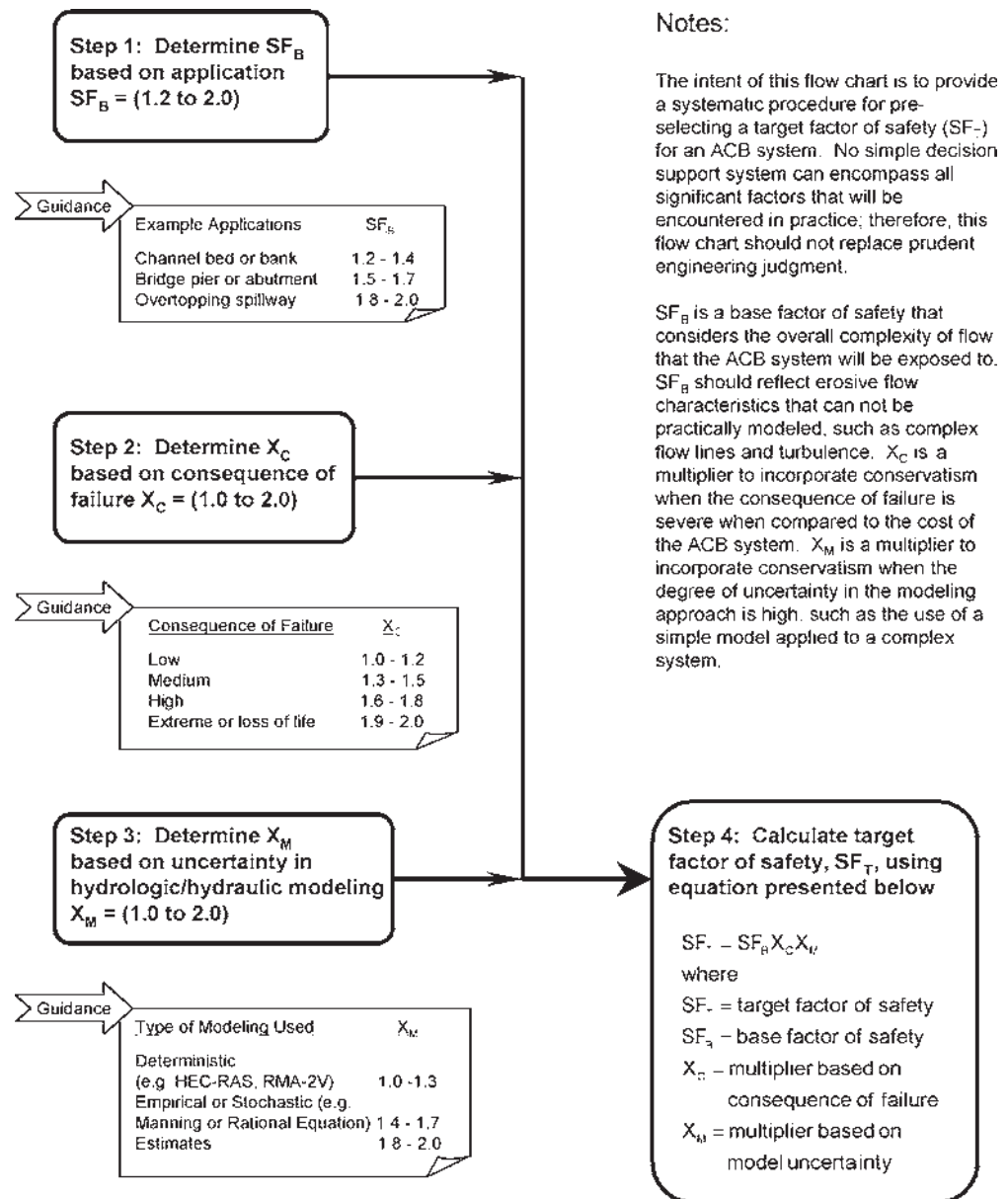
Table E1.1. Concrete properties required by ASTM D 6684.

Property	Average of 3 Units	Individual Unit
Minimum allowable compressive strength, lb/in ²	4,000	3,500
Maximum allowable water absorption, lb/ft ³ , (%)	9.1 (7.0%)	11.7 (9.4%)
Minimum allowable density in air, lb/ft ³	130	125
Freeze-thaw durability	As specified by owner in accordance with ASTM C 67, C 666, or C 1262	

The Harris County Flood Control District, Texas, (Ayres Associates 2001) has developed a simple flow chart approach that considers the type of application, uncertainty in the hydraulic and hydrologic models used to calculate design conditions, and consequences of failure to select an appropriate target factor of safety to use when designing an ACB installation. In this approach, the minimum allowable factor of safety for ACBs at bridge piers is 1.5. This value is then multiplied by two factors, each greater than 1.0, to account for risk and uncertainty. Figure E1.2 shows the HCFCFD flowchart method.

1.2.2 Design Method

The stability of a single block is a function of the applied hydraulic conditions (velocity and shear stress), the angle of the inclined surface on which it rests, and the weight and geometry of the block. Considering flow along a channel bank as shown in Figure E1.3, the forces acting on



Source: modified from Ayres Associates (2001)

Figure E1.2. Selecting a target factor of safety.

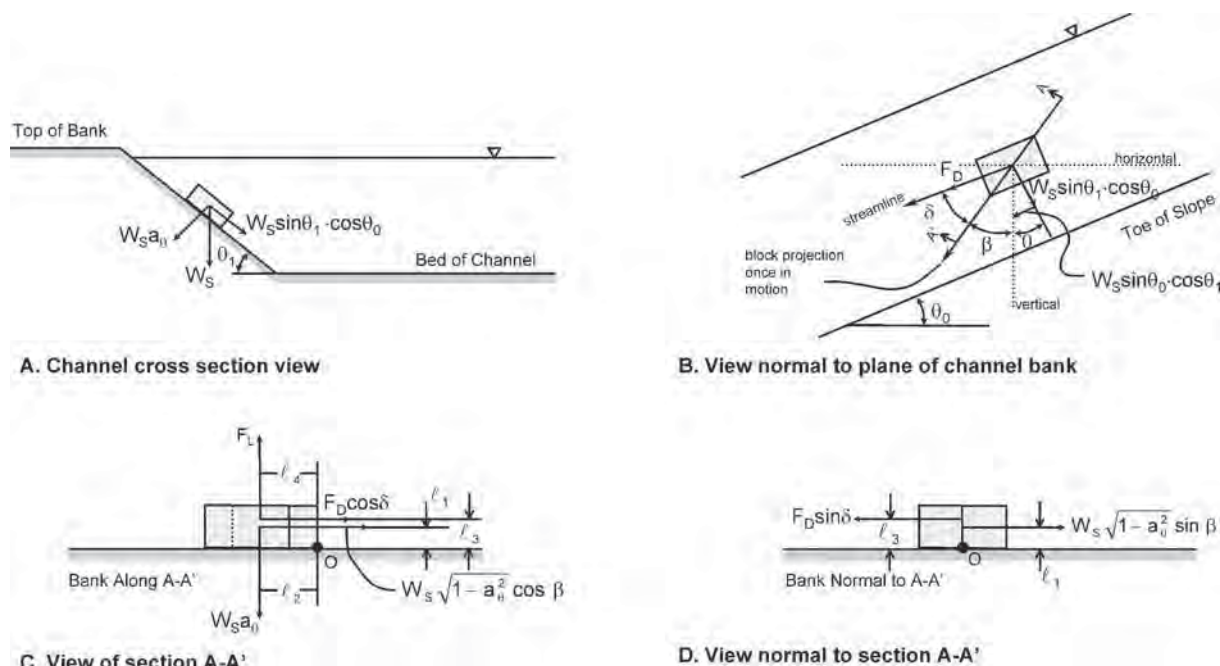


Figure E1.3. Three-dimensional view of a block on a channel side slope with factor of safety variables defined.

a concrete block are the lift force, F_L ; the drag force, F_D ; and the submerged weight of the block, W_S . Block stability is determined by evaluating the moments about the point, O , where rotation can take place. The components of forces are shown in Figure E1.3.

The safety factor (SF) for a single block in an ACB matrix is defined as the ratio of restraining moments to overturning moments (terms are defined in Table E1.2):

$$SF = \frac{\ell_2 W_S a_0}{\ell_1 W_S \sqrt{1 - a_0^2} \cos \beta + \ell_3 F_D \cos \delta + \ell_4 F_L + \ell_3 F'_D \cos \delta + \ell_4 F'_L} \quad (E1.1)$$

Note that additional lift and drag forces F'_L and F'_D are included to account for protruding blocks that incur larger forces due to impact. Dividing Equation E1.1 by $\ell_1 W_S$ and substituting terms yields the final form of the factor of safety equations as presented in Table E1.2. The equations can be used with any consistent set of units; however, variables are indicated here in U.S. customary units.

The moment arms ℓ_1 , ℓ_2 , ℓ_3 , and ℓ_4 are determined from the block dimensions shown in Figure E1.4. In the general case, the pivot point of overturning will be at the downstream corner of the block; therefore, the distance from the center of the block to the corner should be used for both ℓ_2 and ℓ_4 . Since the weight vector acts through the center of gravity, one half the block height should be used for ℓ_1 . The drag force acts both on the top surface of the block (shear drag) and on the body of the block (form drag). Considering both elements of drag, eight-tenths the height of the block is considered a reasonable estimate of ℓ_3 .

While charts have been developed to aid in the design of ACB systems, the charts generally are based on the assumption of a “perfect” installation (i.e., no individual blocks protrude into the flow). In reality, some placement tolerance must be anticipated and the factor of safety equation modified to account for protruding blocks. Because poor installation can cause blocks to exceed the design placement tolerance, the actual factor of safety can be greatly reduced and may lead to failure. Therefore, construction inspection becomes critical to successful performance of ACB systems.

Table E1.2. Design equations for ACB systems.

Equation	Term Definitions
$F_L' = F_D' = 0.5\rho b(\Delta z)(V_{des})^2$ (E1.2)	a_θ = Projection of W_s into plane of subgrade b = Block width normal to flow (ft) F_D', F_L' = added drag and lift forces due to protruding block (lb)
$\eta_0 = \frac{\tau_{des}}{\tau_c}$ (E1.3)	
$\theta = \arctan\left(\frac{\tan\theta_0}{\tan\theta_1}\right)$ (E1.4)	ℓ_x = Block moment arms (ft) γ_c = Concrete density, lb/ft ³ γ_w = Density of water, lb/ft ³ V_{des} = Design velocity (ft/s) W = Weight of block in air (lb) W_s = Submerged block weight (lb)
$a_\theta = \sqrt{(\cos\theta_1)^2 - (\sin\theta_0)^2}$ (E1.5)	
$\beta = \arctan\left(\frac{\cos(\theta_0 + \theta)}{\left(\frac{\ell_4}{\ell_3} + 1\right)\left(\frac{\sqrt{1-a_\theta^2}}{\eta_0(\ell_2/\ell_1)}\right) + \sin(\theta_0 + \theta)}\right)$ (E1.6)	Δz = Height of block protrusion above ACB matrix (ft) β = Angle between block motion and the vertical δ = Angle between drag force and block motion
$\delta = 90^\circ - \beta - \theta$ (E1.7)	η_0 = Stability number for a block on a horizontal surface
$\eta_1 = \eta_0\left(\frac{(\ell_4/\ell_3) + \sin(\theta_0 + \theta + \beta)}{(\ell_4/\ell_3) + 1}\right)$ (E1.8)	η_1 = Stability number for a block on a sloped surface
$W_s = W\left(\frac{\gamma_c - \gamma_w}{\gamma_c}\right)$ (E1.9)	θ = Angle between side slope projection of W_s and the vertical
$SF = \frac{(\ell_2/\ell_1)a_\theta}{\cos\beta\sqrt{(1-a_\theta)^2} + \eta_1(\ell_2/\ell_1) + \frac{(\ell_3F_D'\cos\delta + \ell_4F_L')}{\ell_1W_s}}$ (E1.10)	θ_0 = Channel bed slope (degrees)
	θ_1 = Side slope of block installation (degrees)
	ρ = Mass density of water (slugs/ft ³)
	τ_c = Critical shear stress for block on a horizontal surface (lb/ft ²)
	τ_{des} = Design shear stress (lb/ft ²)
	SF = Calculated factor of safety

Note: The equations cannot be solved for $\theta_1 = 0$ (i.e., division by 0 in Equation E1.4); therefore, a very small but non-zero side slope must be entered for the case of $\theta_1 = 0$.

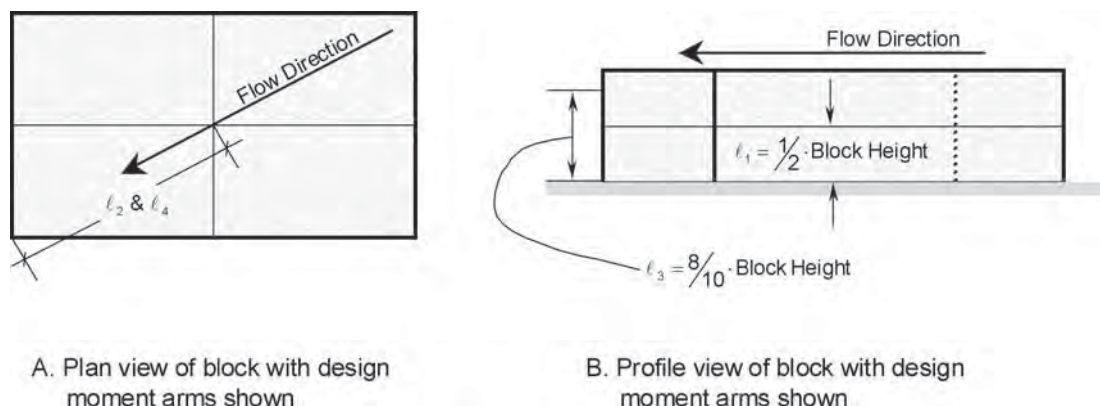


Figure E1.4. Schematic diagram of a block showing moment arms ℓ_1 , ℓ_2 , ℓ_3 , and ℓ_4 .

The design conditions in the immediate vicinity of a bridge pier are more severe than the approach conditions upstream; therefore, the *local* velocity and shear stress should be used in the design equations. As recommended in Hydraulic Engineering Circular No. 23 (Lagasse et al. 2001), the section-average approach velocity, V_{avg} , must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg} \quad (E1.11)$$

where

V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)

K_1 = Shape factor equal to 1.5 for round-nose piers and 1.7 for square-edged piers

K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for pier near the bank in a straight reach to 1.7 for pier located in the main current of flow around a sharp bend)

V_{avg} = Section-average approach velocity (Q/A) upstream of bridge, ft/s (m/s)

If the local velocity, V_{local} , is available from stream tube or flow distribution output from a one-dimensional (1-D) model, or directly computed from a two-dimensional (2-D) model, then only the pier shape coefficient should be used to determine the design velocity. The maximum local velocity is recommended since the channel could shift and the maximum velocity could impact any pier:

$$V_{des} = K_1 V_{local} \quad (E1.12)$$

The local shear stress at a pier should be calculated as

$$\tau_{des} = \left(\frac{n V_{des}}{K_u} \right)^2 \frac{\gamma_w}{y^{1/3}} \quad (E1.13)$$

where

τ_{des} = Design shear stress for local conditions at pier, lb/ft² (N/m²)

n = Manning's "n" value for block system

V_{des} = Design velocity as defined by Equation E1.11 or E1.12, ft/s (m/s)

γ_w = Density of water, 62.4 lb/ft³ (9,810 N/m³) for fresh water

y = Depth of flow at pier, ft (m)

K_u = 1.486 for U.S. customary units, 1 for SI units

1.3 Layout Dimensions

Based on small-scale laboratory studies performed for NCHRP Project 24-07(2) (Lagasse et al. 2007), the optimum performance of ACBs as a pier scour countermeasure was obtained when the blocks were extended a distance of at least 2 times the pier width in all directions around the pier.

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α , which is a function of the width, a , and length, L , of the pier (or pile bents) and the skew angle, α , as given below (after Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cos \alpha + L \sin \alpha}{a} \right)^{0.65} \quad (E1.14)$$

Where only clear-water scour is present, the ACB system may be placed horizontally such that the top of the blocks are flush with the bed elevation, with turndowns provided at the system periphery. However, when other processes or types of scour are present, the block system must be sloped away from the pier in all directions such that the depth of the system at its periphery

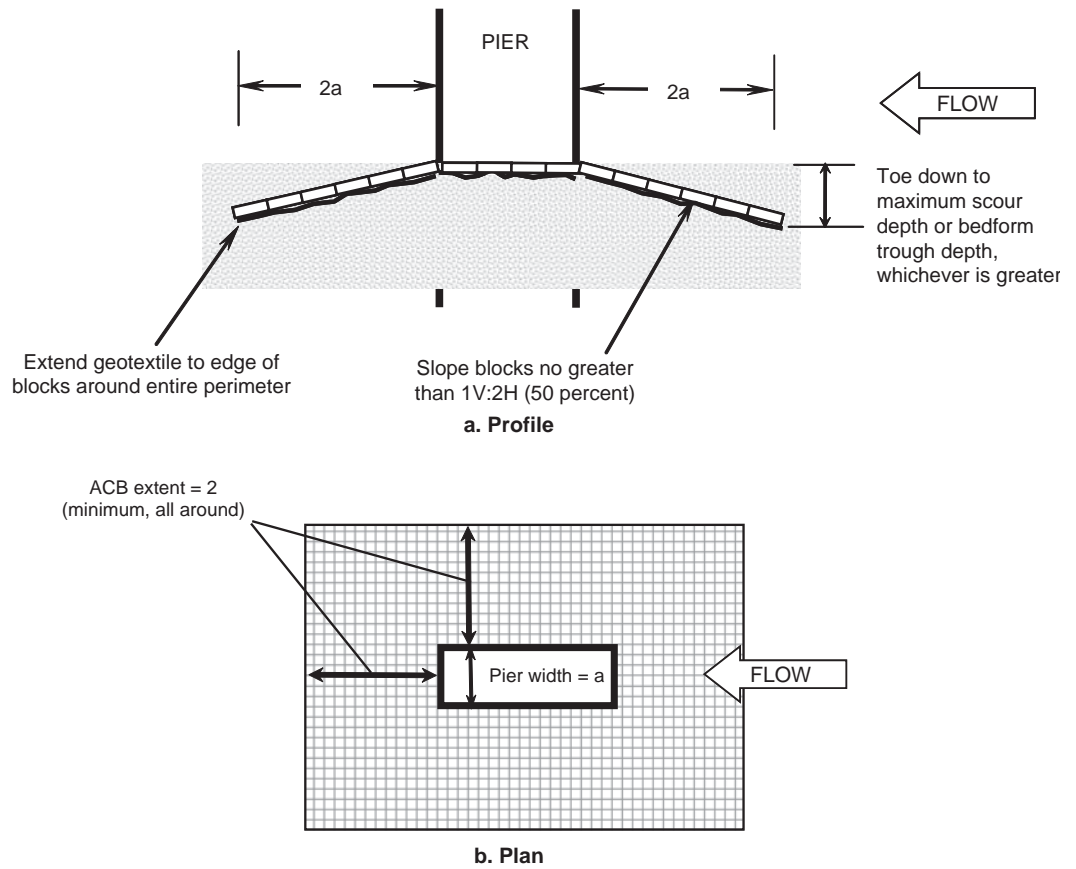


Figure E1.5. ACB layout diagram for pier scour countermeasures.

is greater than the depth of contraction scour and long-term degradation, or the depth of bedform troughs, whichever is greater (Figure E1.5). The blocks should not be laid on a slope steeper than 1V:2H (50%). In some cases, this limitation may result in blocks being placed further than two pier widths away from the pier.

In river systems where dune-type bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of Karim (1999) and/or van Rijn (1984). In general, an upper limit on the crest-to-trough height, Δ , is provided by Bennet (1997) as $\Delta < 0.4y$ where y is the depth of flow. This limit suggests that the maximum depth of the bed-form trough below ambient bed elevation will not exceed 0.2 times the depth of flow.

A filter is typically required for ACB systems at bridge piers. The filter should be extended fully beneath the ACB system. When a granular stone filter is used, the layer should have a minimum thickness of 4 times the d_{50} of the filter stone or 6 in. (15 cm), whichever is greater. The d_{50} size of the granular filter should be greater than one half the smallest dimension of the open cells in the block system. The granular filter layer thickness should be increased by 50% when placing under water.

1.4 Filter Requirements

The importance of the filter component of an ACB installation should not be underestimated. Geotextile filters are most commonly used with ACB systems. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of, the filter layer. *In cases where*

dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.

The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain *all* the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind.

1.4.1 Geotextile Filter Properties

Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.***

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand stresses during installation. These values are available from manufacturers. The following list briefly describes the most relevant properties:

- **Permeability.** The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimeters per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows perpendicularly through the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of (s)⁻¹. Permeability (and permittivity) is extremely important in filter design. For ACB installations, the permeability of the geotextile should be at least 10 times greater than that of the underlying material:

$$K_g > 10K_s \quad (E1.15)$$

where

K_g = Permeability of geotextile (cm/s)

K_s = Permeability of subgrade soil (cm/s)

- **Transmissivity.** The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second. This property is directly related to the drainage function, and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas non-woven, needle-punched fabrics have a much greater capacity due to their three-dimensional (3-D) microstructure. Transmissivity is not particularly relevant to filter design.
- **Apparent opening size (AOS).** Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.
- **Porosity.** Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Percent open area (POA).** POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.

- **Thickness.** As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).
- **Grab strength and elongation.** Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.
- **Tear strength.** Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.
- **Puncture strength.** Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

Table E1.3 provides the recommended characteristics for geotextile filters. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving ACBs at piers have been discussed here. As previously mentioned, geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and

Table E1.3. Recommended requirements for geotextile properties.

Test Designation	Property	Allowable value ⁽¹⁾		Comments
		Elongation < 50% ⁽²⁾	Elongation > 50% ⁽²⁾	
ASTM D 4632	Grab Strength	> 315 lbs (Class 1) > 250 lbs (Class 2) > 180 lbs (Class 3)	> 200 lbs (Class 1) > 160 lbs (Class 2) > 110 lbs (Class 3)	From AASHTO M 288
ASTM D 4632	Sewn Seam Strength ⁽³⁾	> 270 lbs (Class 1) > 220 lbs (Class 2) > 160 lbs (Class 3)	> 180 lbs (Class 1) > 140 lbs (Class 2) > 100 lbs (Class 3)	From AASHTO M 288
ASTM D 4533	Tear Strength ⁽⁴⁾	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4833	Puncture Strength	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4751	Apparent Opening Size	Per design criteria (Section 1.4 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 1.4 of this design guide)		Minimum allowable value
ASTM D 4355	Degradation by Ultraviolet Light	> 50% strength retained after 500 hours of exposure		Minimum allowable value
ASTM D 4873	Guide for Identification, Storage, and Handling			Provides information on identification, storage, and handling of geotextiles.
ASTM D 4759	Practice for the Specification Conformance of Geosynthetics			Provides information on procedures for ensuring that geotextiles at the jobsite meet the design specifications.

(1) Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class. The following descriptions have been modified from AASHTO M 288:

- Class 1 is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including when placement of riprap must occur in multiple lifts, when drop heights may exceed 1 ft (0.3 m) or when repeated vehicular traffic on the installation is anticipated.
- Class 2 is recommended for installation conditions where placement in regular, single lifts are expected and little or no vehicular traffic on the installation will occur, or when placing individual rocks by clamshell, orange-peel grapple or specially equipped hydraulic excavator with drop heights less than 1 ft.
- Class 3 is specified for the least severe installation environments, with drop heights less than 1 ft onto a bedding layer of select sand, gravel or other select imported material.

(2) As measured in accordance with ASTM D 4632.

(3) When seams are required.

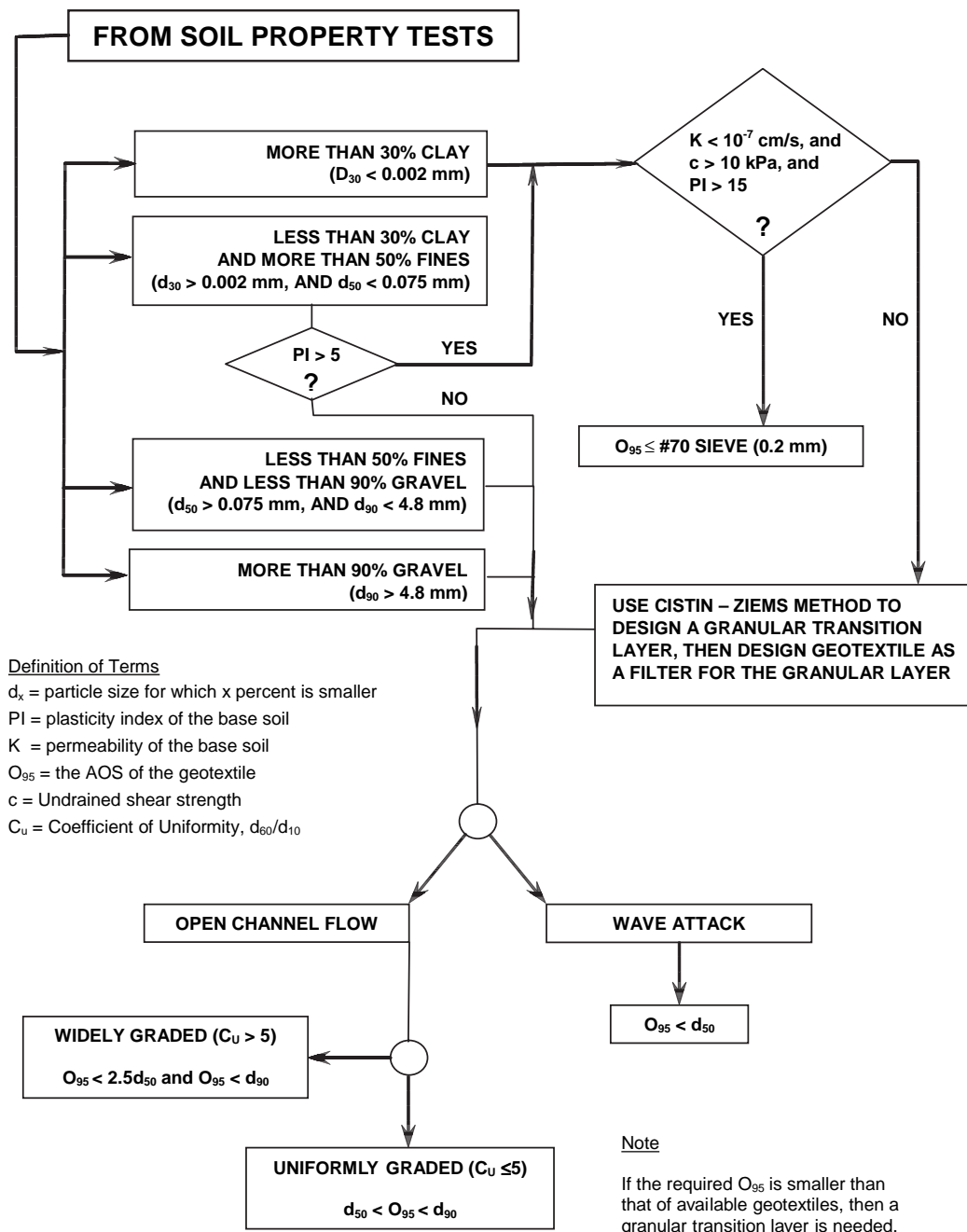
(4) The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 55 lbs. The MARV corresponds to a statistical measure whereby 2.5% of the tested values are less than the mean value minus two standard deviations (Koerner 1998).

covered by the ACB system, these stresses do not represent the environment that the geotextile will experience in the long term.

1.4.2 Geotextile Filter Design Procedure

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. The decision tree approach provided in Figure E1.6 assists in determining the appropriate soil retention criterion for the geotextile. The figure



Source: modified from Koerner (1998)

Figure E1.6. Geotextile selection based on soil retention.

includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater than 30% clay or is predominantly fine-grained soil (more than 50% passing the #200 sieve).

If a granular transition layer is required, the geotextile should be designed to be compatible with the properties of the granular layer. If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required. However, this requirement can be waived if the base soil exhibits the following conditions for hydraulic conductivity, K ; plasticity index, PI ; and undrained shear strength, c :

$$K < 1 \times 10^{-7} \text{ cm/s}$$

$$PI > 15$$

$$c > 10 \text{ kPa}$$

Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS less than a #70 sieve (approximately 0.2 mm) can be used with soils meeting these conditions and essentially functions more as a separation layer than a filter.

Step 3. Determine Permeability Criterion. The permeability criterion requires that the filter exhibit a permeability at least 4 times greater than that of the base soil (Koerner 1998) and, for critical or severe applications, at least 10 times greater (Holtz et al. 1995). Generally speaking, if the permeability of the base soil or granular filter has been determined from laboratory testing, that value should be used. If laboratory testing was not conducted, then an estimate of permeability based on the particle size distribution should be used.

To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s^{-1} . Typically, the designer will need to contact the geotextile manufacturer to obtain values of permeability, permittivity, and thickness.

Step 4. Select a Geotextile that Meets the Required Strength Criteria. Strength and durability requirements depend on the installation environment and the construction equipment that is being used. See Table E1.3 for recommended values based on AASHTO M 288, “Geotextile Specification for Highway Construction,” which provides guidance on allowable strength and elongation values for three categories of installation severity. For additional guidelines regarding the selection of durability test methods, refer to ASTM D 5819, “Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability.”

Step 5. Minimize Long-Term Clogging Potential. When a woven geotextile is used, its POA should be greater than 4% by area. If a non-woven geotextile is used, its porosity should be greater than 30% by volume. A good rule of thumb suggests that the geotextile having the largest AOS that satisfies the particle retention criteria should be used (provided of course that all other minimum allowable values described in this section are met as well).

1.4.3 Granular Filter Properties

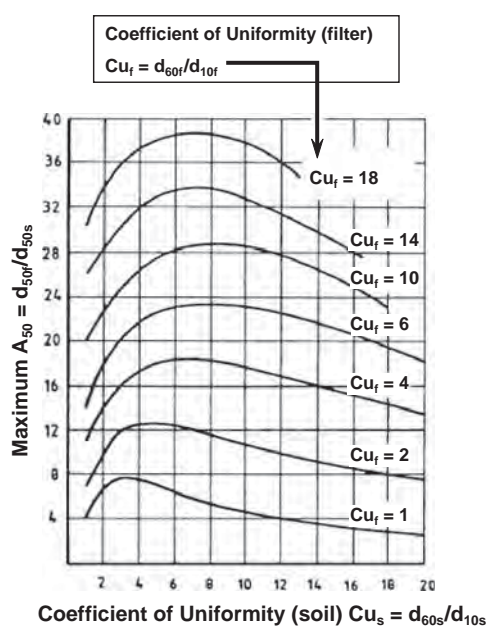
Granular filters are not often used with ACB systems, unless the bed material is coarse sand and gravel such that the granular filter can be composed of stones large enough to resist winnowing through the open cells of the blocks. However, in some circumstances the bed material is composed of fine silt and/or clays with low cohesion, such that the particle retention requirements of Figure E1.6 cannot be met with standard geotextiles. In these cases, it is recommended that a transition layer of 6 in. (15 cm) of sand be placed on the fine bed material and a geotextile selected for compatibility with the sand layer.

The method of Cistin and Ziems as described in *NCHRP Report 568* (Lagasse et al. 2006) is recommended for selecting a sand transition layer that is compatible with very fine, non- or low-cohesive bed sediments. This method is based on the coefficients of uniformity of the bed sedi-

ment and the sand transition layer to determine the maximum allowable ratio $d_{50(\text{sand})}$ to $d_{50(\text{bed})}$. The design chart for selecting a sand transition layer is provided as Figure E1.7.

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters can be used alone or can serve as a transitional layer between a predominantly fine-grained base soil and a geotextile. The following list briefly describes the most relevant properties:

- **Particle size distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziems whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure E1.7 provides a design chart based on the Cistin–Ziems approach.
- **Permeability.** Permeability of a granular filter material is determined by laboratory test or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For ACB installations, the permeability of the granular filter should be at least 10 times greater than that of the underlying material.
- **Porosity.** Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.
- **Thickness.** Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. is specified. For placement under water, thickness should be increased by 50%.
- **Quality and durability.** Aggregate used for a granular filter should be hard, dense, and durable.



Source: Heibaum (2004)

Figure E1.7. Granular filter design chart according to Cistin and Ziems.

1.4.4 Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter. That approach was developed for subsoils consisting of well-graded sands and may not be widely applicable to other soil types. An alternative approach that is considered more robust in this regard is the Cistin–Ziems method.

The suggested steps for proper design of a granular filter using this method are outlined below. Note that the subscript “s” is used to represent the base (finer) soil, and “f” is used to represent the filter (coarser) layer.

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Key Indices for Base Soil. From the grain size information, determine the median grain size, d_{50} , and the coefficient of uniformity, $Cu_s = d_{60}/d_{10}$, of the base soil.

Step 3. Determine Key Indices for Granular Filter. One or more locally available aggregates should be identified as potential candidates for use as a filter material. The d_{50} and coefficient of uniformity, $Cu_f = d_{60}/d_{10}$, should be determined for each candidate filter material.

Step 4. Determine Maximum Allowable d_{50} for Filter. Enter the Cistin–Ziems design chart (Figure E1.7) with the coefficient of uniformity, Cu_s , for the base soil on the x-axis. Find the curve that corresponds to the coefficient of uniformity, Cu_f , for the filter in the body of the chart and, from that point, determine the maximum allowable A_{50} from the y-axis. Compute the maximum allowable d_{50f} of the filter using d_{50fmax} equals A_{50max} times d_{50s} . Check to see if the candidate filter material conforms to this requirement. If it does not, continue checking alternative candidates until a suitable material is identified.

Step 5. Check for Permeability. From laboratory permeameter tests or the grain size distribution of the candidate filter material, determine whether the hydraulic conductivity of the filter is at least 10 times greater than that of the subsoil.

Step 6. Check for Compatibility with the ACB System. The accepted rule of thumb for granular material that directly underlies the ACB system is that it should have a d_{50} greater than 1/2 the smallest dimension of the open cells of the ACB system. This criterion will minimize winnowing of the granular material out through the open cells. If this criterion is not met, then multiple layers of granular filter materials should be considered.

Step 7. Filter Layer Thickness. For practicality of placement, the nominal thickness of a single filter layer should not be less than 6 in. (15 cm). Single-layer thicknesses up to 15 in. (38 cm) may be warranted where irregularities in the subgrade need to be smoothed out prior to placement of the ACB system. When multiple filter layers are required, each individual layer should range from 4 to 8 in. (10 to 20 cm) in thickness (Brown and Clyde 1989).

1.5 Guidelines for Seal Around the Pier

An observed point of failure for ACB systems at bridge piers occurs at the seal where the mat meets the bridge pier. During NCHRP Project 24-07, securing the geotextile to the pier prevented the leaching of the bed material from around the pier (Parker et al. 1998). This procedure worked successfully in the laboratory, but constructability implications must be considered when this technique is used in the field, particularly when the mat is placed under water. During flume studies at the University of Windsor (McCorquodale et al. 1993) and for the NCHRP Project 24-07(2) study (Lagasse et al. 2007), the mat was grouted to the pier.

A grout seal is not intended to provide a structural attachment between the mat and the pier, but instead is a simple method for plugging gaps to prevent bed sediments from winnowing out

from beneath the system. In fact, structural attachment of the mat to the pier is strongly discouraged. The transfer of moments from the mat to the pier may affect the structural stability of the pier, and the potential for increased loadings on the pier must be considered. When a grout seal is placed under water, an anti-washout additive is required.

The State of Minnesota Department of Transportation (Mn/DOT) has installed a cabled ACB mat system for a pier at TH 32 over Clearwater River at Red Lake Falls, Minnesota. Mn/DOT suggested that the riverbed could be excavated around the piers to the top of the footing. The mat could be put directly on top of the footing and next to the pier with concrete placed underneath, on top of, or both, to provide a seal between mat and pier.

The State of Maine Department of Transportation (MDOT) has designed an ACB system for a pier at Tukey's Bridge over Back Cove. MDOT recommended a design in which grout bags were placed on top of the mat at the pier location to provide the necessary seal.

1.6 Anchors

Mn/DOT also recommends the use of anchors when installing a cabled ACB mat, although, as discussed in Section 1.2, no additional stability is attributed to the cables themselves. Mn/DOT requires duckbill-type soil anchors placed 3 to 4 ft deep at the corners of the ACB mats, and at regular intervals of approximately 8 ft on center-to-center spacing throughout the area of the installation.

In reality, if uplift forces on an ACB system were great enough to create tension in the cables, then soil anchors could provide a restraining force that is transmitted to a group of blocks in the matrix. Using the same reasoning, anchors would be of no use in an uncabled system, unless there was a positive physical vertical interlock from block to block in the matrix. It should be noted that the stability analysis procedure presented in Section 1.2.2 is intended to ensure that uplift forces do not exceed the ACB system's capability, irrespective of cables.

The layout guidance presented in Section 1.3 indicates that the system should be toed down to a termination depth at least as deep as any expected contraction scour and long-term degradation, or bed form troughs, whichever is greater. Where such toedown depth cannot be achieved, for example where bedrock is encountered at shallow depth, a cabled system with anchors along the front (upstream) and sides of the installation is recommended. The spacing of the anchors should be determined based on a factor of safety of at least 5.0 for pullout resistance based on calculated drag on the exposed leading edge. Spacing between anchors of no more than 4 ft (1.3 m) is recommended. The following example is provided:

Given

$$\begin{aligned}\rho &= \text{Mass density of water (slugs/ft}^3\text{)} &&= 1.94 \\ V &= \text{Approach velocity, ft/s} &&= 10 \\ \Delta z &= \text{Height of block system, ft} &&= 0.5 \\ b &= \text{Width of block installation (perpendicular to flow), ft} &&= 40\end{aligned}$$

Step 1: Calculate total drag force, F_d , on leading edge of system:

$$F_d = 0.5\rho V^2(\Delta z)(b) = 0.5(1.94)(10^2)(0.5)(40) = \mathbf{1,940 \text{ lb}}$$

Step 2: Calculate required uplift restraint using 5.0 safety factor:

$$F_{\text{restraint}} = 5.0(1,940) = \mathbf{9,700 \text{ lb}}$$

Step 3: Counting anchors at corners of system, calculate required pullout resistance per anchor (rounded to nearest 10 lb):

a) Assume 11 anchors at 4-ft spacing: $9,700 \text{ lb}/11 \text{ anchors} = \mathbf{880 \text{ lb/anchor}}$

b) Assume 21 anchors at 2-ft spacing: $9,700 \text{ lb}/21 \text{ anchors} = \mathbf{460 \text{ lb/anchor}}$

Anchors should never be used as a means to avoid toeing the system down to the full required extent where alluvial materials are present at depth. In this case, scour or bed-form troughs will simply undermine the anchors as well as the system in general.

Part 2: Construction

The guidance in this section has been developed to facilitate the proper installation of ACB systems to achieve suitable hydraulic performance and maintain stability against hydraulic loads. The proper installation of ACB systems is essential to the adequate functioning and performance of the system during the design hydrologic event. Guidelines are provided herein for maximizing the correspondence between the design intent and the actual field-finished conditions of the project.

This section addresses the preparation of the subgrade, geotextile placement, ACB system placement, backfilling and finishing, and measurement and payment.

2.1 General Guidelines

The contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. This responsibility is typically performed by the owner's engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see ER 1180-1-6 [U.S. Army Corps of Engineers 1995]) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for ACB placement are included in the project plans and specifications. Standard ACB mat specifications and layout guidance are found in Part 1 of this appendix. Recommended requirements for the ACBs, including the tests necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications are provided.

Inspection of ACB placement consists of visual inspection of the placement operation and the finished surface. Inspection and quality assurance must be carefully organized to ensure that materials delivered to the job site meet the specifications. Acceptance of the work should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject incorrect or unsuitable materials (e.g., broken blocks, wrong geotextile, etc.) at the job site. Material that has been improperly placed should also be rejected throughout the duration of the contract, and require removal and replacement at the contractor's expense. Rejected material should be removed from the project site.

Construction techniques can vary tremendously due to the following factors:

- Size and scope of the overall project
- Size and weight of the materials
- Placement under water or in the dry
- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions
- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general infor-

mation regarding construction of ACB installations and some basic information and description of techniques and processes involved.

2.2 Materials

2.2.1 Blocks

Materials composing the ACB system shall be in accordance with the guidance provided in Part 1 of this appendix. Blocks shall be sound and free of defects that would interfere with proper placement or that would impair the integrity of the system. Blocks with the following defects shall be discarded:

- Broken blocks
- Blocks having chips larger than 2 in. (50 mm) in any dimension
- Blocks having cracks wider than 0.02 in. (0.5 mm) and longer than one-third the nominal height of the block

Minor cracks incidental to the usual method of manufacture or chipping that results from customary handling during shipping, delivery, and placement will not be deemed grounds for rejection.

2.2.2 Geotextile

Each roll of geotextile shall be labeled with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. The rolls shall not be dragged, lifted by one end, or dropped. Geotextiles shall not be exposed to sunlight prior to placement.

2.2.3 Cable

Cable may be composed of polyester, stainless steel, or galvanized steel unless otherwise specified. Cable used for preassembled mats shall be sufficiently sized and fastened for the size and weight of the assembled mats such that the mats can be placed in compliance with Occupational Safety and Health Administration (OSHA) requirements. The manufacturer shall be responsible for determining the minimum allowable cable strength compatible with mat size and weight to assure safe handling. The cable strength shall be based on a minimum factor of safety of 5 for mat lifting and shall include appropriate reduction factors for mechanically crimped cables, clamps, or other fasteners. Any systems that rely on the geotextile as a carrier fabric instead of cables must also meet the applicable portions of this section, with particular attention given to the grab points.

2.2.4 Subgrade Soils

When placement is in the dry, the ACB system shall be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Smoothing the subgrade prior to block placement is required. Unsatisfactory soils shall be considered those soils having excessive in-place moisture content; soils containing roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils must be removed, and the excavation backfilled with approved material that is compacted prior to placement of the block system. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays, per the geotechnical engineer's recommendations.

When a block system is placed under water, compaction of the subgrade is impractical. However, the surface must be relatively smooth, with no abrupt irregularities that would prevent intimate contact between the ACB system and the subgrade. Under no circumstances may an ACB system be draped over boulders, bridged over subgrade voids, or placed over other irregularities that would prevent achievement of intimate contact between the system and the

subgrade. Placing a layer of bedding stone may assist in achieving a suitable surface on which to place the block system.

2.3 Installation

2.3.1 Subgrade Preparation

Stable and compacted subgrade soil shall be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes shall be compacted, shaped, and uniformly graded to facilitate the development of intimate contact between the ACB system and the underlying grade. Termination between the ACB revetment system and a concrete slab, footer, pier, wall, or similar structure shall be sealed in a manner that prevents soil migration.

The subgrade soil conditions shall meet or exceed the required material properties described in Section 2.2.4 prior to placement of the block. Soils not meeting the requirements shall be removed and replaced with acceptable material.

When placement is in the dry, the areas to receive the ACB system shall be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the geotextile, and between the geotextile and the bottom surface of the ACB system. It is recommended that the subgrade be uniformly compacted to a minimum of 90% of Standard Proctor density (ASTM D 698). If the subgrade surface for any reason becomes rough, corrugated, uneven, textured, or traffic marked prior to ACB installation, such unsatisfactory portion shall be scarified, reworked, recompacted, or replaced as directed by the engineer. Grading tolerance shall be within 2 in. (50 mm) from the prescribed elevations, with no abrupt variations that would cause unacceptable projections of individual blocks.

When placing underwater, divers shall be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create irregularities in the block surface, or that would create voids beneath the system, in accordance with section 2.2.4. Immediately prior to placing the geotextile and ACB system, the prepared subgrade shall be inspected.

2.3.2 Placement of the Geotextile

The geotextile shall be placed directly on the prepared area, in intimate contact with the subgrade and free of folds or wrinkles. The geotextile shall be placed in such a manner that placement of the overlying materials will not excessively stretch or tear the geotextile. After geotextile placement, the work area shall not be trafficked or disturbed so as to result in a loss of intimate contact between the concrete block, the geotextile, and the subgrade. The geotextile shall not be left exposed longer than the manufacturer's recommendation to minimize potential damage due to ultraviolet radiation.

The geotextile shall be placed so that upstream strips overlap downstream strips and so that upslope strips overlap downslope strips. Overlaps shall be in the direction of flow wherever possible. The longitudinal and transverse joints shall be overlapped at least 3 ft (91 cm) for below-water installations and at least 1.5 ft (46 cm) for dry installations. If a sewn seam is to be used for the seaming of the geotextile, the thread to be used shall consist of high-strength polypropylene or polyester and shall be resistant to ultraviolet radiation. For bank protection, the geotextile shall extend beyond the top, toe and side termination points of the revetment. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or weights shall be used.

If the system is to be placed under water, the geotextile shall be securely attached to the bottom of the preassembled ACB mat prior to lifting with crane and spreader bar. In shallow water

where velocities are low, the geotextile may be placed under water and held in place temporarily with weights until the blocks are placed.

2.3.3 Placement of the ACB System

General ACB Placement. Placement of the ACB system, whether as mats or as individual blocks by hand, shall be performed to ensure that each block lies in intimate contact with the geotextile and subgrade. For blocks within a mat and individual blocks that are hand placed, the joint spacing between adjacent blocks is to be maintained so that binding of blocks does not occur and so that block-to-block interconnection is achieved. In areas of curvature or grade change, alignment of an individual block with adjacent blocks shall be oriented such that intimate contact between the block, geotextile, and subgrade is maintained and block-to-block interconnection is achieved.

Care shall be taken during block installation to avoid damage to the geotextile or subgrade during the installation process. Mats or individual blocks shall not be pushed or pulled laterally once they are on the geotextile. Preferably, where the geotextile is laid on the ground prior to the ACB installation, the ACB placement shall begin at the upstream section and proceed downstream. If an ACB system is to be installed starting downstream and proceeding in the upstream direction, a contractor option is to construct a temporary toe trench at the front edge of the ACB system to protect against flow that could otherwise undermine the system during flow events that may occur during construction. On sloped sections where practical, placement shall begin at the toe of the slope and proceed upslope. Block placement shall not bring block-to-block interconnections into tension. Individual blocks within the plane of the finished system shall not exceed a protrusion greater than the tolerance referenced in the contract documents.

If assembled and placed as mats, the ACB mats can be attached to a spreader bar to aid in the lifting and placing of the mats in their proper position with a crane or backhoe. The mats shall be placed side by side and/or end to end, so that the mats abut each other. Mat seams or openings between mats that are 2 in. (50mm) or greater in the matrix shall be filled with grout. Whether the blocks are placed individually or as mats, distinct grade changes shall be accommodated with a well-rounded transition (i.e., minimum radius per specific system characteristics). However, if a discontinuous revetment surface exists in the direction of flow, a grout seam at the grade change location shall be provided to produce a continuous, flush finished surface. Mats may be cut using a concrete saw where mitered joints are required. Partial blocks less than one half of a full-size block shall be removed, and the resulting gaps along the joint shall be filled with grout. Mats must never be overlapped on top of one another.

ACB Placement Under Water. ACB systems placed in water require close observation and increased quality control to ensure a continuous countermeasure system. A systematic process for placing and continuous monitoring to verify the quantity and layer thickness is important.

Excavation, grading, and placement of ACBs and filter under water require additional measures. For installations of a relatively small scale, the stream around the work area may be diverted during the low-flow season. For installations on larger rivers or in deeper water, the area can be temporarily enclosed by a cofferdam, which allows for construction dewatering if necessary. Alternatively, a silt curtain made of plastic sheeting may be suspended by buoys around the work area to minimize environmental degradation during construction. ACBs can be assembled in the dry, and a crane and spreader bar can be used to lift and place the system under water. Once under water and in the correct positions, the individual mats can be cabled together by divers.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remote operated vehicles (ROVs) can provide some information about ACB placement under water.

2.4 Finishing

2.4.1 System Termination

Termination of the ACB system shall be either (1) in excavated trenches that are properly backfilled with approved material flush with the top of the finished surface of the blocks or (2) abutted to a structural feature such as a pier, footing, or pile cap. In the case of blocks abutting a structural feature, the gap between the blocks and the structure shall be filled with cast-in-place concrete or grout, and finished flush with the top surface of the ACB system.

2.4.2 Concrete Joints

The use of cast-in-place concrete joints shall be minimized to the extent practicable. The following joints shall require concrete:

- Joints between cabled mats where the joint is more than 2 in. (50 mm) wider than the nominal joint of the particular ACB system
- Joints where block interlock is discontinuous, for example where mats are saw cut to accommodate bends or structural features
- Locations where the ACB system abuts a structural feature
- Areas where there are partial blocks (to avoid small elements that have reduced hydraulic stability)

2.4.3 Anchors

If soil anchors are used, they may be either helical or duckbill type. Anchors must be capable of being attached directly to the blocks, or to the ACB system cable. Anchors shall have the capability of being load tested to ensure that the specified pullout capacity is achieved. Anchor penetrations through the geotextile shall be sealed with cast-in-place concrete or structural grout to prevent migration of subsoil through the penetration point.

2.4.4 Backfilling the ACB System

The open area of the ACB system is typically either backfilled with suitable soil for revegetation, or with 3/8- to 3/4-inch (10-mm to 20-mm) crushed rock aggregate. Backfilling with soil or granular fill within the cells of the system shall be completed as soon as practicable after the revetment has been installed. When topsoil is used as a fill material above the normal waterline, the ACB system should be overfilled by 1 to 2 in. (25 to 50mm) to account for backfill material consolidation.

2.4.5 Inspection

The subgrade preparation, geotextile placement, ACB system installation, and overall finished condition including termination trenches shall be inspected before work acceptance. Inspection guidelines are presented in detail in Part 3 of this appendix.

2.5 Measurement and Payment

Measurement of the ACB system for payment shall be made on the basis of surface area. The pay lines will be neat lines taken off the contract drawings and will include embedded blocks and/or blocks placed in termination trenches. The work includes grading and preparatory work, furnishing and installing the geotextile and ACB system, backfilling the system, securing cables and fasteners, installing soil anchors, and seeding (where specified). Payment will be made at the

respective unit price per square foot. Payment will be full compensation for all material, labor, and equipment to complete the work.

Part 3: Inspection, Maintenance, and Performance Evaluation

3.1 Inspection During Construction

Inspection during construction shall be conducted by qualified personnel who are independent of the contractor. Underwater inspection of an ACB system shall be performed only by divers specifically trained and certified for such work.

3.1.1 Subgrade

Inspection of the subgrade shall be performed immediately prior to geotextile placement. The construction inspector should be alert to any condition that could cause the ACB system to not be in intimate contact with the subgrade, even if only in small, localized areas. The subgrade should be clean and free of debris, rocks, construction materials, or other foreign objects that would prevent the blocks from being firmly seated. Likewise, there should be no potholes, rills, or other voids that the blocks could bridge over.

The subgrade material itself should not be muddy or frozen and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area shall be noted and photographed; observations of such should be brought to the attention of the project engineer as they may represent conditions that are different than those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

3.1.2 Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product designation. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics and slit-film geotextiles should **never** be used in ACB applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, extra concrete blocks, or U-shaped soil staples may be used to hold the geotextile in position while the blocks are being placed. The blocks should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

3.1.3 Blocks

The inspector shall check the blocks to ensure that they are sound and are not excessively cracked or chipped. Interlocking blocks are typically hand placed and should be installed such that the interlock is not brought into tension. The block-to-block joints should be neutrally spaced such that there is equal free-play in all directions for the joint to be able to open and close.

If the block pattern becomes skewed to an extent that blocks bind or protrude above the allowable placement tolerance, the placed ACB that is determined to be out of tolerance shall be removed and replaced. The inspector must be aware that in cases where warped subgrade slopes or structural elements cause the joint pattern to become skewed, cast-in-place concrete joints may be field located in concurrence with the project engineer.

When pre-assembled mats are placed, the mats should abut one another as tightly as practicable. Mats should never be dragged laterally across the geotextile. If the mattress is not aligned properly, it must be lifted before being repositioned. Mats should never be allowed to overlap one another. Gaps between mats, or between mats and structural features, that are more than 2 in. (50 mm) greater than the nominal system spacing shall be filled with cast-in-place concrete or structural grout.

Unless specifically intended as part of the design, vehicle traffic should not be allowed on either the geotextile or the blocks. If the inspector notices vehicle traffic on the installation, the project engineer should be notified and should clearly identify which pieces of equipment are allowed on the block system and which pieces are not. Usually, light rubber-tired equipment can be tolerated, whereas heavier vehicles and tracked vehicles cannot.

3.2 Periodic and Post-Flood Inspection

As a pier scour countermeasure, the ACB system is typically inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a specified magnitude.

The most important aspect of inspecting an ACB installation is to determine if the system is maintaining intimate contact with the subgrade. In the dry, this can be readily determined provided the system has not been buried by sediment. The inspector should look for the following:

- Cracked, broken, or missing blocks
- Protruding blocks
- Overturned blocks, particularly along the periphery of the system or near structural features such as piers, footers, or pile caps
- Irregularities in the surface of the system (e.g., bumps or depressions)
- Voids beneath the blocks or the geotextile
- Deteriorated blocks (e.g., freeze-thaw or wet-dry weathering)

A length of reinforcing bar is helpful in detecting voids beneath the system. It can be used as a probe to poke into the cells of the blocks or between individual blocks in a system. The inspector can also use it to thump the blocks and listen for a hollow ringing sound that would indicate the presence of a void beneath the system.

Underwater inspection of an ACB system shall be performed only by divers specifically trained and certified for such work. Again, the use of a bar as a probe or thumper is particularly helpful. Whether visually or by feel, the diver should pay particular attention to the areas where the ACB abuts structural elements to ensure that no gaps exist and that subgrade material is not being removed from beneath the ACB system. Also, the perimeter of the installation should be examined to determine if there are any areas where mats, or portions of mats, have been undermined or overturned.

Figures E3.1 through E3.5 are provided as examples of inspection-related topics specific to ACB systems, as discussed in this section.

3.3 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. Because ACB systems are essentially an armor layer that is only one particle thick, any localized area of displaced blocks or voids beneath the system is vulnerable to further destabilization during the next high-flow event. As with any armor system, progressive failure from successive flows must be avoided by providing timely maintenance intervention.

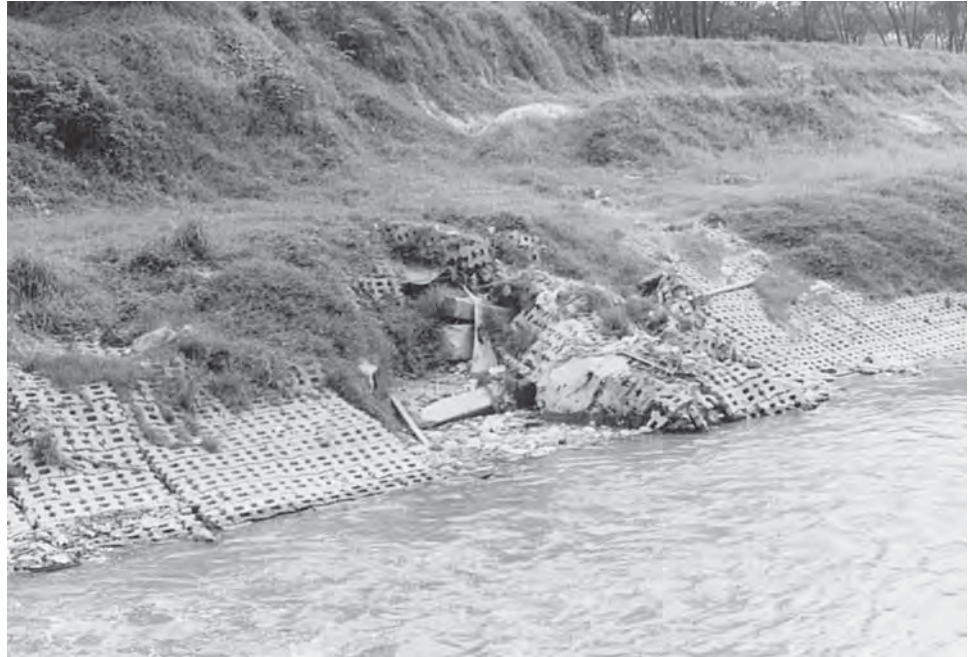


Figure E3.1. *Protruding blocks, not in conformance with placement tolerance.*



Note cast-in-place concrete joints at abutment.

Figure E3.2. *Overtaken cable-tied mats at pier.*



Note voids beneath the overturned mats.

Figure E3.3. Construction debris on subgrade.



Note ponded water at top of slope.

Figure E3.4. Incomplete backfill in termination trench.

Any areas of overturned blocks must be removed and replaced. Cable-tied block mats should not be reused; for this reason, it is important to have a source of replacement blocks, geotextile, and cables/fasteners available. Non-cabled interlocking blocks may be reused if the individual blocks are undamaged. Where localized areas are limited to cracked, broken, or missing blocks, a patch consisting of cast-in-place concrete or structural grout will usually suffice. For larger



Figure E3.5. *Variations in subgrade conditions should be brought to the attention of the project engineer.*

areas, the potential for uplift pressures to develop beneath impermeable patches will require weep holes to replace the lost permeability of the original block system.

Any voids underneath the system must be filled. Depending on the size of the void and the nature of the ACB system, voids can be filled by the following actions:

- Removing the blocks and geotextile, filling the void area with proper fill material, providing proper compaction of the filled area, and replacing the geotextile and blocks
- Removing the blocks, filling the void with sand-filled geocontainers having the same filtration capacity as the original geotextile, and then replacing the blocks
- Filling the void by pumping concrete or grout into the void via tremie pipe

3.4 Performance Evaluation

The evaluation of any countermeasure's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. To properly assess the performance of a pier scour countermeasure, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the pier scour countermeasure. Present-day channel cross-section geometry and planform should be compared to those at the time of countermeasure installation. Both lateral and vertical instability of the channel in the vicinity of the bridge can significantly alter hydraulic conditions at the piers. Approach flows may become skewed to the pier alignment, causing greater local and contraction scour.

It is recognized that the person making the performance evaluation will probably not be the inspector; however, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported as a percentage of the initial capital improvement cost.

3.4.1 Performance Rating Guide

To guide the performance evaluation for ACB systems as a pier scour countermeasure, a rating system is presented in this section. It establishes numerical ratings from 0 (worst) to 6 (best) for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the current condition of the countermeasure?

Tables E3.1 through E3.3 present a rating system for ACB pier scour countermeasures. A single numerical score is not intended; rather, an independent rating (0-6 or U) is given for each of the three topical areas. Recommended actions corresponding to the rating codes are also provided.

3.4.2 Example

To illustrate the concepts involved in performance evaluation, a 14-year-old ACB project installed in 1991 in Fort Collins, Colorado, is described and evaluated in this section.

- Project title: Abutment / bike trail scour protection
- Location: Cache la Poudre River at Mulberry Street
- Installation date: September 1991
- Design event frequency: 100 year
- Design event discharge: 13,300 ft³/s
- Side slope: 1V:4H (25%)
- Surface area of project: 4,000 ft²
- System type: Armorflex 30-s, cabled
- System weight (per unit area): 36 lb/ft²
- Installed cost (1991 dollars): \$24,000

During construction, a temporary earth cofferdam isolated the work area from the river, and portable sump pumps allowed the ACB system to be placed in the dry. Because of limited headroom beneath the bridge deck, the system was not placed using pre-assembled mats. Instead, the contractor placed the blocks by hand and threaded the cables through the blocks later. Figure E3.6 shows the ACB system during installation in September 1991. At intervals, a row of blocks was left out of the matrix to make room for a cast-in-place concrete joint. There were a number of pre-existing structural features in the project area against which the ACB system abutted; those areas also received a cast-in-place concrete seal.

Table E3.1. Rating system for articulating concrete blocks: hydraulic history.

Code	Hydraulic History	Code	Hydraulic History
U	N/A	3	Moderate: The countermeasure has experienced one or more flows greater than the 10-year event.
6	Extreme: The countermeasure has experienced one or more flows greater than the 100-year event.	2	Low: The countermeasure has experienced one or more flows greater than the 5-year event.
5	Severe: The countermeasure has experienced one or more flows greater than the 50-year event.	1	Very Low: The countermeasure has experienced one or more flows greater than the 2-year event.
4	High: The countermeasure has experienced one or more flows greater than the 25-year event.	0	Negligible: The countermeasure has not experienced any flows greater than a 2-year event.

Table E3.2. Rating system for articulating concrete blocks: maintenance history.

Code	Maintenance History	Code	Maintenance History
U	N/A	3	Moderate: The system has required occasional maintenance since installation.
6	None Required: No maintenance has been needed since installation.	2	High: Frequent maintenance has been required.
5	Very Low: The system has required maintenance for very small, local areas once or twice.	1	Very High: Significant maintenance is usually required after flood events.
4	Low: The system has required minor maintenance.	0	Excessive: The system typically requires maintenance every year.

Table E3.3. Rating system for articulating concrete blocks: current condition.

Code	Description of Current Condition	Code	Description of Current Condition
U	The ACB system is uninspectable, due to burial by sediment, debris, or other circumstance.	3	Fair: The system exhibits some missing or overturned blocks. The surface of the system exhibits irregularities or protruding blocks. Localized voids beneath system.
6	Excellent: The system is in excellent condition, with no cracked, broken, or displaced blocks. Concrete joints and seals are intact.	2	Poor: The system exhibits a very irregular surface with many missing or overturned blocks. Voids under large areas of the system.
5	Very Good: The system exhibits only minor deterioration in localized areas.	1	Badly Damaged: The system has experienced substantial deterioration in terms of broken, missing, or displaced blocks, or overturned mats.
4	Good: The system exhibits few cracked or broken blocks. Only minor protrusions noted in localized areas.	0	Severe: The system has suffered damage such that it is no longer repairable. The only recourse is to remove the entire installation and replace it with a redesigned countermeasure.

Recommended actions based on current condition rating:

Code U: The articulating concrete block system cannot be inspected. A plan of action should be developed to determine the condition of the installation. Possible remedies may include removal of debris, excavation during low flow, probing, or non-destructive testing using ground-penetrating radar or seismic methods.

Codes 6 or 5: Continue periodic inspection program at the specified interval.

Codes 4 or 3: Increase inspection frequency. The rating history of the installation should be tracked to determine if a downward trend in the rating is evident. Depending on the nature of the ACB application, the installation of monitoring instruments might be considered.

Code 2: The maintenance engineer's office should be notified and maintenance should be scheduled. The cause of the low rating should be determined, and consideration given to redesign and replacement. Materials other than ACB might be considered as a replacement.

Codes 1 or 0: The maintenance engineer's office should be notified immediately. Depending upon the nature of the ACB application, other local officials and/or law enforcement agencies identified in the Plan of Action for the bridge may also need to be notified.

The flow history is available from a U.S. Geological Survey gauging station located less than 1 mile upstream of the project site. The mean daily discharge is shown on Figure E3.7. In 1999, a flood of approximately 5,950 ft³/s passed the site. That flow was slightly greater than a 10-year event. To date, no maintenance has been necessary and the installation has remained in a stable condition with no cracked or displaced blocks at all. Figures E3.8 through E3.10 show the condition of the system in September 2005.



(a)



(b)

Figure E3.6. Mulberry Street ACB project during construction, September 1991.

**Cache la Poudre River at
USGS Station 0 6752260
Fort Collins, Colorado**

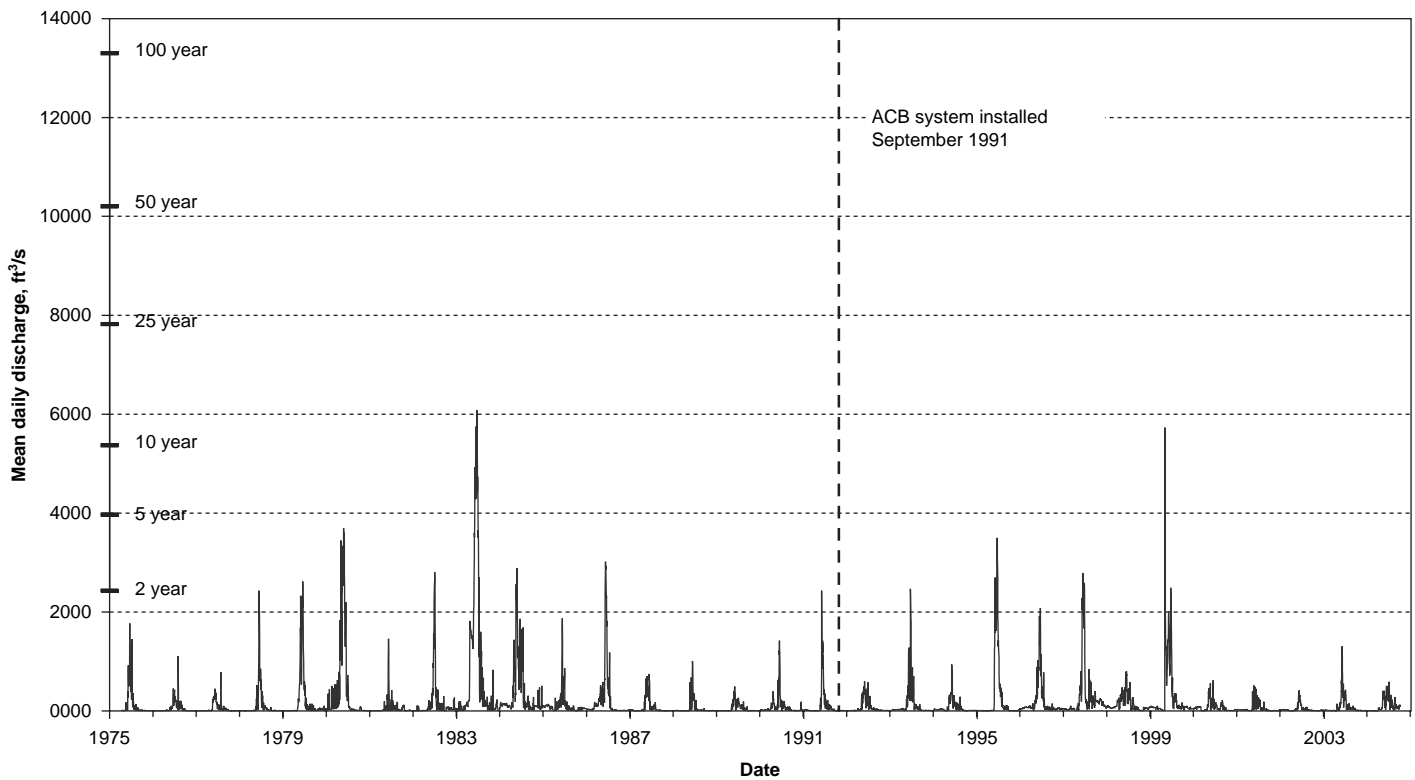


Figure E3.7. Flow history at the Mulberry Street ACB project.

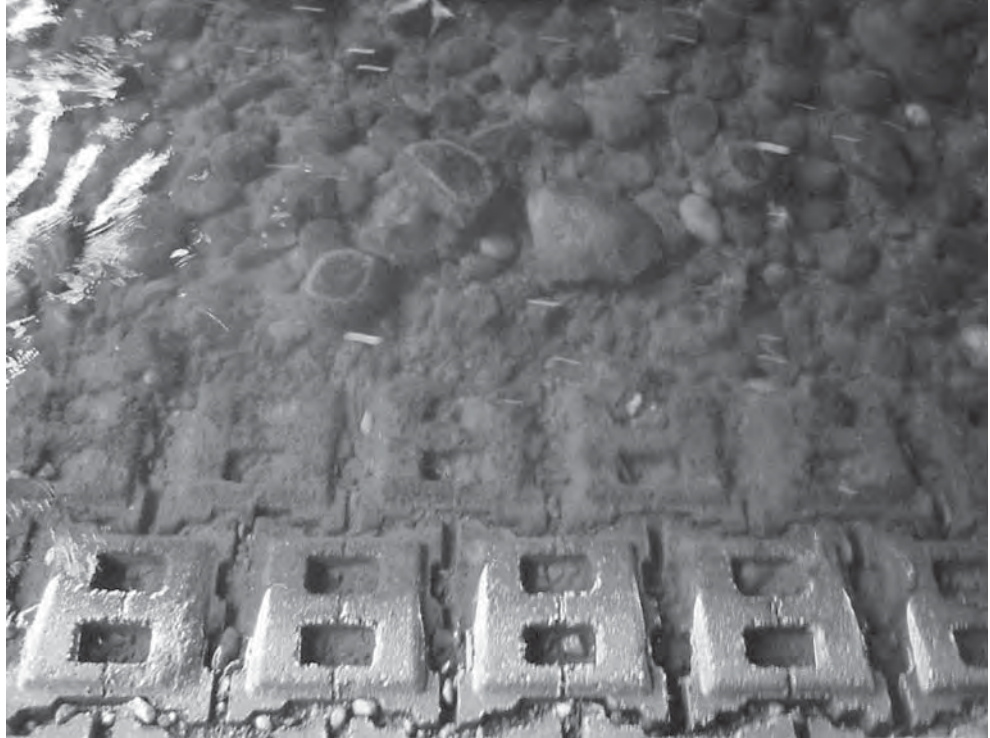


Figure E3.8. View beneath bridge deck of Mulberry Street ACB installation in September 2005.



Note sediment infill and grassy vegetation in blocks.

Figure E3.9. Mulberry Street ACB installation in September 2005 upstream of deck.



Note coarse bed material (3- to 6-in. cobbles)

Figure E3.10. Close-up view of blocks at Mulberry Street ACB installation in September 2005.

Based on the rating system presented in Section 3.4.1, the Mulberry Street ACB installation is rated as follows:

- Hydraulic history: 3 (moderate)
- Maintenance history: 6 (none required)
- Present condition: 6 (excellent)

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APPENDIX F

Guidelines for Pier Scour Countermeasures Using Gabion Mattresses

Introduction, F-2

1 Design and Specification, F-3

2 Construction, F-15

3 Inspection, Maintenance, and Performance Evaluation, F-21

References, F-25

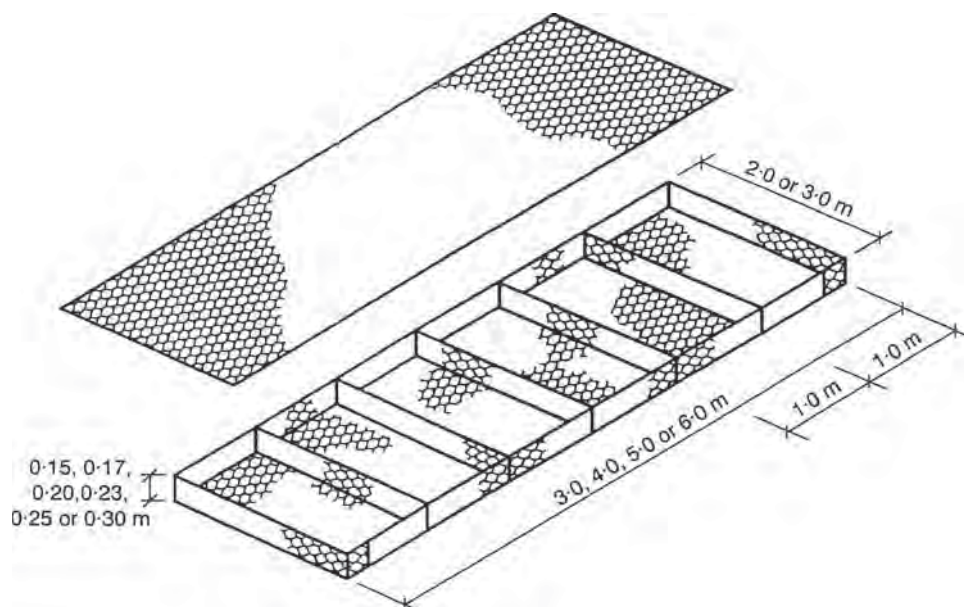
Introduction

Gabion mattresses are containers constructed of wire mesh and filled with rocks. The length of a gabion mattress is greater than the width, and the width is greater than the thickness. Diaphragms are inserted widthwise into the mattress to create compartments (Figure F1.1). Wire is typically galvanized or coated with polyvinyl chloride to resist corrosion, and either welded or twisted into a lattice. Stones used to fill the containers can be either angular rock or rounded cobbles; however, angular rock is preferred because of the higher degree of natural interlocking of the stone fill. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous structure.

The wire mesh allows the gabions to deform and adapt to changes in the bed while maintaining stability. Additionally, when compared to riprap, less excavation of the bed is required and smaller, more economical stone can be used. The obvious benefit of gabion mattresses is that the size of the individual stones used to fill the mattress can be smaller than stone that would individually be too small to withstand the hydraulic forces of a stream (Freeman and Fischenich 2000). This design guideline considers the application of gabion mattresses as a pier scour countermeasure.

There is limited field experience with the use of gabion mattresses systems as a scour countermeasure for bridge piers alone. More frequently, these systems have been used for structures such as dams or dikes, or for channel slope stabilization. The guidance for pier scour applications provided in this document has been developed primarily from the results of Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al. 2001), NCHRP Project 24-07 (Parker et al. 1998), and NCHRP Project 24-07(2) (Lagasse et al. 2007). Durability of the wire mesh under long-term exposure to the flow conditions at bridge piers has not been demonstrated; therefore, the use of gabion mattresses as a bridge pier scour countermeasure has an element of uncertainty (Parker et al. 1998).

Successful long-term performance of gabion mattresses depends on the integrity of the wire. **Because of the potential for abrasion by coarse bed load, gabion mattresses are not appropriate for gravel bed streams and should be considered for use only in sand- or fine-bed streams.** Additionally, water quality of the stream must be non-corrosive (i.e., relatively non-saline and



Source: modified from Hemphill and Bramley (1989)

Figure F1.1. Gabion mattress showing typical dimensions.

non-acidic). A polyvinyl chloride (PVC) coating should be used for applications where the potential for corrosion exists.

This document is organized into three parts:

- Part 1 provides design and specification guidelines for gabion mattress systems.
- Part 2 presents construction guidelines.
- Part 3 provides guidance for inspection, maintenance, and performance evaluation of gabion mattress systems used as a pier scour countermeasure.

Part 1: Design and Specification

1.1 Materials

1.1.1 Rock Fill

Standard test methods relating to material type, characteristics, and testing of rock and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are provided in this section and are recommended for specifying the rock fill used in gabion mattresses. In general, the test methods recommended in this section are intended to ensure that the stone is dense and durable and will not degrade significantly over time.

Rocks used for gabion mattresses should break only with difficulty, have no earthy odor, not have closely spaced discontinuities (joints or bedding planes), and not absorb water easily. Rocks composed of appreciable amounts of clay, such as shales, mudstones, and claystones, are *never* acceptable for use as fill for gabion mattresses. Table F1.1 summarizes the recommended tests and allowable values for rock and aggregate.

1.1.2 Gabion Mattresses and Components

Successful gabion performance depends not only on properly sizing and filling the baskets, but also on the quality and integrity of the wire comprising the basket compartments, diaphragms, lids, and lacing wire. Investigations conducted under NCHRP Project 24-07 (Parker et al. 1998) concluded that the lacing wire in particular proved to be the weakest link of gabion mattress systems. Wire should be single strand galvanized steel; a PVC coating may be added to protect against corrosion where required.

The wire mesh may be formed with a double twist hexagonal pattern or can be made of welded wire fabric. Fasteners, such as ring binders or spiral binders, must be of the same quality and strength as that specified for the gabion mattresses. The following recommendations are provided for twisted-wire and welded-wire gabions, respectively:

- **Twisted-wire gabion mattresses.** A producer's or supplier's certification shall be furnished to the Purchaser that the material comprising the gabion mattress components and lacing wire was manufactured, sampled, tested, and inspected in accordance with the specifications of ASTM A 975, "Standard Specification for Double-Twisted Hexagonal Mesh Gabions and Revet Mattresses (Metallic-Coated Steel Wire or Metallic-Coated Steel Wire with Poly Vinyl Chloride (PVC) Coating)." The certification must indicate that the minimum requirements of this standard have been met.
- **Welded-wire gabion mattresses.** A Producer's or Supplier's certification shall be furnished to the Purchaser that the material comprising the gabion mattress components and lacing wire was manufactured, sampled, tested, and inspected in accordance with the specifications of ASTM A 974, "Standard Specification for Welded Wire Fabric Gabions and Gabion Mattresses (Metallic-Coated or Poly Vinyl Chloride (PVC) Coated)." The certification must indicate that the minimum requirements of this standard have been met.

Table F1.1. Recommended tests for rock quality.

Test Designation	Property	Allowable value	Frequency ⁽¹⁾	Comments	
AASHTO TP 61	Percentage of Fracture	< 5%	1 per 20,000 tons	Percentage of pieces that have fewer than 50% fractured surfaces	
AASHTO T 85	Specific Gravity and Water Absorption	Average of 10 pieces: $S_g > 2.5$ Absorption < 1.0%	1 per year	If any individual piece exhibits an S_g less than 2.3 or water absorption greater than 3.0%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
AASHTO T 103	Soundness by Freezing and Thawing	Maximum of 10 pieces after 25 cycles: < 0.5%	1 per 2 years	Recommended only if water absorption is greater than 0.5% and the freeze-thaw severity index is greater than 15 per ASTM D 5312.	
AASHTO T 104	Soundness by Use of Sodium Sulfate or Magnesium Sulfate	Average of 10 pieces: < 17.5%	1 per year	If any individual piece exhibits a value greater than 25%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
AASHTO TP 58	Durability Index Using the Micro-Deval Apparatus	Value > 90 > 80 > 70	Application Severe Moderate Mild	1 per year	Severity of application per Section 5.4, CEN (2002). Most riverine applications are considered mild or moderate.
ASTM D 3967	Splitting Tensile Strength of Intact Rock Core Specimens	Average of 10 pieces: > 6 MPa	1 per year	If any individual piece exhibits a value less than 4 MPa, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.	
ASTM D 5873	Rock Hardness by Rebound Hammer	See Note (2)	1 per 20,000 tons	See Note (2)	
Shape	Length to Thickness Ratio A/C	< 10%, $d_{50} < 24$ in < 5%, $d_{60} > 24$ in	1 per 20,000 tons	Percentage of pieces that exhibit A/C ratio greater than 3.0 using the Wolman count method (Lagasse et al., 2006)	
ASTM D 5519	Particle Size Analysis of Natural and Man-Made Riprap Materials		1 per year	See Note (3)	
Gradation	Particle Size Distribution Curve		1 per 20,000 tons	Determined by the Wolman count method (Lagasse et al., 2006), where particle size, d , is based on the intermediate (B) axis.	

(1) Testing frequency for acceptance of riprap from certified quarries, unless otherwise noted. Project-specific tests exceeding quarry certification requirements, either in performance value or frequency of testing, must be specified by the Engineer.

(2) Test results from D 5873 should be calibrated to D 3967 results before specifying quarry-specific minimum allowable values.

(3) Test results from D 5519 should be calibrated to Wolman count (Lagasse et al., 2006) results before developing quarry-specific relationships between size and weight; otherwise, assume $W = 85\%$ that of a cube of dimension d having a specific gravity of S_g .

Flexibility of the gabion mattress units is a major factor in the successful performance of these systems as a pier scour countermeasure. The ability to adjust to changes in the environment around a bridge pier is desirable, particularly settling around the perimeter if scour at the edges of the system occurs. Rigid systems are more prone to undermining and subsequent damage to the mesh and are therefore less suitable for use at bridge piers. Designers are encouraged to familiarize themselves with the flexibility and performance of various materials and proprietary products for use in riverine environments.

1.2 Hydraulic Stability Design Procedure

1.2.1 Selecting a Target Factor of Safety

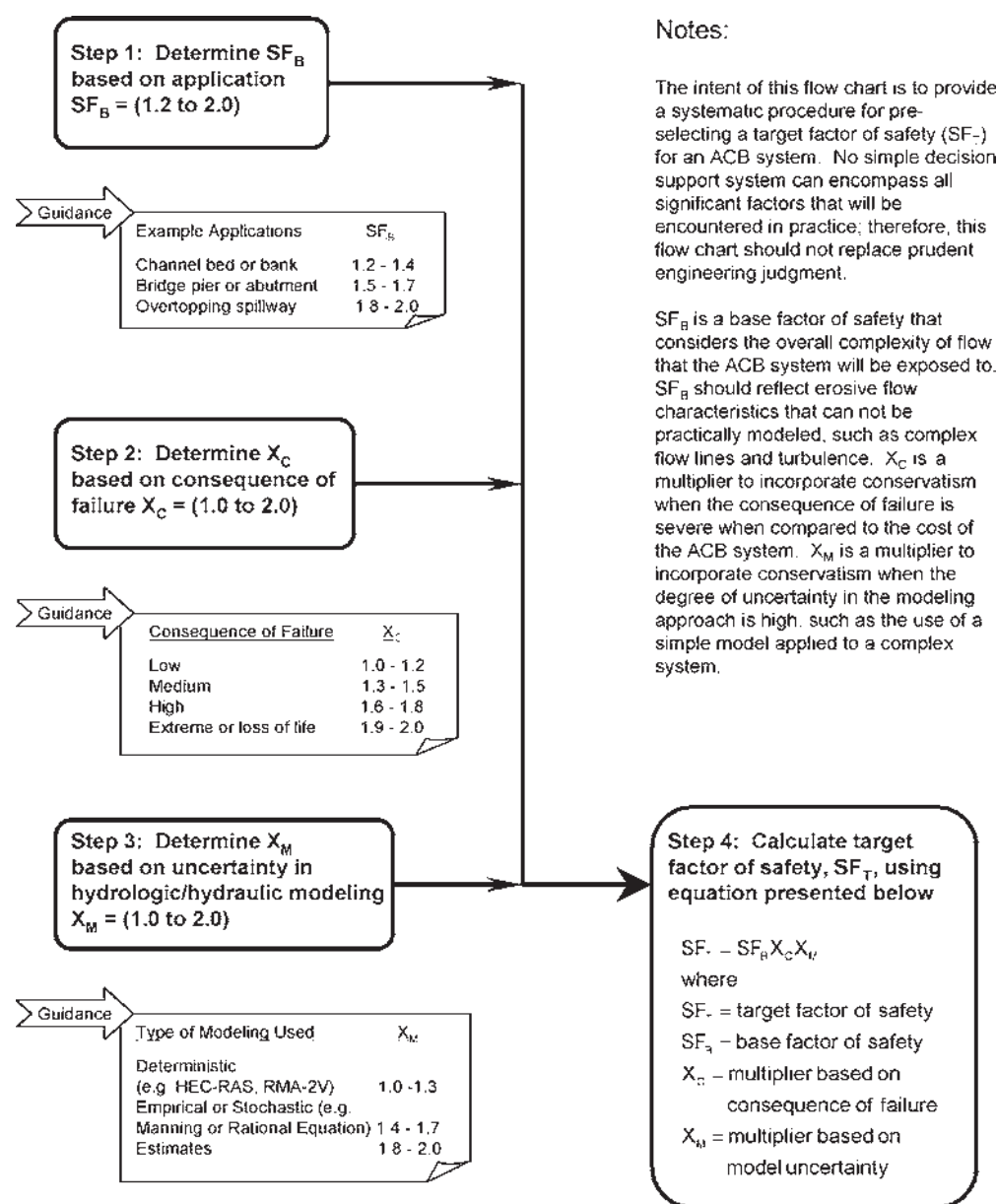
The designer must determine what factor of safety should be used for a particular application. Typically, a minimum allowable factor of safety of 1.2 is used for revetment (bank protection) when the project hydraulic conditions are well known and the installation can be conducted under well-controlled conditions. Higher factors of safety are typically used for protection at bridge piers, abutments, and channel bends because of the complexity in computing hydraulic conditions at these locations.

The Harris County Flood Control District (HCFCD), Texas, has developed a simple flowchart approach that considers the type of application, uncertainty in the hydraulic and hydrologic

models used to calculate design conditions, and consequences of failure to select an appropriate target factor of safety to use when designing an articulating concrete block (ACB) installation (Ayres Associates 2001). In this approach, the minimum allowable factor of safety for ACBs at bridge piers is 1.5. This value is then multiplied by two factors, each equal to or greater than 1.0, to account for risk and uncertainty. Figure F1.2 shows the HCFCFCD flowchart method. The method is also considered appropriate for the use of gabion mattresses at piers.

1.2.2 Design Method

Gabion mattress design methods typically yield a required d_{50} stone size that will result in stable performance under the design hydraulic loading. Because stone is produced and delivered in a range of sizes and shapes, the required size of stone is often stated in terms of a minimum



Source: Ayres Associates (2001)

Figure F1.2. Selecting a target factor of safety.

and maximum allowable size. For example, ASTM D 6711, “Standard Practice for Specifying Rock to Fill Gabions, Revet Mattresses, and Gabion Mattresses,” recommends the ranges in Table F1.2.

ASTM D 6711 also indicates that the fill should be well graded with a full range of sizes between the upper and lower limits. The rocks used to fill gabion mattresses should be hard, dense, and durable. In general, rocks used for filling gabion mattresses should be of the same material quality as would be used for riprap, as described in Section 1.1 of this appendix.

The recommended procedure for determining the permissible shear stress for a gabion mattress is determined using the relationship provided in Hydraulic Engineering Circular No. 15 (HEC-15) third edition (Kilgore and Cotton 2005):

$$\tau_p = C_s(\gamma_s - \gamma_w)d_{50} \quad (\text{F1.1})$$

where

τ_p = Permissible shear stress, lb/ft² (N/m²)

C_s = Stability coefficient for rock-filled gabion mattress equal to 0.10

γ_s = Unit weight of stone, lb/ft³ (N/m³)

γ_w = Unit weight of water, 62.4 lb/ft³ (9,810 N/m³)

d_{50} = Median diameter of rockfill in mattress, ft (m)

The coefficient C_s is an empirical coefficient developed by Maynard (1995) from test data presented in Simons et al. (1984). Use of $C_s = 0.1$ is limited to the conditions of the testing program, which used angular rock and a ratio of maximum to minimum stone size of 1.5 to 2.0.

The design conditions in the immediate vicinity of a bridge pier are more severe than the approach conditions upstream; therefore, the *local* velocity and shear stress should be used in the design equations. As recommended in HEC-23 (Lagasse et al. 2001), the section-average approach velocity, V_{avg} , must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg} \quad (\text{F1.2})$$

where

V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)

K_1 = Shape factor equal to 1.5 for round-nose piers and 1.7 for square-edged piers

K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for pier near the bank in a straight reach to 1.7 for pier located in the main current of flow around a sharp bend)

V_{avg} = Section-average approach velocity (Q/A) upstream of bridge, ft/s

If the local velocity, V_{local} , is available from stream tube or flow distribution output from a one-dimensional (1-D) model, or directly computed from a two-dimensional (2-D) model, then only the pier shape coefficient should be used to determine the design velocity. The maximum local velocity is recommended since the channel could shift and the maximum velocity could impact any pier:

$$V_{des} = K_1 V_{local} \quad (\text{F1.3})$$

Table F1.2. Size ranges for rock to fill gabion mattresses.

Mattress thickness, inches (cm)	Range of stone sizes, inches (cm)
6 (15)	3 to 5 (7.6 to 12.7)
9 (23)	3 to 5 (7.6 to 12.7)
12 (30)	4 to 8 (10 to 20)

The local shear stress at the pier, τ_{des} , is calculated using a rearranged form of Manning's equation:

$$\tau_{des} = \frac{\gamma_w}{y^{1/3}} \left(\frac{nV_{des}}{K_u} \right)^2 \quad (F1.4)$$

where

- τ_{des} = Design shear stress, lb/ft² (N/m²)
- γ_w = Unit weight of water, 62.4 lb/ft³ (9,810 N/m³)
- y = Depth of flow at pier, ft (m)
- n = Manning's n for the gabion mattress (typical range 0.025–0.035)
- K_u = 1.486 for U.S. customary units, 1 for SI units

The factor of safety can be calculated as the ratio of the permissible shear stress divided by the applied shear stress:

$$F.S. = \frac{\tau_p}{\tau_{des}} \quad (F1.5)$$

Minimum rock size should be at least 1.25 times larger than the aperture size of the wire mesh that comprises the mattress (Parker et al. 1998). Rock should be well graded between the minimum and maximum sizes to minimize the size of the voids in the matrix. If design criteria and economic criteria permit, standard gradations may be selected.

The thickness of the gabion mattress should be at least twice the average diameter of the rock fill, $T \geq d_{50}$. If the computed thickness does not match that of a standard gabion thickness, the next larger thickness of mattress should be used (Maynord 1995). At a minimum, the thickness should be 0.15 m (6 in.) (Parker et al. 1998).

1.3 Layout Dimensions

Based on small-scale laboratory studies performed for NCHRP Project 24-07(2) (Lagasse et al. 2007), the optimum performance of gabion mattresses as a pier scour countermeasure was obtained when the mattresses were extended a distance of at least 2 times the pier width in all directions around the pier.

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α , which is a function of the width, a , and length, L , of the pier (or pile bents) and the skew angle, α , as given below (after Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cos \alpha + L \sin \alpha}{a} \right)^{0.65} \quad (F1.6)$$

Gabion mattresses should be placed so that the long axis is parallel to the direction of flow (Yoon 2005). Where only clear-water scour is present, the gabion mattresses may be placed horizontally such that the top of the mattress is flush with the bed elevation; however, when other types of scour are present, the mattresses must be sloped away from the pier in all directions such that the depth of the system at its periphery is greater than the depth of contraction scour and long-term degradation, or the depth of bed-form troughs, whichever is greater (Figure F1.3). The mattresses should not be laid on a slope steeper than 1V:2H (50%). In some cases, this criterion may result in gabions being placed further than two pier widths away from the pier.

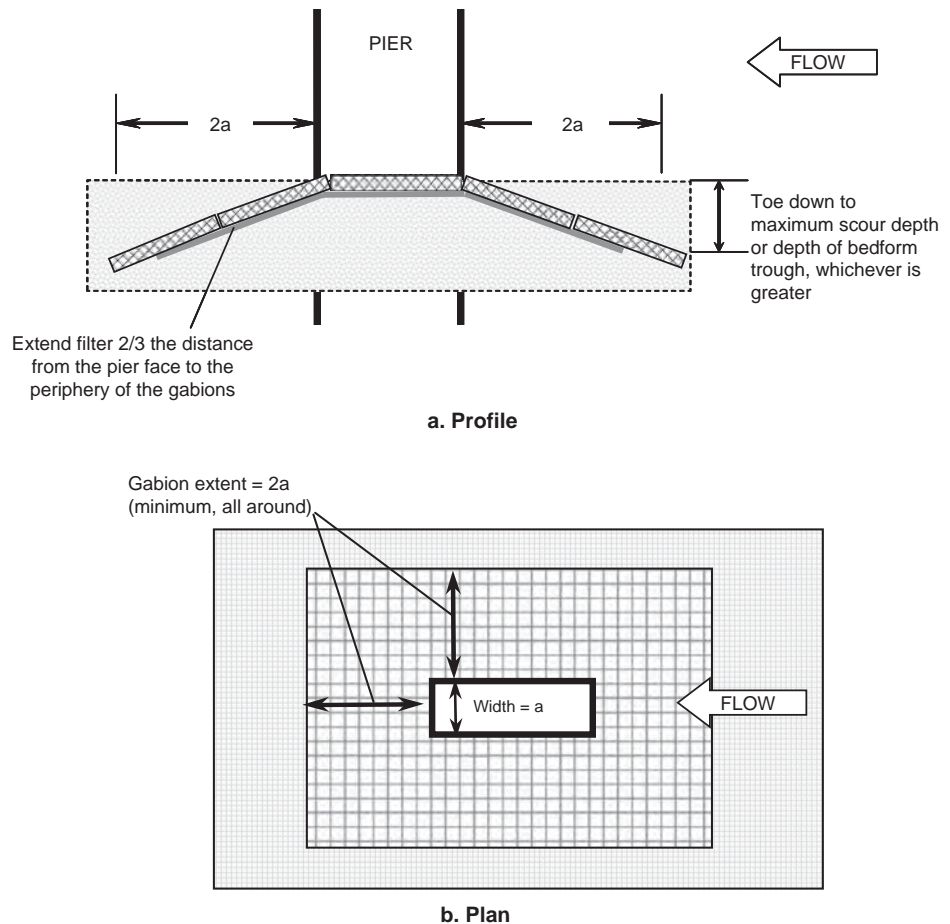


Figure F1.3. Gabion mattress layout diagram for pier scour countermeasures.

In river systems where dune-type bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of Karim (1999) and/or van Rijn (1984). In general, an upper limit on the crest-to-trough height, Δ , is provided by Bennet (1997) as $\Delta < 0.4y$ where y is the depth of flow. This limit suggests that the maximum depth of the bed-form trough below ambient bed elevation will not exceed 0.2 times the depth of flow.

A filter is typically required for gabion mattresses at bridge piers. The filter should not be extended fully beneath the gabions; instead, it should be terminated two-thirds of the distance from the pier to the edge of the gabion mattress. When using a granular stone filter, the layer should have a minimum thickness of 4 times the d_{50} of the filter stone or 6 in. (15 cm), whichever is greater. The granular filter layer thickness should be increased by 50% when placing under water.

1.4 Filter Requirements

The importance of the filter component of a gabion installation should not be underestimated. Two kinds of filters are used in conjunction with gabion mattresses: granular filters and geotextile filters. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of, the filter layer. ***In cases where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.***

The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain *all* the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind.

1.4.1 Geotextile Filter Properties

Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.***

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand stresses during installation. These values are available from manufacturers. The following list briefly describes the most relevant properties:

- **Permeability.** The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimeters per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows perpendicularly through the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of (s)⁻¹. Permeability (and permittivity) is extremely important in gabion mattress filter design. For gabion mattress installations, the permeability of the geotextile should be at least 10 times greater than that of the underlying material:

$$K_g > 10K_s \quad (\text{F1.7})$$

where

K_g = Permeability of geotextile (cm/s)

K_s = Permeability of subgrade soil (cm/s)

- **Transmissivity.** The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second. This property is directly related to the drainage function and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas non-woven, needle-punched fabrics have a much greater capacity due to their three-dimensional (3-D) microstructure. Transmissivity is not particularly relevant to gabion mattress filter design.
- **Apparent opening size (AOS).** Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.
- **Porosity.** Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Percent open area (POA).** POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Thickness.** As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).

- **Grab strength and elongation.** Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.
- **Tear strength.** Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.
- **Puncture strength.** Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

Table F1.3 provides the recommended characteristics for geotextile filters. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving gabion mattress installation at piers have been discussed here. Geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by gabion mattresses, these stresses do not represent the environment that the geotextile will experience in the long term.

Table F1.3. Recommended requirements for geotextile properties.

Test Designation	Property	Allowable value ⁽¹⁾		Comments
		Elongation < 50% ⁽²⁾	Elongation > 50% ⁽²⁾	
ASTM D 4632	Grab Strength	> 315 lbs (Class 1) > 250 lbs (Class 2) > 180 lbs (Class 3)	> 200 lbs (Class 1) > 160 lbs (Class 2) > 110 lbs (Class 3)	From AASHTO M 288
ASTM D 4632	Sewn Seam Strength ⁽³⁾	> 270 lbs (Class 1) > 220 lbs (Class 2) > 160 lbs (Class 3)	> 180 lbs (Class 1) > 140 lbs (Class 2) > 100 lbs (Class 3)	From AASHTO M 288
ASTM D 4533	Tear Strength ⁽⁴⁾	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4833	Puncture Strength	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4751	Apparent Opening Size	Per design criteria (Section 1.4 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 1.4 of this design guide)		Minimum allowable value
ASTM D 4355	Degradation by Ultraviolet Light	> 50% strength retained after 500 hours of exposure		Minimum allowable value
ASTM D 4873	Guide for Identification, Storage, and Handling			Provides information on identification, storage, and handling of geotextiles.
ASTM D 4759	Practice for the Specification Conformance of Geosynthetics			Provides information on procedures for ensuring that geotextiles at the jobsite meet the design specifications.

(1) Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class. The following descriptions have been modified from AASHTO M 288:

- *Class 1* is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including when placement of riprap must occur in multiple lifts, when drop heights may exceed 1 ft (0.3 m) or when repeated vehicular traffic on the installation is anticipated.
- *Class 2* is recommended for installation conditions where placement in regular, single lifts are expected and little or no vehicular traffic on the installation will occur, or when placing individual rocks by clamshell, orange-peel grapple or specially equipped hydraulic excavator with drop heights less than 1 ft.
- *Class 3* is specified for the least severe installation environments, with drop heights less than 1 ft onto a bedding layer of select sand, gravel or other select imported material.

(2) As measured in accordance with ASTM D 4632.

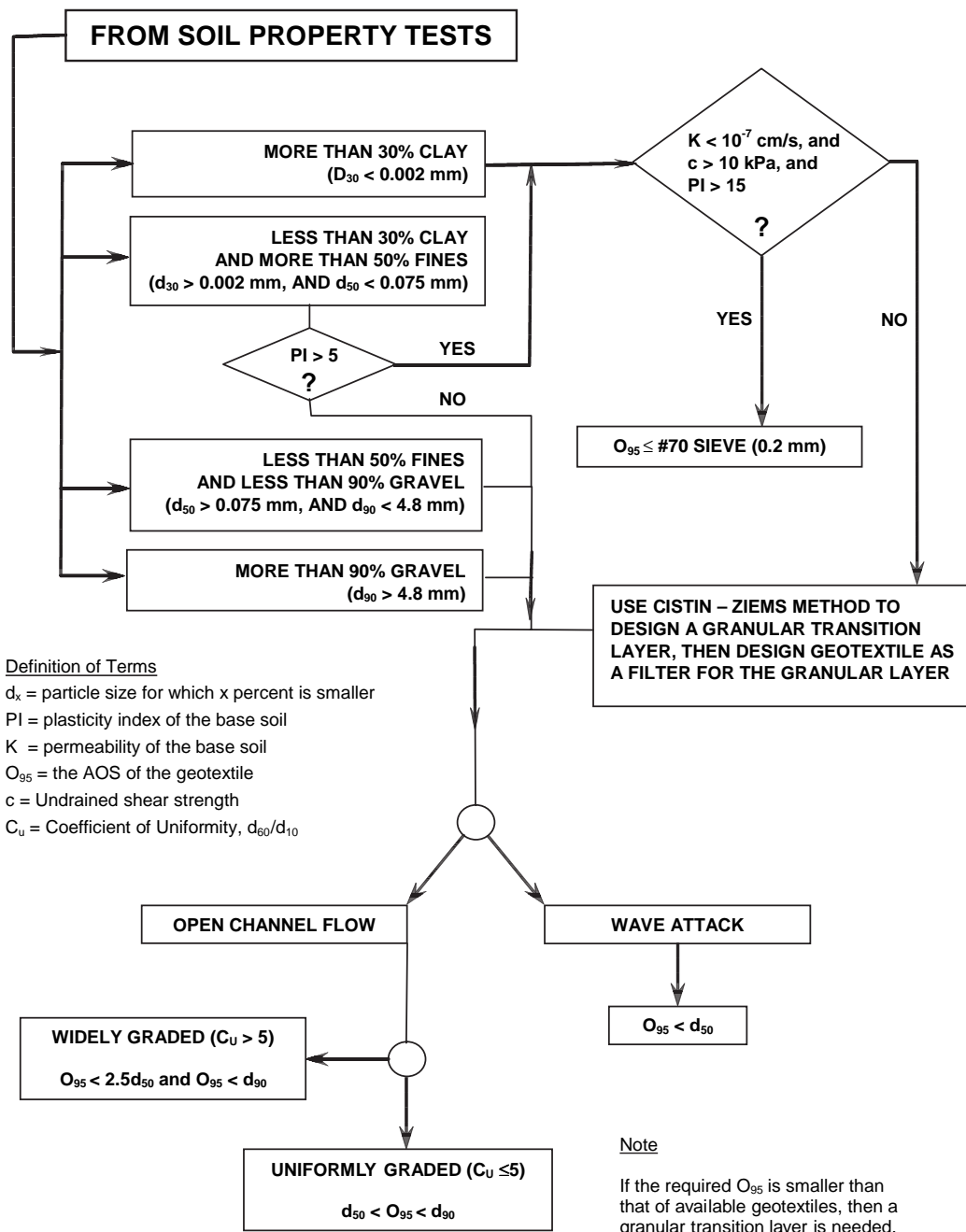
(3) When seams are required.

(4) The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 55 lbs. The MARV corresponds to a statistical measure whereby 2.5% of the tested values are less than the mean value minus two standard deviations (Koerner 1998).

1.4.2 Geotextile Filter Design Procedure

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. A decision tree is provided as Figure F1.4 to assist in determining the appropriate soil retention criterion for the geotextile. The figure includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater than 30% clay or is predominantly fine-grained soil (more than 50% passing the #200 sieve).



Source: modified from Koerner (1998)

Figure F1.4. Geotextile selection based on soil retention.

If a granular transition layer is required, the geotextile should be designed to be compatible with the properties of the granular layer. If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required. However, this requirement can be waived if the base soil exhibits the following conditions for hydraulic conductivity, K ; plasticity index, PI ; and undrained shear strength, c :

$$K < 1 \times 10^{-7} \text{ cm/s}$$

$$PI > 15$$

$$c > 10 \text{ kPa}$$

Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS less than a #70 sieve (approximately 0.2 mm) can be used with soils meeting these conditions and essentially functions more as a separation layer than a filter.

Step 3. Determine Permeability Criterion. The permeability criterion requires that the filter exhibit a permeability at least 4 times greater than that of the base soil (Koerner 1998) and for critical or severe applications, at least 10 times greater (Holtz et al. 1995). Generally speaking, if the permeability of the base soil or granular filter has been determined from laboratory testing, that value should be used. If laboratory testing was not conducted, then an estimate of permeability based on the particle size distribution should be used.

To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s^{-1} . Typically, the designer will need to contact the geotextile manufacturer to obtain values of permeability, permittivity, and thickness.

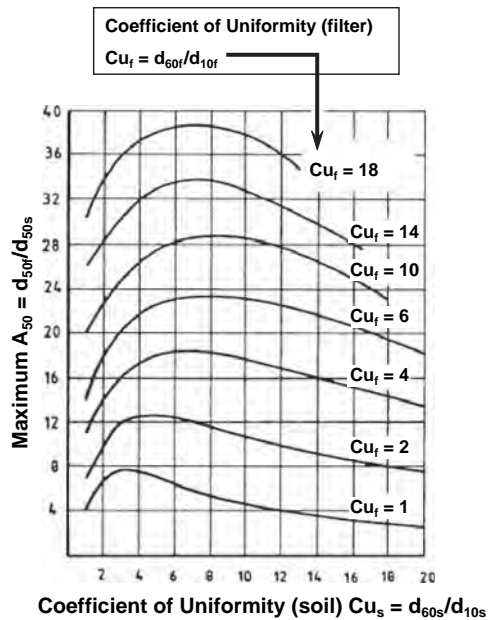
Step 4. Select a Geotextile that Meets the Required Strength Criteria. Strength and durability requirements depend on the installation environment and the construction equipment that is being used. See Table F1.3 for recommended values based on AASHTO M 288, “Geotextile Specification for Highway Construction,” which provides guidance on allowable strength and elongation values for three categories of installation severity. For additional guidelines regarding the selection of durability test methods, refer to ASTM D 5819, “Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability.”

Step 5. Minimize Long-Term Clogging Potential. When a woven geotextile is used, its POA should be greater than 4% by area. If a non-woven geotextile is used, its porosity should be greater than 30% by volume. A good rule of thumb suggests that the geotextile having the largest AOS that satisfies the particle retention criteria should be used (provided of course that all other minimum allowable values described in this section are met as well).

1.4.3 Granular Filter Properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters can be used alone or can serve as a transitional layer between a predominantly fine-grained base soil and a geotextile. The following list briefly describes the most relevant properties:

- **Particle Size Distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziemis whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure F1.5 provides a design chart based on the Cistin–Ziemis approach.



Source: Heibaum (2004)

Figure F1.5. Granular filter design chart according to Cistin and Ziems.

- **Permeability.** Permeability of a granular filter material is determined by laboratory test or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For gabion mattress installations, the permeability of the granular filter should be at least 10 times greater than that of the underlying material.
- **Porosity.** Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.
- **Thickness.** Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. is specified. For placement under water, thickness should be increased by 50%.
- **Quality and durability.** Aggregate used for a granular filter should be hard, dense, and durable.

1.4.4 Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter. That approach was developed for subsoils consisting of well-graded sands and may not be widely applicable to other soil types. An alternative approach that is considered more robust in this regard is the Cistin–Ziems method.

The suggested steps for proper design of a granular filter using this method are outlined below. Note that the subscript “s” is used to represent the base (finer) soil, and “f” is used to represent the filter (coarser) layer.

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Key Indices for Base Soil. From the grain size information, determine the median grain size, d_{50} , and the coefficient of uniformity, $Cu_s = d_{60}/d_{10}$, of the base soil.

Step 3. Determine Key Indices for Granular Filter. One or more locally available aggregates should be identified as potential candidates for use as a filter material. The d_{50} and coefficient of uniformity, $Cu_f = d_{60}/d_{10}$, should be determined for each candidate filter material.

Step 4. Determine Maximum Allowable d_{50} for Filter. Enter the Cistin–Ziems design chart (Figure F1.5) with the coefficient of uniformity, Cu_s , for the base soil on the x-axis. Find the curve that corresponds to the coefficient of uniformity, Cu_f , for the filter in the body of the chart and, from that point, determine the maximum allowable A_{50} from the y-axis. Compute the maximum allowable d_{50f} of the filter using d_{50fmax} equals A_{50max} times d_{50s} . Check to see if the candidate filter material conforms to this requirement. If it does not, continue checking alternative candidates until a suitable material is identified.

Step 5. Check for Permeability. From laboratory permeameter tests or the grain size distribution of the candidate filter material, determine whether the hydraulic conductivity of the filter is at least 10 times greater than that of the subsoil.

Step 6. Check for Compatibility with Gabion Mattress Rock. Repeat steps 1 through 4 above, considering that the filter material is now the “finer” soil and the rock is the “coarser” material. If the Cistin–Ziems criterion is not met, then multiple layers of granular filter materials should be considered.

Step 7. Filter Layer Thickness. For practicality of placement, the nominal thickness of a single filter layer should not be less than 6 in. (15 cm). Single-layer thicknesses up to 15 in. (38 cm) may be warranted where large rock fill particle sizes are used. When multiple filter layers are required, each individual layer should range from 4 to 8 in. (10 to 20 cm) in thickness (Brown and Clyde 1989).

NOTE: In cases where dune-type bed forms may be present or underwater installation, it is strongly recommended that only a geotextile filter be considered.

1.5 Guidelines for Seal Around the Pier

An observed key point of failure for gabion mattress systems at bridge piers during laboratory studies occurs at the joint where the mattress meets the bridge pier. During NCHRP Project 24-07(2), securing the geotextile to the pier prevented the leaching of the bed material from around the pier. This procedure worked successfully in the laboratory, but there are constructability implications that must be considered when this technique is used in the field, particularly when the mattress is placed under water.

A grout seal between the mattress and the pier is recommended. A grout seal is not intended to provide a structural attachment between the mattress and the pier, but instead is a simple method for plugging gaps to prevent bed sediments from winnowing out from beneath the system. In fact, structural attachment of the mattress to the pier is strongly discouraged. The transfer of moments from the mattress to the pier may affect the structural stability of the pier, and the potential for increased loadings on the pier must be considered. When a grout seal is placed under water, an anti-washout additive is required.

1.6 Anchors

Anchors are not typically used with gabion mattress systems; however, the layout guidance presented in Section 1.3 indicates that the system should be toed down to a termination depth at least as deep as any expected contraction scour and long-term degradation, or bed-form troughs, whichever is greater. Where such toedown depth cannot be achieved, for example where bedrock is encountered at shallow depth, a gabion mattress system with anchors along the front

(upstream) and sides of the installation is recommended. The spacing of the anchors should be determined based on a factor of safety of at least 5.0 for pullout resistance based on calculated drag on the exposed leading edge. Spacing between anchors of no more than 4 ft (1.3 m) is recommended. The following example is provided:

Given

$$\begin{aligned}\rho &= \text{Mass density of water, slugs/ft}^3 &&= 1.94 \\ V &= \text{Approach velocity, ft/s} &&= 10 \\ \Delta z &= \text{Height of gabion mattress, ft} &&= 0.5 \\ b &= \text{Width of mattress installation (perpendicular to flow), ft} &&= 40\end{aligned}$$

Step 1: Calculate total drag force, F_d , on leading edge of system:

$$F_d = 0.5\rho V^2(\Delta z)(b) = 0.5(1.94)(10^2)(0.5)(40) = \mathbf{1,940 \text{ lb}}$$

Step 2: Calculate required uplift restraint using 5.0 safety factor:

$$F_{\text{restraint}} = 5.0(1,940) = \mathbf{9,700 \text{ lb}}$$

Step 3: Counting anchors at the corners of the system, calculate required pullout resistance per anchor (rounded to nearest 10 lb):

a) Assume 11 anchors at 4-ft spacing: 9,700 lb/11 anchors = **880 lb/anchor**

b) Assume 21 anchors at 2-ft spacing: 9,700 lb/21 anchors = **460 lb/anchor**

Anchors should never be used as a means to avoid toeing the system down to the full required extent where alluvial materials are present at depth. In this case, scour or bed-form troughs will simply undermine the anchors as well as the system in general.

Part 2: Construction

The guidance in this section has been developed to facilitate the proper installation of gabion mattress systems to achieve suitable hydraulic performance and maintain stability against hydraulic loading. The proper installation of gabion mattress systems is essential to the adequate functioning and performance of the system during the design hydrologic event. Gabion mattress installation can be labor intensive, requiring manual attachment of the lacing wire and/or connectors on mattress components and also for mattress-to-mattress connection. Guidelines are provided herein for maximizing the correspondence between the design intent and the actual field-finished conditions of the project.

This section addresses the preparation of the subgrade, geotextile placement, gabion mattress placement, backfilling and finishing, and measurement and payment.

2.1 General Guidelines

The contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. This responsibility is typically performed through the owner's engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see ER 1180-1-6 [U.S. Army Corps of Engineers 1995]) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for gabion mattress placement are included in the project plans and specifications. Standard gabion mattress specifications and layout guidance are found in Part 1 of this appendix. Recommended requirements for the stone and wire mesh, including the tests

necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications, are provided. Field tests can be performed at the quarry and/or on the job site, or representative samples can be obtained for laboratory testing to ensure that the rock fill is of suitable quality.

Inspection of gabion mattress placement consists of visual inspection of the operation and the finished surface. Inspection and quality assurance must be carefully organized and conducted in case potential problems or questions arise over acceptance of stone or wire mesh material. Acceptance should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject stone at the quarry, at the job site or stockpile, and in place in the structures throughout the duration of the contract. Stone rejected at the job site should be removed from the project site. Stone rejected at the quarry should be disposed of or otherwise prevented from mixing with satisfactory stone.

Construction techniques can vary tremendously because of the following factors:

- Size and scope of the overall project
- Size and weight of the stone fill particles
- Placement under water or in the dry
- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions
- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general information regarding construction of gabion mattress installations and some basic information and description of techniques and processes involved.

2.2 Materials

2.2.1 Mattresses

Materials composing the gabion mattress system shall be in accordance with the guidance provided in Part 1 of this appendix. Mattresses shall be sound and free of defects that would interfere with proper placement or that would impair the integrity of the system. Wire mesh should be inspected upon arrival to ensure no structural damage occurred during transport.

2.2.2 Wire

Wire shall not be kinked or broken. Kinks may be stretched or stamped out if integrity of the gabion is not compromised in doing so. PVC coated wire shall not show sign of cracks, splits, or color changes.

2.2.3 Filter

Geotextile. Either woven or non-woven, needle-punched geotextiles may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.*** Each roll of geotextile shall be labeled with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. Geotextiles shall not be exposed to sunlight prior to placement.

Granular filters. Samples of granular filter material shall be tested for grain size distribution to ensure compliance with the gradation specification used in design. Sampling and testing frequency shall be in accordance with the owner or owner's authorized representative.

2.2.4 Subgrade Soils

When placement is in the dry, the gabion mattress system shall be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Unsatisfactory soils shall be considered those soils having excessive in-place moisture content; soils containing roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils shall be removed, and the excavation backfilled with approved material that is compacted prior to placement of the mattress system. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays, per the geotechnical engineer's recommendations.

When gabion mattresses are placed under water, compaction of the subgrade is impractical. However, the surface must be relatively smooth, with no abrupt irregularities that would prevent intimate contact between the system and the subgrade. Under no circumstances may gabion mattresses be draped over boulders, bridged over subgrade voids, or placed over other irregularities that would prevent achievement of intimate contact between the system and the subgrade. Placing a layer of bedding stone may assist in achieving a suitable surface on which to place the gabion mattress system.

2.3 Installation

2.3.1 Subgrade Preparation

Stable and compacted subgrade soil shall be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes shall be compacted, shaped, and uniformly graded to facilitate the development of intimate contact between the gabion mattress system and the underlying grade. Termination between the gabion mattress system and a concrete slab, footer, pier, wall, or similar structure shall be sealed in a manner that prevents soil migration.

The subgrade soil conditions shall meet or exceed the required material properties described in Section 2.2.4 prior to placement of the gabion mattresses. Soils not meeting the requirements shall be removed and replaced with acceptable material.

When placement is in the dry, the areas to receive the gabion mattress system shall be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the geotextile, and between the geotextile and the bottom surface of the gabion mattress system. The subgrade should be uniformly compacted to the geotechnical engineer's site-specific requirements. If the subgrade surface for any reason becomes rough, corrugated, uneven, textured, or traffic marked prior to gabion mattress installation, such unsatisfactory portion shall be scarified, reworked, recompacted, or replaced as directed by the engineer.

When placement is under water, divers shall be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create irregularities in the mattress placement or that would create voids beneath the system in accordance with Section 2.2.4. Immediately prior to placement of the filter and gabion mattress system, the prepared subgrade must be inspected.

2.3.2 Placing the Filter

Whether the filter comprises one or more layers of granular material or is made of geotextile, its placement should result in a continuous installation that maintains intimate contact with the soil beneath. Voids, gaps, tears, or other holes in the filter must be avoided to the extent practicable, and the filter must be replaced or repaired when they occur.

Placement of Geotextile. The geotextile shall be placed directly on the prepared area, in intimate contact with the subgrade. When a geotextile is placed, it should be rolled or spread out

directly on the prepared area and be free of folds or wrinkles. The rolls shall not be dragged, lifted by one end, or dropped. The geotextile should be placed in such a manner that placement of the overlying materials will not excessively stretch or tear the geotextile.

After geotextile placement, the work area shall not be trafficked or disturbed in a manner that might result in a loss of intimate contact between the gabion mattress, the geotextile, and the subgrade. The geotextile shall not be left exposed longer than the manufacturer's recommendation to minimize potential damage due to ultraviolet radiation; therefore, the overlying materials should be placed as soon as practicable.

The geotextile shall be placed so that upstream strips overlap downstream strips. Overlaps shall be in the direction of flow wherever possible. The longitudinal and transverse joints shall be overlapped at least 1.5 ft (46 cm) for dry installations and at least 3 ft (91 cm) for below-water installations. If a sewn seam is to be used for the seaming of the geotextile, the thread to be used shall consist of high-strength polypropylene or polyester and shall be resistant to ultraviolet radiation. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or temporary weights such as sandbags shall be used.

Placing Geotextiles Under Water. Placing geotextiles under water can be problematic for a number of reasons. Most geotextiles that are used as filters beneath gabion mattresses are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, meaning that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the countermeasure (Koerner 1998).

Flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric (a condition that contractors refer to as "galloping") that are extremely difficult to control. The preferred method of controlling geotextile placement is to isolate the work area from river currents by a temporary cofferdam. In mild currents, geotextiles pre-cut to length can be placed by divers, with sandbags to hold the fabric temporarily.

In the past, geotextiles have been affixed to the base of each mattress or placed on the bottom of the mattress compartments prior to their being filled with stone; however, this method will not result in a good overlap of fabric between individual mattresses and is not recommended.

At bridge piers, sand-filled geocontainers made of non-woven, needle-punched fabric are particularly effective for placement under water as shown in Figure F2.1. The geotextile fabric

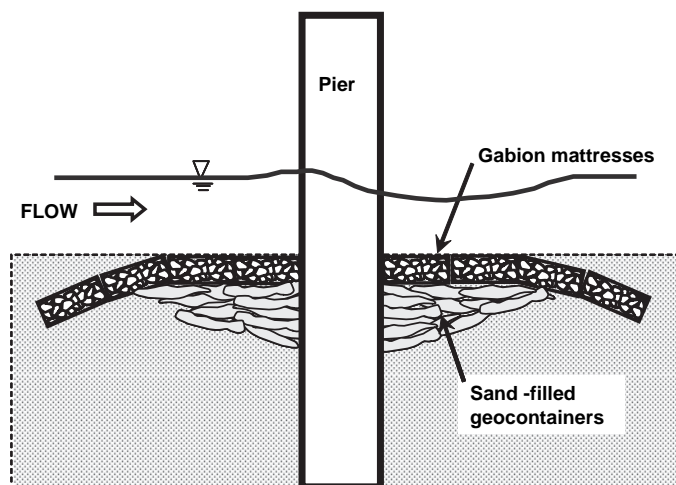


Figure F2.1. Schematic diagram showing the use of sand-filled geocontainers as a filter.

and sand fill that compose the geocontainers should be selected in accordance with the filter design criteria presented in Part 1 and placed such that they overlap to cover the required area. Geocontainers can be fabricated in a variety of dimensions and weights. Each geocontainer should be filled with sand to no more than 80% its total volume so that it remains flexible and “floppy.” The geocontainers also can serve to fill a pre-existing scour hole around a pier prior to placement of the gabion mattresses, as shown in Figure F2.1. For more detail, see Lagasse et al. (2006, 2007).

Placement of Granular Filter. For placing a granular filter, front-end loaders are the preferred method for dumping and spreading the material on slopes milder than approximately 4H:1V. A typical minimum thickness for granular filters is 0.5 to 1.0 ft (0.15 to 0.3 m). Placing granular media under water around a bridge pier is best accomplished using a large-diameter tremie pipe to control the placement location and thickness, while minimizing the potential for segregation. When a granular filter is placed under water, the thickness should be increased by 50%. **NOTE: For riverine applications where dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered.**

2.3.3 Placing the Gabion Mattress System

Manufacturer’s assembly instructions should be followed. Mattresses shall be placed on the filter layer and assembled so that the wire does not kink or bend. Mattresses shall be placed so that the longitudinal axis is parallel to the flow and internal diaphragms are perpendicular to the flow. Prior to filling, adjacent mattresses should be connected along the vertical edges and the top selvedges by lacing, fasteners, or spiral binding. Custom fitting of mattresses around corners or curves should be done according to manufacturer’s recommendations.

Care shall be taken during installation so as to avoid damage to the geotextile or subgrade during the installation process. Mattresses should not be pushed or pulled laterally once they are on the geotextile. Preferably, the mattress placement and filling should begin at the upstream section and proceed downstream. If a mattress system is to be installed starting downstream and proceeding in the upstream direction, a contractor option is to construct a temporary toe trench at the front edge of the mattress system to protect against flow that could otherwise undermine the system during flow events that may occur during construction. On sloped sections where practical, placement and filling shall begin at the toe of the slope and proceed upslope.

Gabion Mattress Placement Under Water. Gabion mattresses placed in water require close observation and increased quality control to ensure a continuous countermeasure system. A systematic process for placing and continuous monitoring to verify the quantity and layer thickness is important.

Excavation, grading, and placement of gabion mattresses and filter under water require additional measures. For installations of a relatively small scale, the stream around the work area may be diverted during the low-flow season. For installations on larger rivers or in deeper water, the area can be temporarily enclosed by a cofferdam, which allows for construction dewatering if necessary. Alternatively, a silt curtain made of plastic sheeting may be suspended by buoys around the work area to minimize environmental degradation during construction. Gabions can be assembled, filled, and closed in the dry, and a crane and spreader bar can be used to lift and place the system under water. Once under water and in the correct positions, the individual mattresses can be laced together or otherwise connected by divers.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remote operated vehicles (ROVs) can provide some information about the gabion mattress placement under water.

2.3.4 Filling of Mattresses

Gabions should be filled carefully, either by machine or by hand placement. Machine placement will typically require some hand positioning to minimize voids and avoid bulging. Compartments shall be filled simultaneously so that the depth of rock in one compartment is never significantly greater than the depth in adjacent compartments. Gabions can be overfilled slightly to account for settling, but overfilling should not cause the lid to bulge or become separated from the sides of the basket.

Any wire damaged during assembly, filling, and/or placement should be promptly repaired or replaced according to manufacturer's instructions. Excessive damage may require replacement of the gabion mattress.

2.3.5 Closing the Gabion Mattresses

Lids shall be tightly stretched over the rock fill and secured using manufacturer-recommended tools. The lids shall then be secured to the gabion mattresses along all top selvages. The wire fabric shall be drawn tightly against the rock on all sides and tied with galvanized wire, locking clips, hog rings, or connectors. When ties, locking clips, hog rings, or connectors are used for tying mesh sections and selvages together, they shall be spaced 3 in. or less apart as specified in the plans. Galvanized wire ties shall be spaced approximately 2 ft on center and shall be anchored to the bottom layer of wire fabric (Lagasse et al. 2001).

2.4 Finishing

2.4.1 System Termination

Termination of the gabion mattress system shall be either (1) in excavated trenches that are properly backfilled with approved material flush with the top of the finished surface of the gabions or (2) abutted to a structural feature such as a pier, footing, or pile cap. In the case of gabions abutting a structural feature, any gaps between the mattress and the structure shall be filled with cast-in-place concrete or grout and finished flush with the top surface of the gabion mattress.

2.4.2 Backfilling the Gabion Mattress System

The gabion mattress system can be either backfilled with suitable soil for revegetation, or with 3/8- to 3/4-in. (10- to 20-mm) crushed rock aggregate. Backfilling with soil or granular fill shall be completed as soon as practicable after the revetment has been installed. When topsoil is used as a fill material above the normal waterline, the gabion system should be overfilled by 1 to 2 in. (25 to 50 mm) to account for backfill material consolidation.

2.4.3 Inspection

The subgrade preparation, geotextile placement, gabion mattress system installation, and overall finished condition including termination trenches shall be inspected before work acceptance. Inspection guidelines for the gabion installation are presented in detail in Part 3 of this appendix.

2.5 Measurement and Payment

Measurement of the gabion system for payment shall be made on the basis of surface area. The pay lines will be neat lines taken off the contract drawings and will include embedded mattresses and/or mattresses placed in termination trenches.

The finished surface of the gabion mattresses should be surveyed to ensure that the as-built lines and grades meet the design plans within the specified tolerance. Survey cross sections perpendicular to the axis of the structure are usually taken at specified intervals. Payment will be full compensation for all material, labor, and equipment to complete the work.

Part 3: Inspection, Maintenance, and Performance Evaluation

3.1 Inspection During Construction

Inspection during construction shall be conducted by qualified personnel who are independent of the contractor. Underwater inspection of a gabion mattress system shall be only performed by divers specifically trained and certified for such work.

3.1.1 Subgrade

Inspection of the subgrade shall be performed immediately prior to geotextile placement. The subgrade should be clean and free of projections, debris, construction materials, or other foreign objects that would prevent the filter from being properly placed. Likewise, there should be no potholes, rills, or other voids that the filter material might bridge over.

The subgrade material itself should not be muddy or frozen and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area shall be noted and photographed; observations of such should be brought to the attention of the project engineer as they may represent conditions that are different from those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

3.1.2 Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product designation. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics or slit-film geotextiles should **never** be used in gabion mattress applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, anchor pins, or U-shaped soil staples may be used to hold the geotextile in position while the gabion mattresses are being placed. The gabions should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

3.1.3 Gabion Mattresses

Inspection of gabion mattress placement typically consists of visual inspection of the operation and the finished surface. Inspection must ensure that the mattresses are sound and have been connected on all vertical surfaces and top selvages to form a continuous unit, that the rock fill is compact and voids are minimal within each compartment, and that the wire has not been broken or kinked.

3.2 Periodic and Post-Flood Inspection

Bridge pier gabion mattress systems are typically inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a specified magnitude.

Underwater inspection of a gabion mattress system shall be performed only by divers specifically trained and certified for such work. Whether visually or by feel, the diver should pay particular attention to the areas where the gabion abuts structural elements to ensure that no gaps exist and that subgrade material is not being removed from beneath the gabion mattress

system. The diver should also ensure that individual mattresses abut one another such that there are no gaps between mattresses, that mattresses have not been placed on top of one another, and that the mattresses have been joined together by lacing wire or other approved connectors.

An important aspect of inspecting a gabion mattress installation is to determine if the system has been repositioned by the flow to the extent that the subgrade or filter has become exposed. The mattresses and filling stone should be examined for evidence of downstream migration. If filling stone is observed to migrate downstream within each compartment, the height difference between the highest and lowest rock surface should be measured as shown in Figure F3.1. Some movement of the stone fill within mattress compartments after exposure to high shear stresses is acceptable.

The following relationship should be maintained to ensure that underlying subgrade remains protected and unexposed (Maccafferri, undated):

$$(\Delta Z) \leq 2d_{50} \left(\frac{t}{d_{50}} - 1 \right) \quad (\text{F3.1})$$

where

ΔZ = Height difference between the highest and lowest rock surface within a mattress, ft (m)

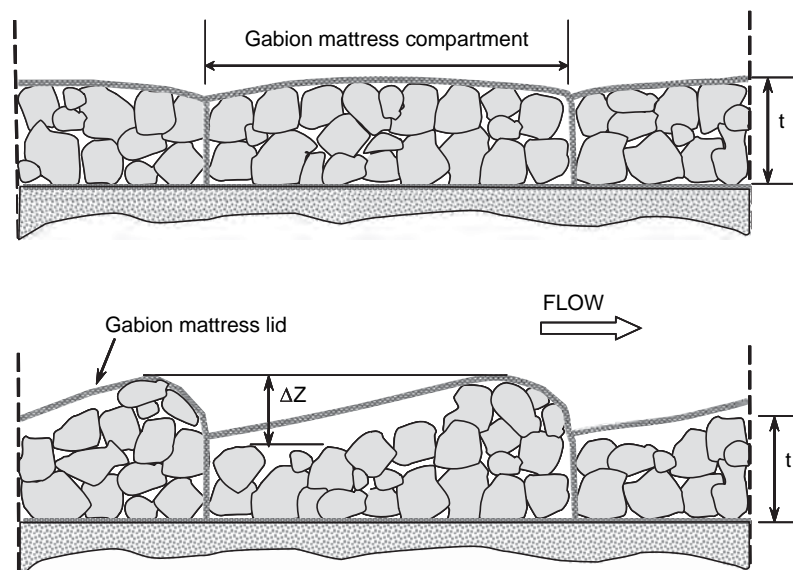
d_{50} = Median diameter of rock fill, ft (m)

t = Thickness of mattress, ft (m)

Finally, the mesh and wire should be checked for signs of deterioration. Waterborne sediment and debris can abrade PVC and galvanized coating, resulting in increased corrosion and thinning of the wire. When damage is detected, the damaged wire should be replaced.

3.3 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. Because gabion mattress systems are essentially an armor layer that is only one particle thick, any localized



Source: Maccafferri (undated)

Figure F3.1. Downstream migration of stones in a gabion mattress.

area of displaced mattresses or voids beneath the system is vulnerable to further destabilization during the next high flow event. As with any armor system, progressive failure from successive flows must be avoided by providing timely maintenance intervention.

Some opportunity may exist to repair gabion mattresses in place by using custom-fit mattresses, wire mesh, and rock (Parker et al. 1998).

Any voids underneath the system must be filled. Depending on the size of the void and the nature of the gabion mattress system, voids can be filled by the following actions:

- Removing the mattress and geotextile, filling the void area with proper fill material, providing proper compaction of the filled area, replacing the geotextile and mattress, and *relacing* the system
- Removing the mattresses, filling the void with sand-filled geocontainers having the same filtration capacity as the original geotextile, and then replacing the mattresses and *relacing* the system
- Filling the void by pumping sand and gravel, concrete, or grout into the void via tremie pipe

3.4 Performance Evaluation

The evaluation of any gabion mattress system's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. To properly assess the performance of gabion mattresses, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the gabion mattresses. Present-day channel cross-section geometry and planform should be compared to those at the time of installation. Both lateral and vertical instability of the channel can significantly alter hydraulic conditions at the piers. Approach flows may exhibit an increasingly severe angle of attack (skew) over time, increasing the hydraulic loading on the gabion mattress.

It is recognized that the person making the performance evaluation will probably not be the inspector; however, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported to allow comparison to the initial capital improvement cost.

3.4.1 Performance Rating Guide

To guide the performance evaluation for gabion mattress systems as a pier scour countermeasure, a rating system is presented in this section. It establishes numerical ratings from 0 (worst) to 6 (best) for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the current condition of the countermeasure?

Tables F3.1 through F3.3 present a rating system for gabion mattress pier scour countermeasures. A single numerical score is not intended; rather, an independent rating (0-6 or U) is given for each of the three topical areas. Recommended actions corresponding to the rating codes are also provided.

Table F3.1. Rating system for gabion mattress: hydraulic history.

Code	Hydraulic History	Code	Hydraulic History
U	N/A	3	Moderate: The countermeasure has experienced one or more flows greater than the 10-year event.
6	Extreme: The countermeasure has experienced one or more flows greater than the 100-year event.	2	Low: The countermeasure has experienced one or more flows greater than the 5-year event.
5	Severe: The countermeasure has experienced one or more flows greater than the 50-year event.	1	Very Low: The countermeasure has experienced one or more flows greater than the 2-year event.
4	High: The countermeasure has experienced one or more flows greater than the 25-year event.	0	Negligible: The countermeasure has not experienced any flows greater than a 2-year event.

Table F3.2. Rating system for gabion mattress: maintenance history.

Code	Maintenance History	Code	Maintenance History
U	N/A	3	Moderate: The system has required occasional maintenance since installation.
6	None Required: No maintenance has been needed since installation.	2	High: Frequent maintenance has been required.
5	Very Low: The system has required maintenance for very small, local areas once or twice.	1	Very High: Significant maintenance is usually required after flood events.
4	Low: The system has required minor maintenance.	0	Excessive: The system typically requires maintenance every year.

Table F3.3. Rating system for gabion mattress: current condition

Code	Description of Current Condition	Code	Description of Current Condition
U	The gabion system is uninspectable, due to burial by sediment, debris, or other circumstance.	3	Fair: Stone fill in mattress has shifted but filter and subgrade are not exposed. No stone fill has been lost. Broken or corroded wire has been replaced or repaired.
6	Excellent: The system is in excellent condition, with no shifting of stone fill, and no broken, corroded, or kinked wire.	2	Poor: Stone fill has shifted severely and the filter or subgrade is exposed. Wire has been broken or corroded and stone fill has been removed.
5	Very Good: The system exhibits only minor deterioration in localized areas.	1	Badly Damaged: The system has experienced substantial deterioration in terms of loss of stone fill and/or undermining. Wire is broken or corroded.
4	Good: Minor shifting of stone fill, no evidence of loss of stone fill. Some broken or corroded wire noticed, but mattresses are intact and functional.	0	Severe: The system has suffered damage such that it is no longer repairable. The only recourse is to remove the entire installation and replace it with a redesigned countermeasure.

Recommended actions based on current condition rating:

Code U: The gabion mattress system cannot be inspected. A plan of action should be developed to determine the condition of the installation. Possible remedies may include removal of debris, excavation during low flow, probing, or non-destructive testing using ground-penetrating radar or seismic methods.

Codes 6 or 5: Continue periodic inspection program at the specified interval.

Codes 4 or 3: Increase inspection frequency. The rating history of the installation should be tracked to determine if a downward trend in the rating is evident. Depending on the nature of the gabion mattress application, the installation of monitoring instruments might be considered.

Code 2: The maintenance engineer's office should be notified and maintenance should be scheduled. The cause of the low rating should be determined, and consideration given to redesign and replacement. Materials other than gabion mattresses might be considered.

Codes 1 or 0: The maintenance engineer's office should be notified immediately. Depending upon the nature of the gabion application, other local officials and/or law enforcement agencies may also need to be notified.

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APPENDIX G

Guidelines for Pier Scour Countermeasures Using Grout-Filled Mattresses

Introduction, G-2

1 Design and Specification, G-3

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Introduction

Grout-filled mattresses (mats) are composed of a double layer of strong synthetic fabric, typically woven nylon or polyester, sewn into a series of pillow-shaped compartments that are connected internally by ducts. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Mattresses are typically sewn together or otherwise connected (less commonly) by special zips, straps, or ties prior to filling.

When set, the grout forms a mattress made up of a grid of interconnected blocks. Grout-filled mattresses are reinforced by cables laced through the mattress (Figure G1.1) before the concrete is pumped into the fabric form, creating what is often called an articulating block mat (ABM). Flexibility and permeability are important functions for pier scour countermeasures. Therefore, systems that incorporate filter points or weep holes (allowing for pressure relief through the mat) combined with relatively small-diameter ducts (to allow breakage and articulation between blocks) are the preferred products.

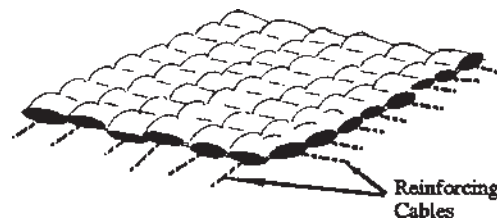
Grout-filled mattress systems can range from very smooth, uniform surface conditions that approach cast-in-place concrete in terms of surface roughness, to extremely irregular surfaces exhibiting roughness similar to that of moderate size rock riprap. Because this type of revetment is fairly specialized, comprehensive technical information on specific mattress types and configurations is available from a number of manufacturers of this type of revetment. Mattresses are typically available in standard nominal thicknesses of 4, 6, and 8 in. (100, 150, and 200 mm). A few manufacturers produce mattresses up to 12 in. (300 mm) thick.

There is limited field experience with the use of grout-filled mattress systems as a scour countermeasure for bridge piers. More frequently, these systems have been used for shoreline protection, protective covers for underwater pipelines, and channel armoring where the mattress is placed across the entire channel width and keyed into the abutments or banks. The guidance for pier scour applications provided in this appendix has been developed primarily from Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al. 2001) and the results of NCHRP Project 24-07(2) (Lagasse et al. 2007).

The benefits of grout-filled mattresses are that the fabric installation can be completed quickly and without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the forms and pumping them with concrete grout can be performed in areas where room for construction equipment is limited.

This document is organized into three parts:

- Part 1 provides design and specification guidelines for grout-filled mattress systems.
- Part 2 presents construction guidelines.
- Part 3 provides guidance for inspection, maintenance, and performance evaluation of grout-filled mattress systems used as a pier scour countermeasure.



Source: Fotherby (1995)

Figure G1.1. Grout-filled mattress with reinforcing cables, also known as articulating block mat.

Part 1: Design And Specification

1.1 Materials

1.1.1 Fabric Form

The geotextile composing the fabric form must exhibit sufficient strength to resist the pressure of the grout during filling. Cords shall connect the two layers of fabric at the center of each compartment. The cords shall be interwoven with the fabric in two sets of four cords each: one set for the upper layer and one set for the lower layer. Each cord shall have a minimum breaking strength of 160 lb (712 N).

The grout-filled ducts shall be no more than 10% of the maximum thickness of the block compartment so that flexibility and articulation can be achieved in the finished installation. Cables shall enter and exit each compartment through opposing grout ducts; alternatively, cable ducts may be provided for insertion of cables through each compartment. When cable ducts are used, the maximum allowable diameter shall be 1.0 in. (25 mm).

The geotextile composing the fabric form shall meet or exceed the values shown for the properties shown in Table G1.1 (Iowa Department of Transportation 2004).

1.1.2 Cables

Cables shall be installed between the two layers of fabric and through the compartments in a manner that provides for lateral and longitudinal connection. The cables shall enter and exit the compartments through opposing grout ducts. Cables shall be high-tenacity, low-elongation continuous filament polyester fibers, with a core contained within an outer jacket. The core shall be 65% to 75% of the total weight of the cable.

Cable splices shall be made with aluminum compression fittings such that a single fitting results in a splice strength of 80% of the breaking strength of the cable. Two fittings separated by a minimum of 6 in. (150 mm) shall be used per splice. When the installation is completed, the cables and splices shall be completely encased by the concrete grout.

Table G1.1. Geotextile minimum property requirements.

Property	Test Method	Units	Value
Composition			Nylon or polyester
Mass per unit area (double layer)	ASTM D 5261	oz/yd ² (g/m ²)	12 (403)
Thickness	ASTM D 5199	mils (mm)	25 (0.6)
Mill width		in (m)	76 (1.92)
Wide-width tensile strength (Machine direction)	ASTM D 4595	lbf/in (kN/m)	140 (24.5)
(Cross direction)	ASTM D 4595	lbf/in (kN/m)	110 (19.3)
Elongation at break (Machine direction)	ASTM D 4595	%	20
(Cross direction)	ASTM D 4595	%	30
Trapezoid tear strength (Machine direction)	ASTM D 4533	lbf (N)	150 (665)
(Cross direction)	ASTM D 4533	lbf (N)	100 (445)
Apparent opening size	ASTM D 4751	US Std Sieve (mm)	40 (0.425)
Flow Rate	ASTM D 4491	gal/min/ft ² (l/min/m ²)	90 (3665)

Notes:

1. Conformance of fabric to specification property requirements per ASTM D 4759.
2. Numerical values represent minimum average roll values (MARV). Lots shall be sampled per ASTM D 4354.

1.1.3 Grout

The concrete grout shall consist of a mixture of Portland cement, fine aggregate, water, admixtures, and fly ash (optional) to provide a pumpable slurry. The grout shall have an air content of not less than 5% nor more than 8% of the volume of the grout. The mix shall obtain a minimum 28-day compressive strength of 2,000 lb/in² (13,750 kPa). The mix shall result in a dry unit weight of the cured concrete of no less than 130 lb/ft³ (2,080 kg/m³). The grout shall be tested for flowability using the flow cone method of ASTM D 6449 and shall have an efflux time not less than 9 seconds nor more than 12 seconds using this method.

The engineer may require adjustment of the mix proportions to achieve proper solids suspension and optimum flowability. After the mix has been designated, it shall not be changed without approval of the engineer. A recommended basic mix design consists of the following:

- **Cement:** Cement shall be Portland Type I or Type II, at the rate of 10 sacks (940 lb) per cubic yard (558 kg/m³).
- **Fly Ash:** Fly ash may be substituted for cement for up to 25% by weight (mass) of cement.
- **Fine aggregate:** 2,100 lb (surface dry weight)/yd³ (1,246 kg [surface dry weight]/m³).
- **Water:** 45 gal (375 lb)/yd³ (170 L [221 kg]/m³), or enough to provide a thick, creamy consistency.
- **Air-entraining admixtures:** Air-entraining admixtures may be required to achieve the required air content.
- **Liquid curing compounds:** Liquid curing compounds may be required to achieve the required strength and set time.

1.1.4 Grout-Filled Mattress

When installed, the grout-filled mattress shall exhibit the nominal properties as shown in Table G1.2.

1.2 Hydraulic Stability Design Procedure

Hydrodynamic forces of drag and lift both act to destabilize a grout-filled mattress. The destabilizing forces are resisted by the grout-filled mattress weight and frictional resistance between the bottom of the grout-filled mattress and the channel subgrade material. While the individual slabs may articulate within the mattress and the mattress remains structurally sound, the general design approach is to consider the mattress as a rigid monolithic layer. Because grout-filled mattresses are in essence a thin-section monolithic layer, the mode of failure exhibited in field installations is one of sliding. In the following analysis, it is assumed that potential uplift force due to soil water pressure beneath the mattress is negligible, or that allowance for pressure relief has been made by installing weep holes or selecting a mattress with integral filter points between the blocks (compartments).

Figure G1.2 presents a schematic diagram of the forces acting to destabilize a grout-filled mattress on a channel bed. The analysis methodology purposely omits any restraining forces due to

Table G1.2. Nominal grout-filled mattress properties.

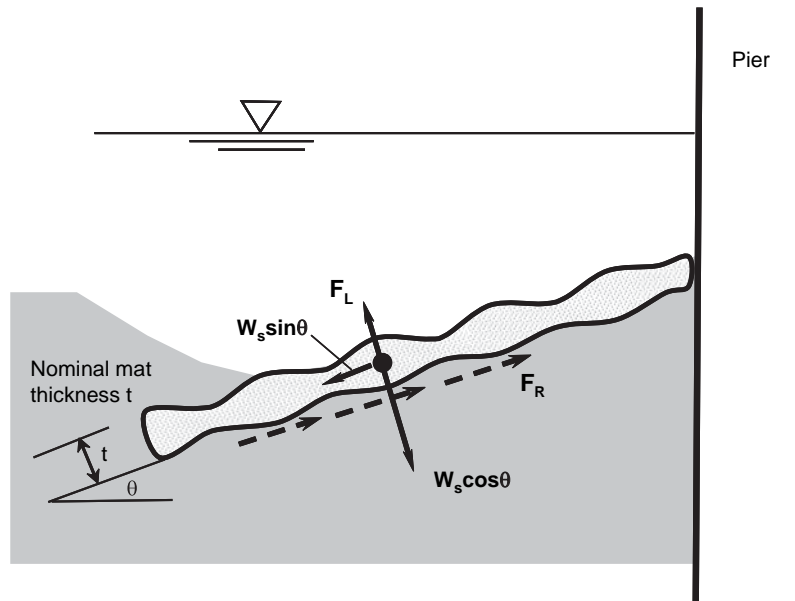
Property	4-in. (100-mm) mattress	6-in. (150-mm) mattress	8-in. (200-mm) mattress
Average thickness, in. (mm)	4 (100)	6 (150)	8 (200)
Mass per unit area, lb/ft ² (kg/m ²)	45 (220)	68 (330)	90 (440)
Mass per block, lb (kg)	88 (40)	188 (85)	325 (148)
Nominal block dimensions, in. (m)	20x14 (0.5x0.36)	20x20 (0.5x0.5)	20x26 (0.5x0.66)
Cable diameter, in. (mm)	0.25 (6.35)	0.312 (7.94)	0.312 (7.94)
Cable breaking strength, lbf (kN)	3,700 (16.5)	4,500 (20)	4,500 (20)

Stabilizing Forces:

$$F_R = \text{Frictional resistance to sliding} \\ = \mu F_N = \mu (W_s \cos \theta \cos \alpha - F_L)$$

Destabilizing Forces:

$$F_D = \text{Drag force (into page)} \\ W_s \sin \theta = \text{Submerged weight parallel to slope}$$



Source: Lagasse et al. (2001)

Figure G1.2. Forces acting on a grout-filled mattress on a channel bed.

cables or the additional stability afforded by mechanical anchoring devices for the sake of conservatism in design.

1.2.1 Selecting a Target Factor of Safety

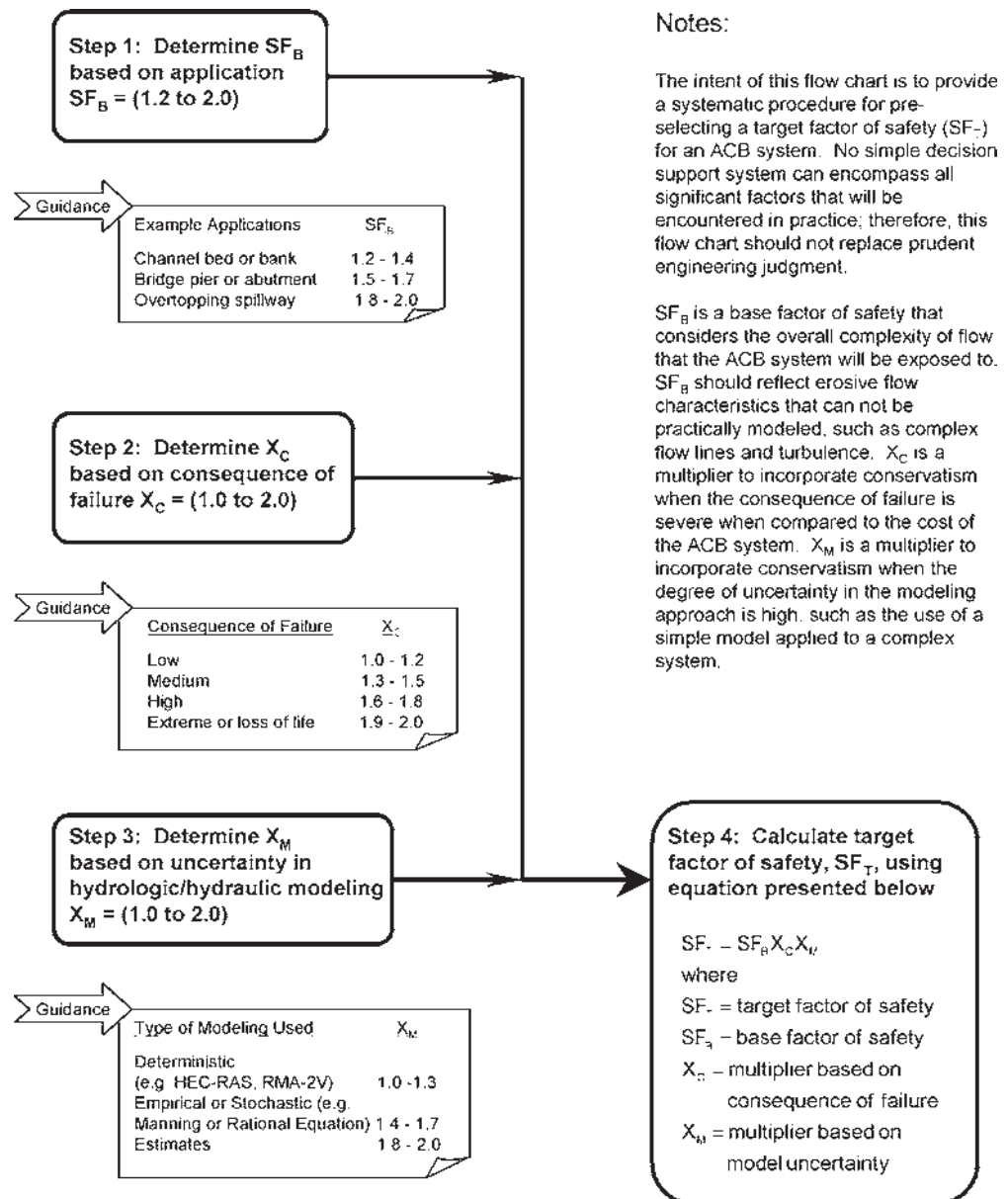
The designer must determine what factor of safety should be used for a particular application. Typically, a minimum factor of safety of 1.2 is used for revetment (bank protection) when the project hydraulic conditions are well known and the installation can be conducted under well-controlled conditions. Higher factors of safety are typically used for protection at bridge piers, abutments, and channel bends because of the complexity in computing shear stress at these locations.

The Harris County Flood Control District (HCFCD), Texas, has developed a simple flowchart approach that considers the type of application, uncertainty in the hydraulic and hydrologic models used to calculate design conditions, and consequences of failure to select an appropriate target factor of safety to use when designing an articulated concrete block (ACB) installation (Ayers Associates 2001). In this approach, the minimum allowable factor of safety for ACBs at bridge piers is 1.5. This value is then multiplied by two factors, each greater than 1.0, to account for risk and uncertainty. Figure G1.3 shows the HCFCD flowchart method. The method is also considered appropriate for the use of grout-filled mattresses at piers.

1.2.2 Design Method

Lagasse et al. (2001) provides the following grout-filled mattress selection and sizing criteria based on analysis of sliding stability on the channel bed. The sliding safety factor (SF) is a ratio of forces resisting sliding to forces causing sliding to occur:

$$SF = \mu \left[\frac{t(\gamma_c - \gamma_w) \cos \theta \cos \alpha - \tau_{des}}{\sqrt{[t(\gamma_c - \gamma_w) \sin \theta]^2 + \tau_{des}^2}} \right] \quad (G1.1)$$



Source: Ayres Associates (2001)

Figure G1.3. Selecting a target factor of safety.

where

SF = Safety factor against sliding

μ = Coefficient of static friction (dimensionless)

t = Thickness of grout mattress, ft (m)

γ_c = Unit weight of grout, lb/ft³ (N/m³)

γ_w = Unit weight of water, 62.4 lb/ft³ (9,810 N/m³)

θ = Angle of side slope (i.e., toe down angle), degrees

α = Angle of bed slope, degrees

τ_{des} = Design shear stress at base of pier, lb/ft² (N/m²)

In practice, the coefficient of static friction μ depends on the characteristics of the mattress-subsoil interface, which is a function of the mattress geometry, geotextile, soil type, and degree

to which the mattress can be seated into the subsoil to achieve intimate contact. Manufacturers typically supply the value of μ for use with their various products for different soil types. These design values may often be quoted as an equivalent friction angle, δ , expressed in degrees. The relationship between μ and δ is:

$$\mu = \tan \delta \quad (\text{G1.2})$$

Manufacturers should also supply the appropriate Manning's n resistance coefficient for each product. Grout-filled mattress 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.

The design conditions in the immediate vicinity of a bridge pier are more severe than the approach conditions upstream; therefore, the *local* velocity and shear stress should be used in the design equations. As recommended in HEC-23 (Lagasse et al. 2001), the section-average approach velocity, V_{avg} , must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg} \quad (\text{G1.3})$$

where

V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)

K_1 = Shape factor equal to 1.5 for round-nose piers and 1.7 for square-edged piers

K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for pier near the bank in a straight reach to 1.7 for pier located in the main current of flow around a sharp bend)

V_{avg} = Section-average approach velocity (Q/A) upstream of bridge, ft/s

If the local velocity, V_{local} , is available from stream tube or flow distribution output from a one-dimensional (1-D) model, or directly computed from a two-dimensional (2-D) model, then only the pier shape coefficient should be used to determine the design velocity. The maximum local velocity is recommended since the channel could shift and the maximum velocity could impact any pier:

$$V_{des} = K_1 V_{local} \quad (\text{G1.4})$$

The local shear stress at the base of the pier, τ_{des} , is calculated using a rearranged form of Manning's equation:

$$\tau_{des} = \frac{\gamma_w}{y^{1/3}} \left(\frac{n V_{des}}{K_u} \right)^2 \quad (\text{G1.5})$$

where

τ_{des} = Design shear stress, lb/ft² (N/m²)

γ_w = Unit weight of water, 62.4 lb/ft³ (9,810 N/m³)

y = Depth of flow at pier, ft (m)

n = Manning's n for the grout mattress (typical range 0.020–0.030)

K_u = 1.486 for U.S. customary units, 1 for SI units

1.3 Layout Dimensions

Based on small-scale laboratory studies performed for NCHRP Project 24-07(2) (Lagasse et al. 2007), the optimum performance of grout-filled mattresses as a pier scour countermeasure was obtained when the mattresses were extended a distance of at least 2 times the pier width in all directions around the pier.

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential caused by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor, K_α , which is a function of the width, a , and length, L , of the pier (or pile bents) and the skew angle, α , as given below (after Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cos \alpha + L \sin \alpha}{a} \right)^{0.65} \quad (\text{G1.6})$$

Grout-filled mattresses should be placed so that the long axis is parallel to the direction of flow. Where only clear-water scour is present, the mattresses may be placed horizontally such that the top of the mattress is flush with the bed elevation; however, when other types of scour are present, the mattresses must be sloped away from the pier in all directions such that the depth of the system at its periphery is greater than the depth of contraction scour and long-term degradation (Figure G1.4).

Tests conducted under NCHRP Project 24-07(2) confirmed that grout-filled mattresses can be effective scour countermeasures for piers under clear-water conditions. **However, when dune-type bed forms were present, the mattresses were subject to both undermining and uplift, even when they were toed down below the depth of the bed-form troughs. Therefore, grout-filled mattresses are not recommended for use as pier scour countermeasures under live-bed conditions when dunes may be present.**

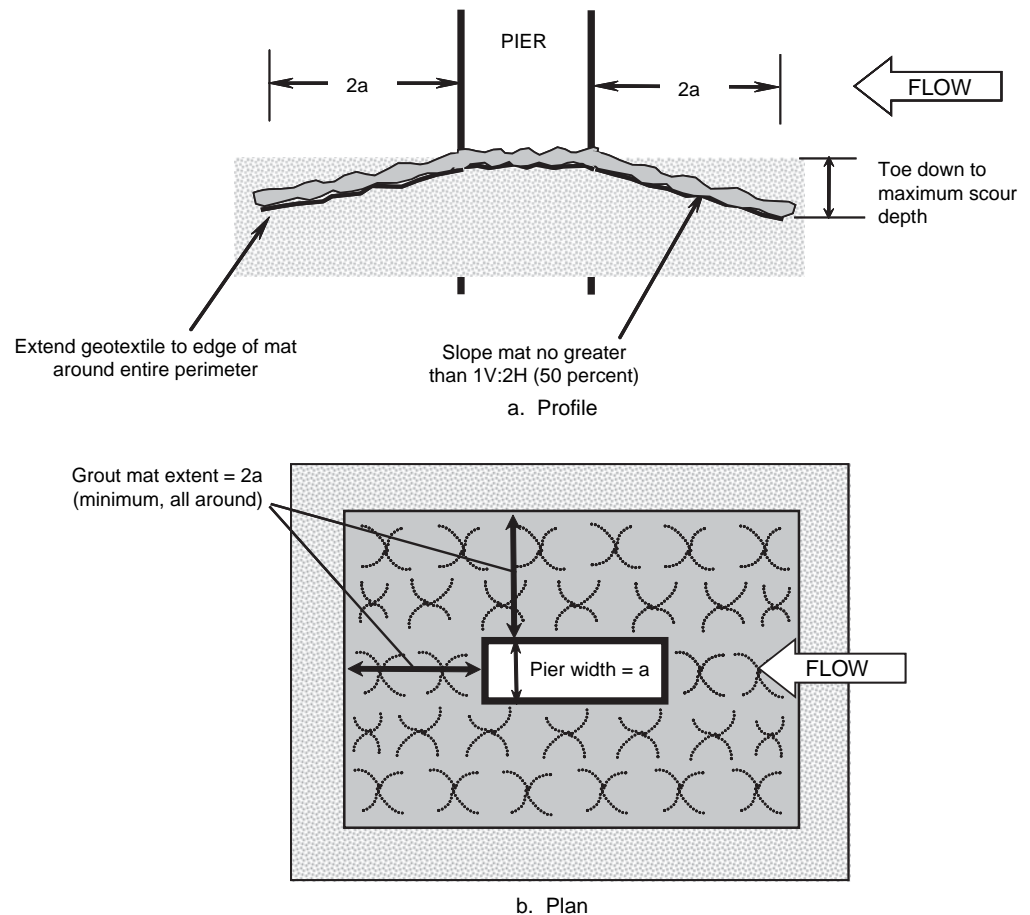


Figure G1.4. Grout-filled mattress layout diagram for pier scour countermeasures.

The mattresses should not be laid on a slope steeper than 1V:2H (50%). In some cases, this criterion may result in mattresses being placed further than two pier widths away from the pier. Mattresses must be placed so the top surface of the mattress is level with the ambient channel bed elevation to reduce susceptibility to uplift forces.

A filter layer is typically required for grout-filled mattresses at bridge piers. The filter should be extended fully beneath the grout-filled mat. When using a granular stone filter the layer thickness should be at least 4 times the d_{50} size of the filter stone or 6 in. (15 cm), whichever is greater. The granular filter thickness should be increased by 50% when placing under water.

1.4 Filter Requirements

The importance of the filter component of a grout-filled mattress installation should not be underestimated. Two kinds of filters are used in conjunction with grout-filled mattresses: granular filters and geotextile filters. Some situations call for a composite filter consisting of both a granular layer and a geotextile. The specific characteristics of the base soil determine the need for, and design considerations of, the filter layer. In cases where the base soil is composed primarily of relatively large particles (coarse sands, gravels, or cobbles), a filter layer may not be necessary. ***Grout-filled mattresses are not recommended for use when dune-type bed forms may be present.***

The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain *all* the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind.

1.4.1 Geotextile Filter Properties

Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.***

For compatibility with site-specific soils, geotextiles must exhibit the appropriate values of permeability, pore size (otherwise known as apparent opening size), and porosity (or percent open area). In addition, geotextiles must be sufficiently strong to withstand stresses during installation. These values are available from manufacturers. The following list briefly describes the most relevant properties.

- **Permeability.** The permeability, K , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water across its thickness. It is typically reported in units of centimeters per second (cm/s). This property is directly related to the filtration function that a geotextile must perform, where water flows perpendicularly through the geotextile into a crushed stone bedding layer, perforated pipe, or other more permeable medium. The geotextile must allow this flow to occur without being impeded. A value known as the permittivity, ψ , is used by the geotextile industry to more readily compare geotextiles of different thicknesses. Permittivity, ψ , is defined as K divided by the geotextile thickness, t , in centimeters; therefore, permittivity has a value of (s)⁻¹. Permeability (and permittivity) is extremely important in filter design. For grout-filled mattress installations, the permeability of the geotextile should be at least 10 times greater than that of the underlying material:

$$K_g > 10K_s$$

where

K_g = Permeability of geotextile (cm/s)

K_s = Permeability of subgrade soil (cm/s)

- **Transmissivity.** The transmissivity, θ , of a geotextile is a calculated value that indicates the ability of a geotextile to transmit water within the plane of the fabric. It is typically reported in units of square centimeters per second. This property is directly related to the drainage function and is most often used for high-flow drainage nets and geocomposites, not geotextiles. Woven monofilament geotextiles have very little capacity to transmit water in the plane of the fabric, whereas non-woven, needle-punched fabrics have a much greater capacity due to their three-dimensional microstructure. Transmissivity is not particularly relevant to grout-filled mattress filter design.
- **Apparent opening size (AOS).** Also known as equivalent opening size, this measure is generally reported as O_{95} , which represents the aperture size such that 95% of the openings are smaller. In similar fashion to a soil gradation curve, a geotextile hole distribution curve can be derived. The AOS is typically reported in millimeters, or in equivalent U.S. standard sieve size.
- **Porosity.** Porosity is a comparison of the total volume of voids to the total volume of geotextile. This measure is applicable to non-woven geotextiles only. Porosity is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Percent open area (POA).** POA is a comparison of the total open area to the total geotextile area. This measure is applicable to woven geotextiles only. POA is used to estimate the potential for long-term clogging and is typically reported as a percentage.
- **Thickness.** As mentioned above, thickness is used to calculate traditional permeability. It is typically reported in millimeters or mils (thousandths of an inch).
- **Grab strength and elongation.** Grab strength is the force required to initiate a tear in the fabric when pulled in tension. It is typically reported in Newtons or pounds as measured in a testing apparatus having standardized dimensions. The elongation measures the amount the material stretches before it tears and is reported as a percentage of its original (unstretched) length.
- **Tear strength.** Tear strength is the force required to propagate a tear once initiated. It is typically reported in Newtons or pounds.
- **Puncture strength.** Puncture strength is the force required to puncture a geotextile using a standard penetration apparatus. It is typically reported in Newtons or pounds.

Table G1.3 provides the recommended characteristics for geotextile filters. There are many other tests to determine various characteristics of geotextiles; only those deemed most relevant to applications involving grout-filled mattress installation at piers have been discussed here. As previously mentioned, geotextiles should be able to withstand the rigors of installation without suffering degradation of any kind. Long-term endurance to stresses such as ultraviolet solar radiation or continual abrasion are considered of secondary importance, because once the geotextile has been installed and covered by a grout-filled mattress, these stresses do not represent the environment that the geotextile will experience in the long term.

1.4.2 Geotextile Filter Design Procedure

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. A decision tree is provided as Figure G1.5 to assist in determining the appropriate soil retention criterion for the geotextile. The figure includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater than 30% clay or is predominantly fine-grained soil (more than 50% passing the #200 sieve).

If a granular transition layer is required, the geotextile should be designed to be compatible with the properties of the granular layer. If the required AOS is smaller than that of available

Table G1.3. Recommended requirements for geotextile properties.

Test Designation	Property	Allowable value ⁽¹⁾		Comments
		Elongation < 50% ⁽²⁾	Elongation > 50% ⁽²⁾	
ASTM D 4632	Grab Strength	> 315 lbs (Class 1) > 250 lbs (Class 2) > 180 lbs (Class 3)	> 200 lbs (Class 1) > 160 lbs (Class 2) > 110 lbs (Class 3)	From AASHTO M 288
ASTM D 4632	Sewn Seam Strength ⁽³⁾	> 270 lbs (Class 1) > 220 lbs (Class 2) > 160 lbs (Class 3)	> 180 lbs (Class 1) > 140 lbs (Class 2) > 100 lbs (Class 3)	From AASHTO M 288
ASTM D 4533	Tear Strength ⁽⁴⁾	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4833	Puncture Strength	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	> 110 lbs (Class 1) > 90 lbs (Class 2) > 70 lbs (Class 3)	From AASHTO M 288
ASTM D 4751	Apparent Opening Size	Per design criteria (Section 1.4 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 1.4 of this design guide)		Minimum allowable value
ASTM D 4355	Degradation by Ultraviolet Light	> 50% strength retained after 500 hours of exposure		Minimum allowable value
ASTM D 4873	Guide for Identification, Storage, and Handling			Provides information on identification, storage, and handling of geotextiles.
ASTM D 4759	Practice for the Specification Conformance of Geosynthetics			Provides information on procedures for ensuring that geotextiles at the jobsite meet the design specifications.

(1) Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class. The following descriptions have been modified from AASHTO M 288:

- *Class 1* is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including when placement of riprap must occur in multiple lifts, when drop heights may exceed 1 ft (0.3 m) or when repeated vehicular traffic on the installation is anticipated.
- *Class 2* is recommended for installation conditions where placement in regular, single lifts are expected and little or no vehicular traffic on the installation will occur, or when placing individual rocks by clamshell, orange-peel grapple or specially equipped hydraulic excavator with drop heights less than 1 ft.
- *Class 3* is specified for the least severe installation environments, with drop heights less than 1 ft onto a bedding layer of select sand, gravel or other select imported material.

(2) As measured in accordance with ASTM D 4632.

(3) When seams are required.

(4) The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 55 lbs. The MARV corresponds to a statistical measure whereby 2.5% of the tested values are less than the mean value minus two standard deviations (Koerner 1998).

geotextiles, then a granular transition layer is required. However, this requirement can be waived if the base soil exhibits the following conditions for hydraulic conductivity, K ; plasticity index, PI ; and undrained shear strength, c :

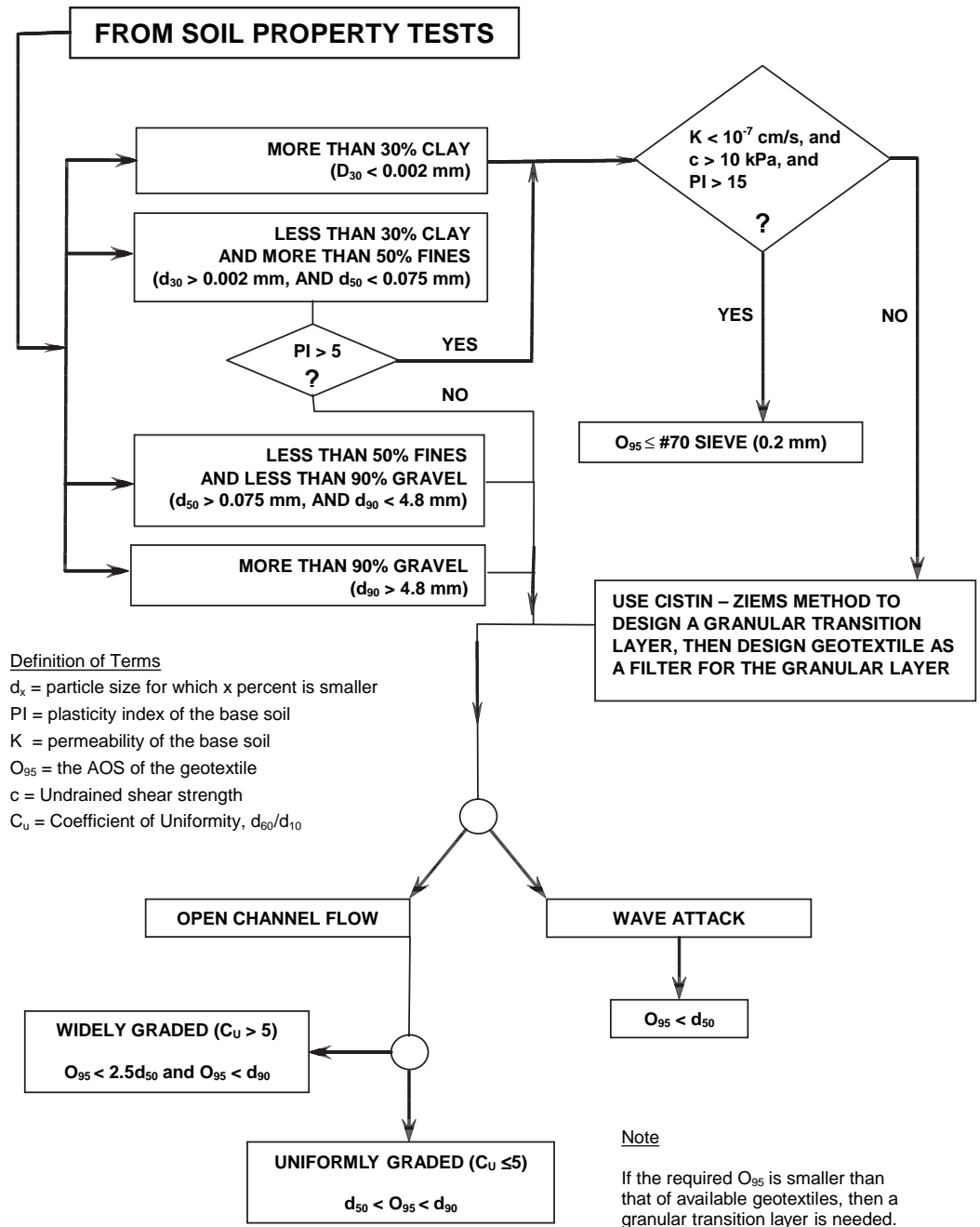
$$K < 1 \times 10^{-7} \text{ cm/s}$$

$$PI > 15$$

$$c < 10 \text{ kPa}$$

Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS smaller than a #70 sieve (approximately 0.2 mm) can be used with soils meeting these conditions and essentially functions more as a separation layer than a filter.

Step 3. Determine Permeability Criterion. The permeability criterion requires that the filter exhibit a permeability at least 4 times greater than that of the base soil (Koerner 1998) and for critical or severe applications such as at piers, at least 10 times greater (Holtz et al. 1995). Generally speaking, if the permeability of the base soil or granular filter has been determined from laboratory testing, that value should be used. If laboratory testing was not conducted, then an estimate of permeability based on the particle size distribution should be used.



Source: modified from Koerner (1998)

Figure G1.5. Geotextile selection based on soil retention.

To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s⁻¹. Typically, the designer will need to contact the geotextile manufacturer to obtain values of permeability, permittivity, and thickness.

Step 4. Select a Geotextile that Meets the Required Strength Criteria. Strength and durability requirements depend on the installation environment and the construction equipment that is being used. See Table G1.3 for recommended values based on AASHTO M 288, “Geotextile Specification for Highway Construction,” which provides guidance on allowable strength and elongation values for three categories of installation severity. For additional guidelines regarding the

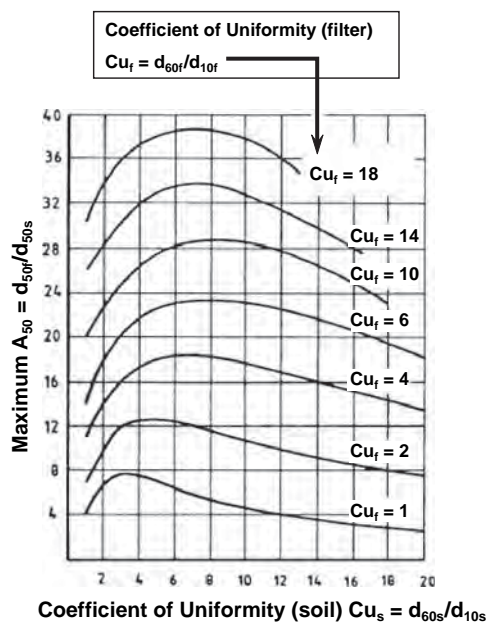
selection of durability test methods, refer to ASTM D 5819, “Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability.”

Step 5. Minimize Long-Term Clogging Potential. When a woven geotextile is used, its POA should be greater than 4% by area. If a non-woven geotextile is used, its porosity should be greater than 30% by volume. A good rule of thumb suggests that the geotextile having the largest AOS that satisfies the particle retention criteria should be used (provided of course that all other minimum allowable values described in this section are met as well).

1.4.3 Granular Filter Properties

Generally speaking, most required granular filter properties can be obtained from the particle size distribution curve for the material. Granular filters can be used alone or can serve as a transitional layer between a predominantly fine-grained base soil and a geotextile. The following list briefly describes the most relevant properties:

- **Particle size distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibaum (2004) presents a summary of a procedure originally developed by Cistin and Ziems whereby the d_{50} size of the filter is selected based on the coefficients of uniformity (d_{60}/d_{10}) of both the base soil and the filter material. With this method, the grain size distribution curves do not necessarily need to be approximately parallel. Figure G1.6 provides a design chart based on the Cistin–Ziems approach.
- **Permeability.** Permeability of a granular filter material is determined by laboratory test, or estimated using relationships relating permeability to the particle size distribution. The permeability of a granular layer is used to select a geotextile when designing a composite filter. For grout-filled mattress installations, the permeability of the granular filter should be at least 10 times greater than that of the underlying material.
- **Porosity.** Porosity is that portion of a representative volume of soil that is interconnected void space. It is typically reported as a dimensionless fraction or a percentage. The porosity of soils



Source: Heibaum (2004)

Figure G1.6. Granular filter design chart according to Cistin and Ziems.

is affected by the particle size distribution, the particle shape (e.g., round vs. angular), and degree of compaction and/or cementation.

- **Thickness.** Practical issues of placement indicate that a typical minimum thickness of 6 to 8 in. is specified. For placement under water, thickness should be increased by 50%.
- **Quality and durability.** Aggregate used for a granular filter should be hard, dense, and durable.

1.4.4 Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter. That approach was developed for subsoils consisting of well-graded sands and may not be widely applicable to other soil types. An alternative approach that is considered more robust in this regard is the Cistin–Ziems method.

The suggested steps for proper design of a granular filter using this method are outlined below. Note that the subscript “s” is used to represent the base (finer) soil, and “f” is used to represent the filter (coarser) layer.

Step 1. Obtain Base Soil Information. Typically, the required base soil information consists simply of a grain size distribution curve, a measurement (or estimate) of permeability, and the plasticity index (PI is required only if the base soil is more than 20% clay).

Step 2. Determine Key Indices for Base Soil. From the grain size information, determine the median grain size, d_{50} , and the coefficient of uniformity, $Cu_s = d_{60}/d_{10}$, of the base soil.

Step 3. Determine Key Indices for Granular Filter. One or more locally available aggregates should be identified as potential candidates for use as a filter material. The d_{50} and coefficient of uniformity, $Cu_f = d_{60}/d_{10}$, should be determined for each candidate filter material.

Step 4. Determine Maximum Allowable d_{50} for Filter. Enter the Cistin–Ziems design chart (Figure G1.6) with the coefficient of uniformity, Cu_s , for the base soil on the x-axis. Find the curve that corresponds to the coefficient of uniformity, Cu_f , for the filter in the body of the chart and, from that point, determine the maximum allowable A_{50} from the y-axis. Compute the maximum allowable d_{50f} of the filter using d_{50fmax} equals A_{50max} times d_{50s} . Check to see if the candidate filter material conforms to this requirement. If it does not, continue checking alternative candidates until a suitable material is identified.

Step 5. Check for Permeability. From laboratory permeameter tests or the grain size distribution of the candidate filter material, determine whether the hydraulic conductivity of the filter is at least 10 times greater than that of the subsoil.

Step 6. Filter Layer Thickness. For practicality of placement, the nominal thickness of a single filter layer should not be less than 6 in. (15 cm). When multiple filter layers are required, each individual layer should range from 4 to 8 in. (10 to 20 cm) in thickness (Brown and Clyde 1989).

NOTE: *Grout-filled mattresses are not recommended for use when dune-type bed forms may be present.*

1.5 Guidelines for Seal Around the Pier

An observed key point of failure for grout-filled mattress systems at bridge piers during laboratory studies occurs at the joint where the mattress meets the bridge pier. During NCHRP Project 24-07(2) and according to Fotherby (1995), securing the geotextile to the pier prevented the leaching of the bed material from around the pier. This procedure worked successfully in the laboratory, but there are constructability implications that must be considered when this technique is used in the field, particularly when the mattress is placed under water.

A grout seal between the grout-filled mattress and the pier is recommended. A grout seal is not intended to provide a structural attachment between the mattress and the pier, but instead is a simple method for plugging gaps to prevent bed sediments from winnowing out from beneath the system. In fact, structural attachment of the mattress to the pier is strongly discouraged. The transfer of moments from the mattress to the pier may affect the structural stability of the pier, and the potential for increased loadings on the pier must be considered. When a grout seal is placed under water, an anti-washout additive is required.

1.6 Anchors

Anchors may be used with grout-filled mattresses; however, the layout guidance presented in Section 1.3 indicates that the system should be toed down to a termination depth at least as deep as any expected contraction scour and long-term degradation. Where such toedown depth cannot be achieved, for example where bedrock is encountered at shallow depth, anchoring the grout-filled mattress along the front (upstream) and sides of the installation is recommended. The spacing of the anchors should be determined based on a factor of safety of at least 5.0 for pullout resistance based on calculated drag on the exposed leading edge. Spacing between anchors of no more than 4 ft (1.3 m) is recommended. The following example is provided:

Given:

$$\rho = \text{Mass density of water, slugs/ft}^2 = 1.94$$

$$V = \text{Approach velocity, ft/s} = 10$$

$$\Delta z = \text{Height of grout-filled mattress, ft} = 0.5$$

$$b = \text{Width of mattress installation (perpendicular to flow), ft} = 40$$

Step 1: Calculate total drag force, F_d , on leading edge of system:

$$F_d = 0.5\rho V^2(\Delta z)(b) = 0.5(1.94)(10^2)(0.5)(40) = \mathbf{1,940 \text{ lb}}$$

Step 2: Calculate required uplift restraint using 5.0 safety factor:

$$F_{\text{restraint}} = 5.0(1,940) = \mathbf{9,700 \text{ lb}}$$

Step 3: Counting anchors at the corners of the system, calculate required pullout resistance per anchor (rounded to nearest 10 lb):

a) Assume 11 anchors at 4-ft spacing: $9,700 \text{ lb}/11 \text{ anchors} = \mathbf{880 \text{ lb/anchor}}$

b) Assume 21 anchors at 2-ft spacing: $9,700 \text{ lb}/21 \text{ anchors} = \mathbf{460 \text{ lb/anchor}}$

Anchors should never be used as a means to avoid toeing the system down to the full required extent where alluvial materials are present at depth. In this case, scour or bed-form troughs will simply undermine the anchors as well as the system in general.

Part 2: Construction

The guidance in this section has been developed to facilitate the proper installation of grout-filled mattress systems to achieve suitable hydraulic performance and maintain stability against hydraulic loading. The proper installation of grout-filled mattress systems is essential to the adequate functioning and performance of the system during the design hydrologic event. Guidelines are provided herein for maximizing the correspondence between the design intent and the actual field-finished conditions of the project.

Grout-filled mattresses form a continuous layer of strong synthetic fabric sewn into a series of bags or compartments that are filled in place with a concrete grout. Grout-filled mattress systems

can be supplied as cable-tied and non-cabled systems. Only cabled systems, also known in the industry as articulating block mats, are recommended for use as pier scour countermeasures. The term “articulating,” as used in this document, implies the ability of individual grout-filled compartments of the system to conform to changes in the subgrade while remaining interconnected by virtue of the cables.

This section addresses the preparation of the subgrade, geotextile placement, grout-filled mattress placement and installation, backfilling and finishing, and measurement and payment.

2.1 General Guidelines

The contractor is responsible for constructing the project according to the plans and specifications; however, ensuring conformance with the project plans and specifications is the responsibility of the owner. This responsibility is typically performed by the owner’s engineer and inspectors. Inspectors observe and document the construction progress and performance of the contractor. Prior to construction, the contractor should provide a quality control plan to the owner (for example, see ER 1180-1-6 [U.S. Army Corps of Engineers 1995]) and provide labor and equipment to perform tests as required by the project specifications.

Construction requirements for grout-filled mattress placement are included in the project plans and specifications. Standard grout-filled mattress specifications and layout guidance are found in Part 1 of this appendix. Recommended requirements for the mattresses, including the tests necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications, are provided. Tests for engineering properties of the fabric forms and filter materials should be provided by the manufacturer’s certified quality assurance/quality control (QA/QC) testing program at the point of manufacture. Grout mix specifications should be designated, and the appropriate tests for 28-day compressive strength, air content, dry density and flowability should be performed prior to mobilizing construction equipment to the site.

Inspection of grout-filled mattress placement consists of visual inspection of the placement operation and the finished surface. Inspection and quality assurance must be carefully organized to ensure that materials delivered to the job site meet the specifications. Acceptance of the work should not be made until measurement for payment has been completed. The engineer and inspectors reserve the right to reject incorrect or unsuitable materials (e.g., wrong geotextile, torn or ripped fabric, grout that does not exhibit the proper flowability, etc.) at the job site. Material that has been improperly placed should also be rejected throughout the duration of the contract and require removal and replacement at the contractor’s expense. Rejected material should be removed from the project site.

Construction techniques can vary tremendously because of the following factors:

- Size and scope of the overall project
- Size and weight of the materials
- Placement under water or in the dry
- Physical constraints to access and/or staging areas
- Noise limitations
- Traffic management and road weight restrictions
- Environmental restrictions
- Type of construction equipment available

Competency in construction techniques and management in all their aspects cannot be acquired from a book. Training on a variety of job sites and project types under the guidance of experienced senior personnel is required. The following sections provide some general information regarding construction of grout-filled mattress installations and some basic information and description of techniques and processes involved.

2.2 Materials

2.2.1 Grout-Filled Mattress Fabric Forms

Materials composing the grout-filled mattress system shall be in accordance with the requirements provided in Part 1 of this appendix. Fabric forms shall be sound and free of defects that would interfere with proper placement or that would impair the integrity of the system. Factory seams should be inspected to ensure integrity. Factory seams shall meet or exceed a strength of 90 lbf/in (15.75 kN/m) when tested in accordance with ASTM D 4884. Fabric may not be exposed to sunlight for more than 5 days prior to installation and grout filling.

Field sewing shall be permitted only to join the factory assembled fabric panels together. It is recommended that field seams be constructed of colored thread to allow broken thread or poor seams to be more visible. All field seams shall be made using two lines of U.S. Federal Standard Type 101 stitches with nylon and/or polyester thread.

2.2.2 Cables

Cables shall be jacketed polyester cable as described in Part 1 of this appendix.

2.2.3 Concrete Grout

Grout used to fill the fabric forms shall be a fluid, pumpable, fine aggregate concrete. The fine aggregate shall conform to ASTM C 33. Portland cement should comply with the requirements of ASTM C 150-04 Type I or Type II and AASHTO M85. See Part 1 of this appendix for more detailed information on grout mix specifications and required properties.

2.2.4 Filter

Geotextile. Either woven or non-woven, needle-punched fabrics may be used. If a non-woven fabric is used, it must have a mass density greater than 12 oz/yd² (400 g/m²). ***Under no circumstances may spun-bond or slit-film fabrics be allowed.*** Each roll of geotextile shall be labeled with the manufacturer's name, product identification, roll dimensions, lot number, and date of manufacture. Geotextiles shall not be exposed to sunlight prior to placement.

Granular Filters. Samples of granular filter material shall be tested for grain size distribution to ensure compliance with the gradation specification used in design. Sampling and testing frequency shall be in accordance with the requirements of the owner or the owner's authorized representative.

2.2.5 Subgrade Soils

When placement is in the dry, the grout-filled mattress system shall be placed on undisturbed native soil, on an excavated and prepared subgrade, or on acceptably placed and compacted fill. Unsatisfactory soils shall be considered those soils having excessive in-place moisture content; soils containing roots, sod, brush, or other organic materials; soils containing turf clods or rocks; or frozen soil. These soils shall be removed, and the excavation backfilled with approved material that is compacted prior to placement of the fabric forms. Unsatisfactory soils may also be defined as soils such as very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays, per the geotechnical engineer's recommendations.

When the grout-filled mattress system is placed under water, compaction of the subgrade is impractical. However, the surface must be relatively smooth, with no abrupt irregularities that would prevent intimate contact between the system and the subgrade. Under no circumstances may grout-filled mattresses be draped over boulders, bridged over subgrade voids, or placed over other irregularities that would prevent achievement of intimate contact between the system and the subgrade. Placing a granular filter layer or a sand transition layer

(with associated geotextile filter) may assist in achieving a suitable surface on which to place the grout-filled mattress system.

2.3 Installation

2.3.1 Subgrade Preparation

Stable and compacted subgrade soil shall be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes, embankment crests, benches, berms, and toes shall be compacted, shaped, and uniformly graded to facilitate the development of intimate contact between the grout-filled mattress system and the underlying grade. Termination between the grout-filled mattress system and a concrete slab, footer, pier, wall, or similar structure shall be sealed in a manner that prevents soil migration.

The subgrade soil conditions shall meet or exceed the required material properties described in Section 2.2.5 prior to placement of the grout-filled mattress. Soils not meeting the requirements shall be removed and replaced with acceptable material.

When placement is in the dry, the areas to receive the grout-filled mattress shall be graded to establish a smooth surface and ensure that intimate contact is achieved between the subgrade surface and the filter, and between the filter and the mattress. The subgrade should be uniformly compacted to the geotechnical engineer's site-specific requirements. If the subgrade surface for any reason becomes rough, corrugated, uneven, textured, or traffic marked prior to grout-filled mattress installation, such unsatisfactory portion shall be scarified, reworked, recompacted, or replaced as directed by the engineer.

When placement is under water, divers shall be used to ensure that the bed is free of logs, large rocks, construction materials, or other blocky materials that would create irregularities in the mattress surface, or that would create voids beneath the system as described in Section 2.2.5. Immediately prior to placement of the filter and grout-filled mattress system, the prepared subgrade must be inspected.

2.3.2 Placing the Filter

Whether the filter comprises one or more layers of granular material or is made of geotextile, its placement should result in a continuous installation that maintains intimate contact with the soil beneath. Voids, gaps, tears, or other holes in the filter must be avoided to the extent practicable, and the filter must be replaced or repaired when they occur.

Placement of Geotextile. The geotextile shall be placed directly on the prepared area, in intimate contact with the subgrade. When a geotextile is placed, it should be rolled or spread out directly on the prepared area and be free of folds or wrinkles. The rolls shall not be dragged, lifted by one end, or dropped. The geotextile should be placed in such a manner that placement of the overlying materials will not excessively stretch or tear the geotextile.

After geotextile filter placement, the work area shall not be trafficked or disturbed in a manner that might result in a loss of intimate contact between the grout-filled mattress, the geotextile, and the subgrade. The geotextile shall not be left exposed more than 48 hours so that potential damage due to ultraviolet radiation is minimized; therefore, the grout-filled mattress should be placed as soon as practicable.

The geotextile shall be placed so that upstream strips overlap downstream strips. Overlaps shall be in the direction of flow wherever possible. The longitudinal and transverse joints shall be overlapped at least 1.5 ft (46 cm) for dry installations and at least 3 ft (91cm) for below-water installations. If a sewn seam is to be used for the seaming of the geotextile, the thread to be used

shall consist of high-strength polypropylene or polyester and shall be resistant to ultraviolet radiation. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins, U-staples, or temporary weights such as sandbags may be used.

Placing Geotextiles Under Water. Placing geotextiles under water can be problematic for a number of reasons. Most geotextiles that are used as filters beneath grout-filled mattresses are made of polyethylene or polypropylene. These materials have specific gravities ranging from 0.90 to 0.96, meaning that they will float unless weighted down or otherwise anchored to the subgrade prior to placement of the grout-filled mattress (Koerner 1998).

Flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric (a condition that contractors refer to as “galloping”) that are extremely difficult to control. The preferred method of controlling geotextile placement is to isolate the work area from river currents by a temporary cofferdam. In mild currents, geotextiles precut to length can be placed by divers, with sandbags to hold the fabric temporarily.

At bridge piers, sand-filled geocontainers made of non-woven, needle-punched fabric are particularly effective for placement under water as shown in Figure G2.1. The geotextile fabric and sand fill that compose the geocontainers should be selected in accordance with the filter design criteria presented in Part 1 and placed such that they overlap to cover the required area. Geocontainers can be fabricated in a variety of dimensions and weights. Each geocontainer should be filled with sand to no more than 80% its total volume so that it remains flexible and “floppy.” The geocontainers also can serve to fill a pre-existing scour hole around a pier prior to placement of a grout-filled mattress system, as shown in Figure G2.1. For more information see Lagasse et al. (2006, 2007).

Placement of Granular Filter. For placing a granular filter, front-end loaders are the preferred method for dumping and spreading the material on slopes milder than approximately 4H:1V. A typical minimum thickness for granular filters is 0.5 to 1.0 ft (0.15 to 0.3 m). Placing granular media under water around a bridge pier is best accomplished using a large-diameter tremie pipe to control the placement location and thickness, while minimizing the potential for segregation. When a granular filter is placed under water, the thickness should be increased by 50%.

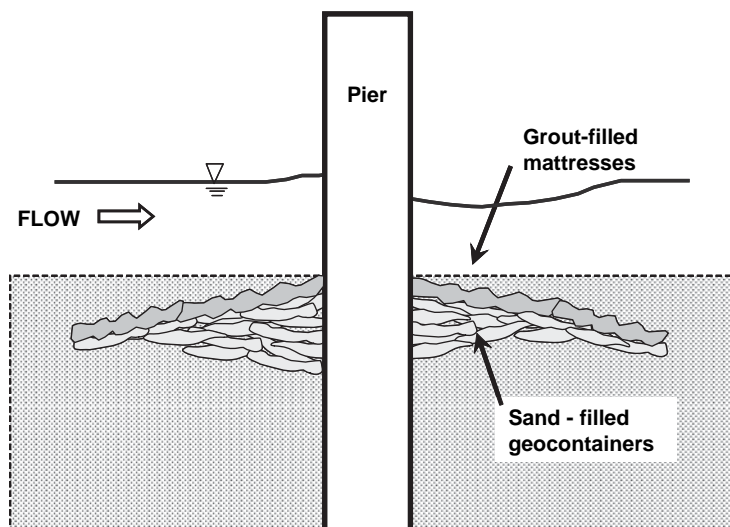


Figure G2.1. Schematic diagram showing the use of sand-filled geocontainers as a filter.

2.3.3 Placing the Grout-Filled Mattress System

Manufacturer's assembly instructions should be followed. Fabric forms shall be placed on the filter layer and arranged according to the contract drawings prior to field seaming. ***An excess of fabric should be included to allow for as much as a 10% contraction in size after filling of the fabric forms.*** The manufacturer should be consulted to determine the amount of contraction anticipated for site-specific conditions. Fabric forms should be positioned so that the direction of grout placement shown on the contract drawing is followed, with the preferred direction being from upstream to downstream. Filling must *always* be performed from the lowest elevation first to the uppermost elevation last. Prior to filling, the double layers of adjacent mattresses should be connected by sewing with a hand-held sewing machine or zipping, depending on the manufacturer's instructions, as described in Section 2.2.1. Custom fitting of mattresses around corners or curves should be done in accordance with the manufacturer's recommendations.

Care shall be taken during installation so as to avoid damage to the geotextile or subgrade during the installation process. Preferably, the grout-filled mattress placement and filling shall begin at the upstream section and proceed downstream. If a mattress system is to be installed starting downstream and proceeding in the upstream direction, a contractor option is to construct a temporary toe trench at the front edge of the mattress system to protect against flow that could otherwise undermine the system during flow events that may occur during construction. Only the amount of fabric forms that can be filled in a day should be laid into position. ***After being filled with grout, the mattresses shall not be pulled or pushed in any direction.***

Placement Under Water. Grout-filled mattresses placed under water require close observation and increased quality control to ensure a continuous countermeasure system. A systematic process for placing and continuous monitoring to verify that the grout is flowing to achieve the desired thickness is important.

Excavation, grading, and placement of grout-filled mattresses and filter under water require additional measures. For installations of a relatively small scale, the stream around the work area may be diverted during the low-flow season. For installations on larger rivers or in deeper water, the area can be temporarily enclosed by a cofferdam, which allows for construction dewatering if necessary. Alternatively, a silt curtain made of plastic sheeting may be suspended by buoys around the work area to minimize environmental degradation during construction. Once under water and in the correct positions, the individual fabric forms can be sewn together or otherwise connected by divers prior to filling with grout.

Depending on the depth and velocity of the water, sounding surveys using a sounding pole or sounding basket on a lead line, divers, sonar bottom profiles, and remote operated vehicles (ROVs) can provide some information about the mattress placement and toedown.

2.3.4 Filling of Mattresses

Concrete grout should be checked each day of construction using the flow cone method of ASTM D 6449 prior to pumping. Grout is pumped into the fabric form by inserting a small-diameter pipe into a slit or valve in the upper layer of fabric. A seal is typically created by wrapping the pipe with extra geotextile fabric.

Concrete spilled on the outside of the fabric form should be cleaned up immediately using a hand trowel and the area smoothed by cloth or trowel. Freshly pumped concrete units should never be washed or sprayed under pressure in an effort to clean or remove spills. Washing may remove Portland cement from inside the fabric form, resulting in a poor surface condition of the finished product.

Fabric forms might be considered to serve as filters as well as forms (Sprague and Koutsourais 1992). Water in the grout mix will bleed through the fabric, producing a reduction in

the water/cement ratio, which increases strength and durability. The cement film provides a bond between the concrete fill and the fabric, as well as a degree of protection against ultraviolet degradation. ***However, in view of the long-term performance at bridge piers that grout-filled mattresses must provide, performance should not depend on the fabric form material, but instead upon the weight and durability of the (cured) concrete grout, its cabled connections, and its ability to articulate, combined with the effectiveness of the underlying filter.***

Care should be taken to avoid over-pressurizing the mattress fabric forms that may cause seams to burst. Overfilling of the fabric form may create an obstruction to the stream flow and should be avoided. Any seam damaged during assembly, placement, or filling with grout should be promptly repaired or replaced according to manufacturer's instructions. Excessive damage may require replacement of entire fabric form panels.

Foot or vehicle traffic on the newly filled grout mattress system should be kept to a minimum for at least 1 hour, or until the concrete grout can resist indentation. If traffic is unavoidable, temporary boardwalks should be constructed over the grout-filled mattress.

2.4 Finishing

2.4.1 System Termination

Termination of the grout-filled mattress system shall be either (1) in excavated trenches that are properly backfilled with approved material flush with the top of the finished surface of the pillow-shaped blocks or (2) abutted to a structural feature such as a pier, footing, or pile cap. In the case of mattresses abutting a structural feature, the gap between the mattress and the structure shall be filled with cast-in-place concrete or structural grout and finished flush with the top surface of the grout-filled mattress system.

2.4.2 Anchors

If soil anchors are used, they may be either helical or duckbill type. Anchors must be capable of being attached directly to the mattresses, or to the ABM system cables. Anchors should have the capability of being load tested to ensure that the specified pullout capacity is achieved. Anchor penetrations through the geotextile shall be sealed with approved structural grout, mastic, or other sealant as approved by the engineer to prevent migration of subsoil through the penetration point.

2.4.3 Backfilling the Grout-Filled Mattress System

Backfilling with soil or granular fill within the cells of the system shall not be completed for a time designated by the engineer, generally at least 8 hours or more until the concrete grout has set and is durable enough for surface loading. When topsoil is used as a fill material above the normal waterline, the grout-filled mattress system should be overfilled by 1 to 2 in. (25 to 50 mm) to account for backfill material consolidation.

2.4.4 Inspection

The subgrade preparation, filter placement, grout-filled mattress system installation, and overall finished condition including termination trenches shall be inspected before work acceptance. Inspection guidelines are presented in detail in Part 3 of this appendix.

2.5 Measurement and Payment

Measurement of the grout-filled mattress system for payment shall be made on the basis of surface area. The pay lines will be neat lines taken off the contract drawings and will include embedded mattresses and/or mattresses placed in termination trenches.

The work includes grading and preparatory work, furnishing and installing the geotextile and grout-filled mattress system, constructing termination trenches, securing cables and fasteners, providing cable splices, installing soil anchors, backfilling, and seeding (where specified). Payment will be made at the respective unit price per square foot. Payment will be full compensation for all material, labor, and equipment to complete the work. The finished surface of the installation should be surveyed to ensure that the as-built lines and grades meet the design plans within the specified tolerance. Survey cross sections perpendicular to the axis of the structure are usually taken at specified intervals.

Part 3: Inspection, Maintenance, and Performance Evaluation

3.1 Inspection During Construction

Inspection during construction shall be conducted by qualified personnel who are independent of the contractor. Underwater inspection of a grout-filled mattress system shall be performed only by divers specifically trained and certified for such work.

3.1.1 Subgrade

Inspection of the subgrade shall be performed immediately prior to geotextile placement. The subgrade should be clean and free of projections, debris, construction materials, or other foreign objects that would prevent the filter and fabric form system from being properly placed. Likewise, there should be no potholes, rills, or other voids that the geotextile might bridge over.

The subgrade material itself should not be muddy or frozen and should not contain organic material or other deleterious substances. Variations in subgrade characteristics over the project area shall be noted and photographed; observations of such should be brought to the attention of the project engineer as they may represent conditions that are different from those used for design. It is generally recommended that compaction testing be performed at a frequency of one test per 2,000 ft² (186 m²) of surface area, unless project specifications require otherwise.

3.1.2 Geotextile

Each roll of geotextile delivered to the job site must have a label with the manufacturer's name and product designation. The inspector must check the labels to ensure that the geotextile is the same as that specified in the design. It is a good idea for inspectors to familiarize themselves with the different kinds of geotextiles on the market. Spun-bond fabrics and slit-film geotextiles should **never** be used in grout mattress applications.

The geotextile must be stored so that it is out of direct sunlight, as damage can occur from exposure to ultraviolet radiation. When placed, it must be free of wrinkles, folds, or tears. Sandbags, anchor pins, or U-shaped soil staples may be used to hold the geotextile in position while the mattress is being placed and filled with grout. The grout-filled mattresses should be placed within 48 hours after the geotextile is placed unless unusual circumstances warrant otherwise.

3.1.3 Grout-Filled Mattresses

The inspector shall check to ensure that the mattresses are sound and have been connected at edges with seams or zippers to form a continuous unit. Grout-filled mattresses used for pier scour countermeasures should never be allowed to overlap one another. The fabric forms must be stored out of direct sunlight, as damage can occur from exposure to ultraviolet radiation.

During inspection prior to pumping, the fabric form should be loose and an excess of fabric should be visible to account for form contraction, up to 10% in each direction. Seams joints, or zippers, should be carefully inspected. Termination trenches must be checked to ensure that the

specified toe-down depth is achieved prior to pumping. After pumping is complete, the trenches must be backfilled to ambient bed level with approved material.

3.2 Periodic and Post-Flood Inspection

As a pier scour countermeasure, a grout-filled mattress system will typically be inspected during the biennial bridge inspection program. However, more frequent inspection might be required by the Plan of Action for a particular bridge or group of bridges. In some cases, inspection may be required after every flood that exceeds a specified magnitude.

The most important aspect of inspecting a grout-filled mattress installation is to determine if the system is maintaining intimate contact with the subgrade. In the dry, such contact can be readily determined provided the system has not been buried by sediment. The inspector should look for the following:

- Buckled areas of mattresses
- Undermining or lifting of the system around the periphery
- Irregularities in the surface of the system (e.g., bumps or depressions)
- Gaps between the mattress and the structure
- Voids beneath the mattress or geotextile
- Deteriorated mattresses (e.g., freeze-thaw or wet-dry weathering, damage by abrasion or debris)

A length of reinforcing bar is helpful in detecting voids beneath the system. It can be used as a probe to poke into gaps between the mattress and the structure, or around the perimeter of the system. The inspector can also use it to thump the blocks and listen for a hollow ringing sound that would indicate the presence of a void beneath the system.

Underwater inspection of a grout-filled mattress system shall be performed only by divers specifically trained and certified for such work. Whether visually or by feel, the diver should pay particular attention to the areas where the mattress abuts structural elements to ensure that no gaps exist and that subgrade material is not being removed from beneath the grout-filled mattress system. Again, the use of a bar as a probe or thumper is particularly helpful. In addition, the diver should ascertain that the grout-filled mattresses have not been separated at their field-sewn seams, and that mattresses have not been repositioned such that one mattress is lying on top of an adjacent mattress. Also, the perimeter of the installation should be examined to determine if there are any areas where mattresses, or portions of mattresses, have been undermined or overturned.

An important aspect of inspecting a grout-filled mattress installation is to determine if the system has been repositioned by the flood flow to the extent that the subgrade or filter has become exposed, **or if the mattress has been undermined and is vulnerable to rolling up during the next flood**. The mattresses should be examined for evidence of displacement after a flood using the following criteria:

1. Grout-filled mattresses should have a **granular or synthetic geotextile filter** between the grout-filled mattress and the subgrade material.
2. At bridge piers, grout-filled mattresses should generally extend up to the bed elevation so that the top of the mattress is visible to the inspector during and after floods.
3. When grout-filled mattresses are inspected, affirmative answers to the following are strong indicators of problems:
 - Has the mattress been **displaced** downstream?
 - Are **voids** present under the mattress?
 - Has the mattress been undermined, **rolled**, or **overturned**?
 - Are there **holes** in the mattress where the filter has been exposed or breached?
 - Is the concrete grout **deteriorating** due to weathering or impact forces?

3.3 Maintenance

Deficiencies noted during the inspection should be corrected as soon as possible. Because grout-filled mattress systems are essentially an armor layer that is only one particle thick, any void beneath the system is vulnerable to further destabilization during the next high-flow event. Deficiencies noted during the inspection should be corrected as soon as possible. As with any armor system, progressive failure from successive flows must be avoided by providing timely maintenance intervention.

Any voids underneath the system must be filled. Depending on the size of the void and the nature of the grout-filled mattress system, the void often can be filled by pumping sand and gravel and/or concrete grout into the void via tremie pipe. Any gaps that are opening up between the mattress and the structure should be filled with grout to seal the gap against winnowing of the substrate material.

Grout-filled mattresses that have shifted or been uplifted may require removal and replacement. In this case, the cause of the displacement should be ascertained, and the use of a heavier mattress, deeper toedown, or anchors should be considered. If new mattresses are placed next to old mattresses, they should be seamed to the old mattresses prior to filling.

3.4 Performance Evaluation

The evaluation of any countermeasure's performance should be based on its design parameters as compared to actual field experience, longevity, and inspection/maintenance history. To properly assess the performance of a pier scour countermeasure, the history of hydraulic loading on the installation, in terms of flood magnitudes and frequencies, must also be considered and compared to the design loading.

Changes in channel morphology may have occurred over time subsequent to the installation of the pier scour countermeasure. Present-day channel cross-section geometry and planform should be compared to those at the time of countermeasure installation. Both lateral and vertical instability of the channel in the vicinity of the bridge can significantly alter hydraulic conditions at the piers. Approach flows may become skewed to the pier alignment, causing greater local and contraction scour.

It is recognized that the person making the performance evaluation will probably not be the inspector; however, inspection records will be fundamental to the evaluation. Maintenance records must also be consulted so that costs can be documented and reported as a percentage of the initial capital improvement cost.

3.4.1 Performance Rating Guide

To guide the performance evaluation for grout-filled mattress systems as a pier scour countermeasure, a rating system is presented in this section. It establishes numerical ratings from 0 (worst) to 6 (best) for each of three topical areas:

- Hydraulic history: Has the countermeasure been subjected to severe hydraulic loading since it was constructed?
- Maintenance history: Has the installation required a lot of attention and repair over its installed life to date?
- Current condition: What is the current condition of the countermeasure?

Tables G3.1 through G3.3 present a rating system for grout-filled mattress pier scour countermeasures. A single numerical score is not intended; rather, an independent rating (0-6 or U) is given for each of the three topical areas.

Table G3.1. Rating system for grout-filled mattress: hydraulic history.

Code	Hydraulic History	Code	Hydraulic History
U	N/A	3	Moderate: The countermeasure has experienced one or more flows greater than the 10-year event.
6	Extreme: The countermeasure has experienced one or more flows greater than the 100-year event.	2	Low: The countermeasure has experienced one or more flows greater than the 5-year event.
5	Severe: The countermeasure has experienced one or more flows greater than the 50-year event.	1	Very Low: The countermeasure has experienced one or more flows greater than the 2-year event.
4	High: The countermeasure has experienced one or more flows greater than the 25-year event.	0	Negligible: The countermeasure has not experienced any flows greater than a 2-year event.

Table G3.2. Rating system for grout-filled mattress: maintenance history.

Code	Maintenance History	Code	Maintenance History
U	N/A	3	Moderate: The system has required occasional maintenance since installation.
6	None Required: No maintenance has been needed since installation.	2	High: Frequent maintenance has been required.
5	Very Low: The system has required maintenance for very small, local areas once or twice.	1	Very High: Significant maintenance is usually required after flood events.
4	Low: The system has required minor maintenance.	0	Excessive: The system typically requires maintenance every year.

Table G3.3. Rating system for grout-filled mattress: current condition.

Code	Description of Current Condition	Code	Description of Current Condition
U	The grout-filled mattress system is uninspectable, due to burial by sediment, debris, or other circumstance.	3	Fair: Scour is evident at edges of mattress. Voids or undermining have been detected under the mattress, and/or significant areas have experienced articulation.
6	Excellent: The system is in excellent condition, with no evidence of mattress shifting, voids, gaps, or undermined areas.	2	Poor: The system exhibits significant shifting of mattresses. Undermining, gaps, or voids are present. Articulation indicates severe undermining has occurred.
5	Very Good: The system exhibits no shifting of mattresses. Some scour noted around the mattress periphery. No voids, gaps, or undermined areas evident.	1	Badly Damaged: The system has experienced substantial deterioration in terms of displaced mattresses, overturned edges, undermining, gaps or voids.
4	Good: The system exhibits minor voids in local areas, or small gaps between mattress and structure. Evidence that some undermining of the mattress has occurred, but the mattress has articulated properly to settle and seal the periphery.	0	Severe: The system has suffered damage such that it is no longer repairable. The only recourse is to remove the entire installation and replace it with a redesigned countermeasure.

Recommended actions based on current condition rating:

Code U: The grout-filled mattress system cannot be inspected. A plan of action should be developed to determine the condition of the installation. Possible remedies may include removal of debris, excavation during low flow, probing, or non-destructive testing using ground penetrating radar or seismic methods.

Codes 6 or 5: Continue periodic inspection program at the specified interval.

Codes 4 or 3: Increase inspection frequency. The rating history of the installation should be tracked to determine if a downward trend in the rating is evident. Depending on the nature of the grout-filled mattress application, the installation of monitoring instruments might be considered.

Code 2: The maintenance engineer's office should be notified and maintenance should be scheduled. The cause of the low rating should be determined, and consideration given to redesign and replacement. Materials other than grout-filled mattresses might be considered.

Codes 1 or 0: The maintenance engineer's office should be notified immediately. Depending upon the nature of the grout-filled mattress application, other local officials and/or law enforcement agencies identified in the Plan of Action for the bridge may also need to be notified.

3.4.2 Example Installations

A design for abutment protection that has been used by the Oregon Department of Transportation (ODOT) features an ABM system with continuous horizontal seams, allowing the blocks to bend downward by hinging along this seam line. The individual blocks are connected internally by a series of flexible polyester cables that keep the individual blocks firmly connected while allowing them to bend (Figure G3.1). Typical individual block sizes are on the order of 2.5 to 4 ft² (0.2 to 0.37 m²) and the weight of each block is approximately 400 lb (180 kg).

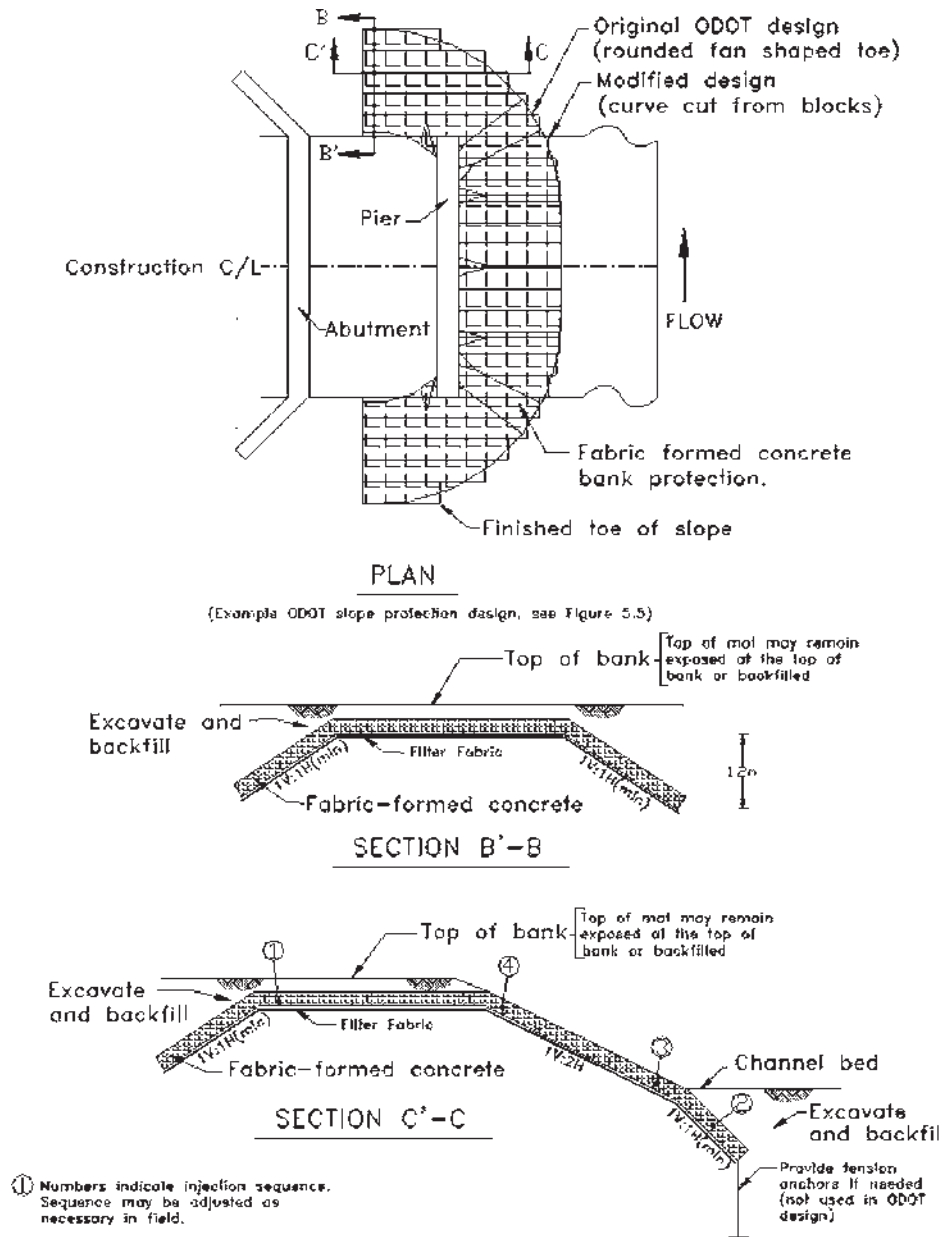
The following recommendations reflect experience from the ODOT and the Arizona Department of Transportation. Research reports from an ODOT installation of an articulating grout-filled mattress erosion control system on Salmon Creek in Oakridge, Oregon, also provide experience and insight on the use of these mattresses (Scholl 1991, Hunt 1993).

- Both upstream and downstream ends of the mattress should be trenched (Figure G3.2). The use of soil anchors can increase the stability of the mattress at the edges.
- All edges should be keyed in and protected to prevent undermining and flow behind the mattress.
- At abutments, the mattress can be wrapped around the abutment and buried to provide anchorage and to control flanking.
- It is recommended that weep holes be cut into the fabric at the seam to allow for proper drainage relief of pore pressure in the subgrade.
- The mattress should be filled with Portland cement slurry consisting of a mixture of cement, fine aggregate, and water. The mix should be in such proportion of water to be able to pump the mix easily, while having a compressive strength of 2,500 lb/in² (17,000 kPa).
- Fabric mattresses have been installed on slopes of 1V:1.5H or flatter.
- Large boulders, stumps, and other obstructions should be removed from slopes to provide a smooth application surface.
- Use sand and gravel for any backfill required to level slopes. Silty sand is acceptable if silt content is 20% or less. Do not use fine silt, organic material, or clay for backfill.
- The injection sequence should proceed from toe of slope to top of slope, but the mattress should be anchored at the top of slope first by pumping grout into the first rows of bags.



Source: ODOT

Figure G3.1. Articulating block mat appearance after filling.



Source: Scholl (1991) and Hunt (1993)

Figure G3.2. Typical articulated grout-filled mattress design.

by attaching the mattress to a structure, or by using soil anchors (see Figures G3.3 and G3.4).

- If the mattress is to be permanently anchored to a pier or abutment, there are implications that must be considered. The transfer of moments from the mattress to the pier may affect the structural stability of the bridge. When the mattress is attached to the pier, the increased loadings on the pier must be investigated.
- Curved-edge designs may require communication with the fabric manufacturer on shaping limitations and field adjustments.
- The need for a geotextile or granular filter should be addressed. Guidelines on the selection, design, and specifications of filter material can be found in HEC-11 (Brown and Clyde 1989) and Holtz et al. (1995).



Source: Scholl (1991)

Figure G3.3. Installation of articulating grout-filled mattress proceeding upslope (ODOT).



Source: Scholl (1991)

Figure G3.4. ABM underneath Salmon Creek Bridge (ODOT).

The following problems and solutions were identified in the construction process by ODOT (Scholl 1991, Hunt 1993):

- *Problem:* In the original attempt to create a smooth working surface for laying the fabric, sand was placed over the native material. This was a problem because footprints readily disturbed the surface.
Solution: The native material (a gravelly sand) was used for the final surface by first clearing it of major rocks, then compacting it.
- *Problem:* There was difficulty in estimating where the toe of the finished slope would be.
Solution: The fabric was assumed to contract by 10% in length after filling with grout.
- *Problem:* Straight lines were difficult to maintain along the horizontal seams when grout was being pumped.
Solution: The fabric was kept straight by tying it to a series of #6 reinforcing bars.
- *Problem:* Several of the bags were sewn in such a way that the grout ducts connecting them to the other bags were blocked off. This occurrence was mostly in areas where the bags were cut during fabrication to only half the original size.
Solution: The bags were split and filled individually. This solution should not affect the strength or function of the system.

Figures G3.5 and G3.6 show the construction of a grout-filled mattress installation on the Gila River in Yuma County, Arizona, in 1998. The grout-filled mattress was placed in a continuous layer to protect the abutment slopes, pier columns, and the channel bed between the piers. This same design was used successfully at three county road bridges (20E, 45E, and 64E) by the Engineering Division of the Yuma County Department of Development Services.



Note dewatering wellpoint system.

Source: Yuma County Department of Development Services

Figure G3.5. Slope preparation prior to grout-filled mattress placement.



Note toedown depth from crest to toe of structure.

Source: Yuma County Department of Development Services

Figure G3.6. Completed grout-filled mattress placement showing backfilling in progress.

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APPENDIX H

Summary of Laboratory Testing Program

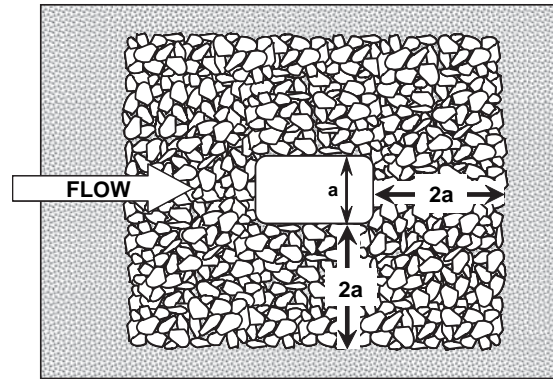
This appendix contains detailed summary tables for the laboratory testing program for each pier scour countermeasure type. Tests were conducted for baseline conditions and to optimize the extent of coverage, termination details, and filter recommendations for the following:

- Riprap
- Partially grouted riprap and geotextile containers
- Articulating concrete block systems
- Gabion mattresses
- Grout-filled mattresses

These tests were conducted in the 8-foot wide, 200-foot long, and 4-foot deep recirculating flume at Colorado State University (CSU). Additional testing of partially grouted riprap was conducted at a prototype scale in the CSU outdoor flume (20 feet wide, 108 feet long, and 8 feet deep) to evaluate constructability and environmental issues related to placing this countermeasure type.

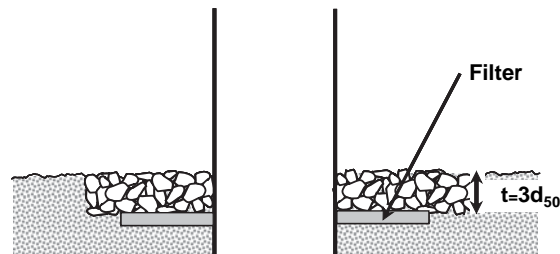
For each test series, the tables provide information on layout, filter, and design intent for each test, as well as a brief commentary on observations and/or results. Identification of a specific test run and pier is also provided for cross-reference to the more detailed description of testing results in the Reference Document (available on the TRB website: http://www.trb.org/news/blurbs_detail.asp?id=7998).

Riprap



Pier width = "a" (normal to flow)

Riprap placement = 2(a) from pier (all around)



Riprap thickness = $3d_{50}$ (minimum)

Filter placement = $4/3(a)$ from pier (all around)

Figure H.1. Baseline riprap design.

Table H.1. Baseline Riprap Tests.

Layout	Filter	Design Intent	Observations	Test	Pier
5a areal extent, $3d_{50}$ thickness	Geotextile filter, 2/3 coverage	HEC-23 guidelines with recommended geotextile, no skew	Stable, minimal particle launching after 8 hours at $2.5V_{crit}$, catastrophic loss after exposure to 8 more hours of testing at $3.0V_{crit}$	4a	B
			Stable, minimal particle launching after 2 hours at $2.0V_{crit}$. Stable, minimal particle launching after 2 more hours at $2.5V_{crit}$. Stable, minimal particle launching after 2 hours at $2.0V_{crit}$. Loss/damage moderate after 2 more hours at $3.0V_{crit}$	5	B
			Stable, minimal particle launching after 2 hours at $2.0V_{crit}$. Stable, minimal particle launching after 5 more hours of testing at $2.0V_{crit}$	7	C
			Stable, minimal particle launching on downstream corner after 5 hours at $1.9V_{crit}$	11-3	C
			Stable, minimal particle launching after 2 hours at $2.0V_{crit}$	C12	B
			Loss/damage significant after 2 hours at $2.0V_{crit}$, pier skewed to flow 15°	C12	A
	Loss/damage significant after 2 hours at $2.0V_{crit}$, pier skewed to flow 30°	C12	C		

Table H.2. Riprap Coverage Tests.

Layout	Filter	Design Intent	Comments	Test	Pier	
4a lateral, 2a upstream, 1a downstream, $3d_{50}$ thickness at pier for 1/3 extent, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter	Geotextile filter, 2/3 coverage	Areal riprap coverage and edge treatment with recommended geotextile	Stable after 2 hours at $2.0V_{crit}$, some particle launching into side troughs	8	B	
3a areal extent, $3d_{50}$ thickness at pier, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter			Loss/damage severe after 2 hours at $2.0V_{crit}$, particle displacement in troughs and downstream of pier	8	C	
4a decreased areal extent, $3d_{50}$ thickness at pier for 1/3 extent, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter			Stable after 2 hours at $2.0V_{crit}$	8	A	
3a decreased areal extent, $3d_{50}$ thickness	None	Areal riprap coverage variation from HEC-23 with recommended geotextile	Loss/damage moderate after 2 hours at $2.0V_{crit}$, particle launching on all sides	7	A	
4a decreased areal extent, $3d_{50}$ thickness			Loss/damage after 2 hours at $2.0V_{crit}$, particle displacement in side troughs	7	B	
			Loss/damage severe launching and particle displacement after 8 hours at $2.5V_{crit}$. Catastrophic loss/damage after exposure to 8 more hours of testing at $3.0V_{crit}$	4a	C	
			Loss/damage minimal after 2 hours at $2.0V_{crit}$	C16	C	
4a decreased areal extent, $4.5d_{50}$ thickness	None	Areal riprap coverage and thickness variation from HEC-23 with recommended geotextile	Loss/damage severe, scour at nose after 8 hours at $2.5V_{crit}$	4c	B	
Based on scour hole dimension, $3d_{50}$ thickness			Scour hole extent with recommended geotextile	Stable, minimal particle launching after 2 hours at $2.0V_{crit}$	C14	B
				Stable, minimal particle launching after 2 hours at $2.0V_{crit}$ pier skewed to flow 15°	C14	A
		Stable, minimal particle launching after 2 hours at $2.0V_{crit}$ pier skewed to flow 30°	C14	C		
	None	Scour hole extent	Stable, minimal particle displacement after 2 hours at $2.0V_c$	3	C	
5.5a areal extent, $3d_{50}$ thickness for 3a, bottom or riprap sloped upward 1.5H:1V to thickness of $1d_{50}$ at perimeter	Geotextile filter, full coverage	Examine HEC-18 guidelines	Stable after 2 hours at $2.0V_c$	3	B	
5a areal extent, $4.5d_{50}$ thickness	None	Thickness and filter variation from HEC-23 guidelines	Loss/damage moderate, scour at nose after 8 hours at $2.5V_{crit}$. Loss/damage severe scour at nose after 8 more hours at $2.5V_{crit}$	6	C	
	Granular filter, $4d_{50}$ thickness, 2/3 coverage		Loss/damage moderate, scour at nose after 8 hours at $2.5V_c$. Loss/damage severe scour at nose after more 8 hours at $2.5V_c$	6	B	
5a areal extent, $4d_{50}$ thickness at pier, slope to height of $1d_{50}$ at perimeter, mounded	None	Examine mounded riprap	Loss/damage after 2 hours at $2.0V_{crit}$, particle displacement in trough and sand filling in riprap matrix after 3 hours at $2.0V_{crit}$	C16	A	
5a areal extent, $2.5d_{50}$ thickness, mounded			Loss/damage after 2 hours at $2.0V_{crit}$, particle displacement in trough and sand filling in riprap matrix after 4 hours at $2.0V_{crit}$	C16	B	
5a areal extent, $3d_{50}$ thickness all around, mounded			Loss/damage moderate after 2 hours at $2.0V_{crit}$, particle launching on all sides	13	C	

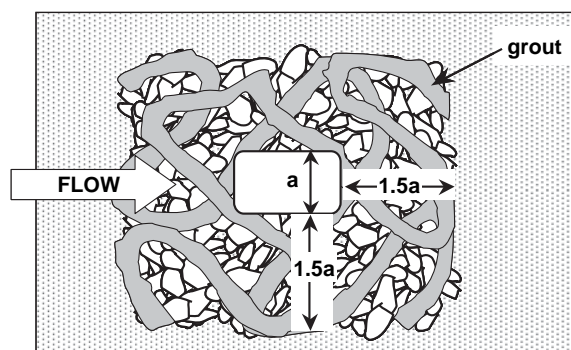
Table H.3. Riprap Termination Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
4a lateral, 2a upstream, 1a downstream. $3d_{50}$ thickness at pier for 1/3 extent, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter	Geotextile filter, 2/3 coverage	Areal coverage and edge treatment with recommended geotextile	Stable after 2 hours at $2.0V_{crit}$, some particle launching into side troughs	8	B
3a areal extent, $3d_{50}$ thickness at pier, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter			Loss/damage severe after 2 hours at $2.0V_{crit}$, particle displacement in troughs and downstream of pier	8	C
4a decreased areal extent, $3d_{50}$ thickness at pier for 1/3 extent, then 2:1 slope at bottom to thickness of $6d_{50}$ at perimeter			Stable after 2 hours at $2.0V_{crit}$	8	A
5.5a areal extent, $3d_{50}$ thickness for 3a, bottom or riprap sloped upward 1.5H:1V on perimeter	Geotextile filter, full coverage	Examine HEC-18 (3 rd edition) guidelines	Stable after 2 hours at $2.0V_{crit}$	3	B

Table H.4. Riprap Filter Tests.

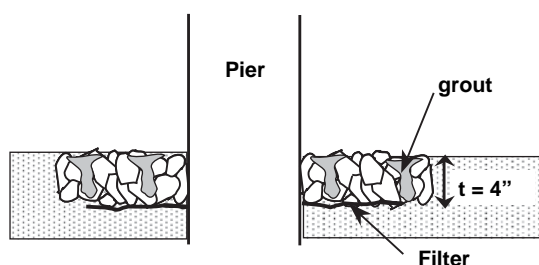
Layout	Filter	Design Intent	Comments	Test	Pier
5a areal extent, $4.5d_{50}$ thickness	2/3 granular filter $4d_{50}$ thickness	Thickness and filter variation from HEC-23 guidelines	Loss/damage moderate, scour at nose after 8 hours at $2.5V_{crit}$. Loss/damage severe scour at nose after 8 more hours at $2.5V_{crit}$	6	B
5a areal extent, $3d_{50}$ thickness		HEC-23 guidelines, filter type	Loss/damage severe, scour at nose after 8 hours at $2.5V_{crit}$	4c	C
	Full granular filter, $4d_{50}$ thickness		Loss/damage moderate after 2 hours at $2.0V_{crit}$. Loss/damage moderate after 2 more hours at $2.5V_{crit}$. Loss/damage catastrophic after 2 more hours at $3.0V_{crit}$	5	C
	Full geotextile		Stable, minimal particle loss after 2 hours at $2.0V_g$	5	A
5.5a areal extent, $3d_{50}$ thickness for 3a, bottom of riprap sloped upward 1.5H:1V on perimeter		Examine HEC-18 (3 rd edition) guidelines	Stable after 2 hours at $2.0V_{crit}$	3	B
Based on scour hole dimension, $3d_{50}$ thickness	None	Examine current practice	Stable, minimal particle launching after 2 hours at $2.0V_{crit}$	3	C
5a areal extent, $3d_{50}$ thickness		Examine current guidelines	Stable after 2 hours at $2.0V_{crit}$	3	A
			Stable, minimal particle launching after 2 hours at $2.0V_{crit}$	12	C
5a areal extent, $4.5d_{50}$ thickness		Thickness and filter variation from HEC-23 guidelines	Loss/damage moderate, scour at nose after 8 hours at $2.5V_{crit}$. Loss/damage severe, scour at nose after 8 more hours at $2.5V_{crit}$	6	C
5a areal extent, $4d_{50}$ thickness at pier, slope to height of $1d_{50}$ at perimeter		Examine mounded riprap	Loss/damage after 2 hours at $2.0V_{crit}$, particle displacement in trough and sand filling in riprap matrix after 3 hours at $2.0V_{crit}$	C16	A
5a areal extent, $2.5d_{50}$ thickness			Loss/damage after 2 hours at $2.0V_{crit}$. Particle displacement in trough and sand filling in riprap matrix after 2 more hours at $2.0V_{crit}$	C16	B
5a areal extent, $3d_{50}$ thickness all around, mounded			Loss/damage moderate after 2 hours at $2.0V_{crit}$, particle launching on all sides	13	C

Partially Grouted Riprap



Pier width = "a" (normal to flow)

Extend partially grouted riprap a distance of 1.5(a) from pier (minimum, all around)



Filter placement = 1.0(a) from pier (all around)

Figure H.2. Partially grouted riprap baseline design layout.

Table H.5. Partially Grouted Riprap Baseline Tests.

Layout	d_{50}	Filter	Design Intent	Comments	Test	Pier
4a areal extent, $6.67d_{50}$ full extent of thickness, 100 mL grout per spot	0.6 in	2/3 geotextile filter	Rock size performance	Stable, minimal particle launching at sides after 2 hours at $2.0V_{crit}$	15	A
4a areal extent, $4d_{50}$ full extent of thickness, 65 mL grout per spot	1.0 in			Loss/damage severe after 8 hours at $2.5V_{crit}$ breakup into numerous conglomerates	14	C
4a areal extent, $4d_{50}$ full extent of thickness, 100 mL grout per spot				Stable after 2 hours at $2.0V_{crit}$, some undermining at downstream sides, excess grout created rigid mass	15	B
4a areal extent, $3d_{50}$ full extent of thickness, 100 mL grout per spot	1.3 in			Undermining on all sides, after 2 hours at $2.0V_{crit}$, excess grout created rigid mass	15	C

Table H.6. Partially Grouted Riprap Coverage Tests.

Layout	d ₅₀	Filter	Design Intent	Comments	Test	Pier
4a lateral extent, turndown 4:1, 3.3d ₅₀ thickness, 65 mL grout per spot	0.6 in	2/3 geotextile filter	Layer thickness and termination detail	Stable after 2 hours at 2.0V _{crit}	16	A
4a lateral extent, 3.3d ₅₀ thickness, 65 mL grout per spot				Stable after 2 hours at 2.0V _{crit}	16	B
4a lateral extent, turndown 4:1, 3.3d ₅₀ thickness, 65 mL grout per spot				Stable after 2 hours at 2.0V _{crit}	17	C
4a lateral extent, 5d ₅₀ thickness, 100 mL grout per spot			Layer thickness	Loss/damage minimal, undermining on sides after 2 hours at 2.0V _{crit}	16	C
3a lateral extent, 3.3d ₅₀ thickness, 65 mL grout per spot			Areal coverage and thickness	Stable, minimal particle launching at sides after 2 hours at 2.0V _{crit}	17	A
3a lateral extent, turndown 4:1, 3.3d ₅₀ thickness, 65 mL grout per spot			Areal coverage, layer thickness, and termination detail	Stable, minimal particle launching at sides after 2 hours at 2.0V _{crit}	17	B
5a lateral extent, turndown 2:1, 3d ₅₀ thickness, 65 mL grout per spot	1.0 in			Stable after 8 hours at 2.5V _{crit}	18	B

Table H.7. Partially Grouted Riprap Termination Tests.

Layout	d ₅₀	Filter	Design Intent	Comments	Test	Pier
4a lateral extent, 3.3d ₅₀ thickness, turndown 4:1, 65 mL grout per spot	0.6 in	2/3 geotextile filter	Layer thickness and termination detail	Stable after 2 hours at 2.0V _{crit}	16	A
4a lateral extent, sloped down 4:1, 3.3d ₅₀ thickness, 65 mL grout per spot				Stable after 2 hours at 2.0V _{crit}	17	C
3a lateral extent, sloped down 4:1, 3.3d ₅₀ thickness, 65 mL grout per spot			Areal coverage, layer thickness, and termination detail	Stable, minimal particle launching at sides after 2 hours at 2.0V _{crit}	17	B
5a lateral extent, sloped down 2:1, 3d ₅₀ thickness, 65 mL grout per spot	1.0 in			Stable after 8 hours at 2.5V _{crit}	18	B

Table H.8. Partially Grouted Riprap Prototype Scale Tests.

Layout	Filter	Design Intent	Comment	Test
3.5 pier widths from pier face	Sand-filled geocontainers	Constructability	Installation of geocontainers, riprap, and grout in slow-moving water (1.5 ft/s) was accomplished	C1
		Environmental issues	Water quality parameters during underwater grout application remained below maximum allowable	
		High-velocity performance test	Partially grouted riprap remained stable after exposure to 5.6 ft/s flow conditions	
3.5 pier widths from pier face, half of installation composed of loose riprap		High-velocity, comparison to loose riprap	Partially grouted riprap remained stable, loose riprap was displaced after exposure to 6.4 ft/s flow conditions	C20

Articulating Concrete Block Systems

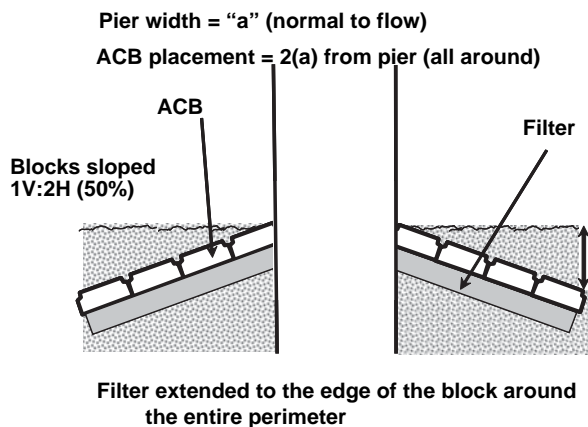
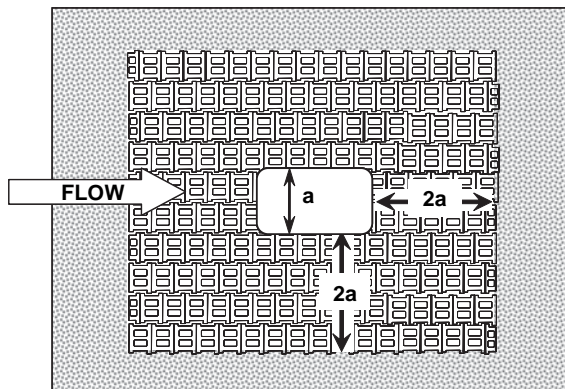


Figure H.3. Baseline ACB design.

Table H.9. ACB Baseline Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
2:1 turndown for 5a lateral	Geotextile filter, full coverage	Standard ACB layout	Minor loss after 2 hours at $1.9V_{crit}$	12	B
		Standard ACB layout with grouted interface	Stable after 2 hours at $1.9V_{crit}$	13	B
			Loss/damage after 8 hours at $2.5V_{crit}$ on right side	14	B

Table H.10. ACB Extent of Coverage Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
3:1 turndown on downstream and sides for 1.5a to a depth of 0.5a, upstream extends horizontal for 1.5a then 4:1 turndown ending at 0.5a depth	Geotextile filter, full coverage	Areal coverage and termination detail	Stable after 2 hours at $1.9V_{crit}$, loss from upstream after 5 hours at $1.9V_{crit}$	11-2	B
3:1 turndown from pier all sides from 0.25a below bed for 2a lateral ending at a depth of 1.5a			Stable after 2 hours at $1.9V_{crit}$, stable after 5 hours at $1.9V_{crit}$	11-3	B
2:1 turndown for 1a, riprap key installed to 5a at a depth of 0.5a	Geotextile filter extended beyond perimeter of the blocks into the riprap	ACBs in conjunction with riprap	Stable after 2 hours at $1.9V_{crit}$	12	A
2:1 turndown for 1a, riprap installed to 5a with inverted 2:1 slope at bottom edge			Stable after 2 hours at $1.9V_{crit}$	13	A

Table H.11. ACB Termination Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
5a horizontal with 10-inch radius of curvature ending at 0.5a depth	Geotextile filter, full coverage	Termination detail	Loss/damage downstream after 2 hours at $1.9V_{crit}$, undamaged at leading edge	11-1	B
3:1 turndown on downstream and sides for 1.5a to a depth of 0.5a, downstream extends horizontal for 1.5a then 4:1 turndown ending at 0.5a depth		Areal coverage and termination detail	Stable after 2 hours at $1.9V_{crit}$, loss from upstream after 5 hours at $1.9V_{crit}$	11-2	B
3:1 turndown from pier all sides from 0.25a below bed for 2a lateral ending at a depth of 1.5a			Stable after 2 hours at $1.9V_{crit}$, stable after 5 hours at $1.9V_{crit}$	11-3	B
2:1 turndown for 1a, riprap key installed to 5a at a depth of 0.5a	Geotextile filter extended beyond perimeter of the blocks into the riprap	ACB used in conjunction with riprap	Stable after 2 hours at $1.9V_{crit}$	12	A
2:1 turndown for 1a, riprap installed to 5a with inverted 2:1 slope at bottom edge			Stable after 2 hours at $1.9V_{crit}$	13	A

Table H.12. ACB Filter Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
2:1 turndown for 5a lateral beginning 0.5a below bed with interface grouted	Geotextile filter, 2/3 coverage	Standard ACB layout with grouted interface below the bed surface	Loss/damage nearly complete after 8 hours at $2.5V_{crit}$ and deformation of subgrade beneath geotextile	18	C
2:1 turndown for 1a, riprap key installed to 5a at a depth of 0.5a	Geotextile filter extended beyond perimeter of the blocks into the riprap	ACB used in conjunction with riprap	Stable after 2 hours at $1.9V_{crit}$	12	A
2:1 turndown for 1a, riprap installed to 5a with inverted 2:1 slope at bottom edge			Stable after 2 hours at $1.9V_{crit}$	13	A

Gabion Mattresses

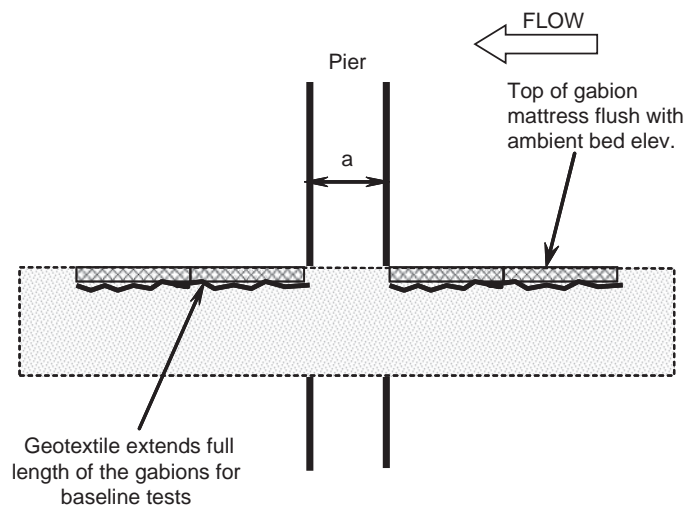


Figure H.4. Baseline unconnected gabion mattress coverage test layout.

Table H.13. Unconnected Gabion Mattress Coverage Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
4a areal extent parallel to flow, 4a normal to flow	Full geotextile	Areal coverage	Minimal gabion displacement on front and sides	C3	B
4a areal extent parallel to flow, 3a normal to flow			Loss/damage severe, gabions displaced downstream, deformation of subgrade and scour at nose	C3	C
5.5a areal extent parallel to flow, 5a normal to flow			Stable, gabion displacement on periphery	C3	A

Table H.14. Unconnected Gabion Mattress Termination Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
4a horizontal extent adjacent to the pier, 3:1 turndown at periphery to depth of 0.25a, total areal extent 5.5a	Full geotextile	Termination detail	Loss of gabions from front of pier; gabion displacement on sides and in back	C4	C
4:1 turndown to a depth of 0.38a, 4a areal extent			Loss of gabions from periphery downstream of pier		
2:1 turndown to a depth of 0.75a, 4a lateral extent	2/3 geotextile filter	Filter extent and termination detail	Loss displacement of gabions on periphery	C5	B
			Loss/damage catastrophic, gabions completely displaced downstream, deformation of subgrade and significant scour at nose	C5	C
0.75a horizontal extent from pier face, 2:1 turndown at periphery to depth of 0.38a, 4a total areal extent		Termination detail	Loss/damage severe, deformation of subgrade below geotextile, significant scour at nose	C5	A

Table H.15. Unconnected Gabion Mattress Filter Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
2:1 turnout to a depth of 0.75a, 4a lateral extent	2/3 geotextile filter	Filter extent and termination detail	Loss/damage catastrophic, gabions completely displaced downstream, deformation of subgrade and significant scour at nose	C5	C
4:1 turnout to a depth of 0.38a, 4a areal extent			Loss displacement of gabions on periphery	C5	B
0.75a horizontal extent from pier face, 2:1 turnout at periphery to depth of 0.38a, 4a total areal extent			Loss/damage severe, deformation of subgrade and scour at nose	C5	A
5.5a areal extent parallel to flow, 5a normal to flow	None	Filter extent	Displacement of gabions into trough on sides of installation	C4	A

Table H.16. Connected Gabion Mattresses Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
0.75a horizontal extent from pier face, 2:1 turnout at periphery to depth of 0.38a, 4a total areal extent	2/3 geotextile filter	Gabions connected at top edge	Undermining of gabions behind pier	C19	A
2:1 turnout to a depth of 0.75a, 4a lateral extent			Loss of material under gabion mattress at pier nose	C19	C
4:1 turnout to a depth of 0.38a, 4a areal extent			Stable	C19	B

Grout-Filled Mattresses

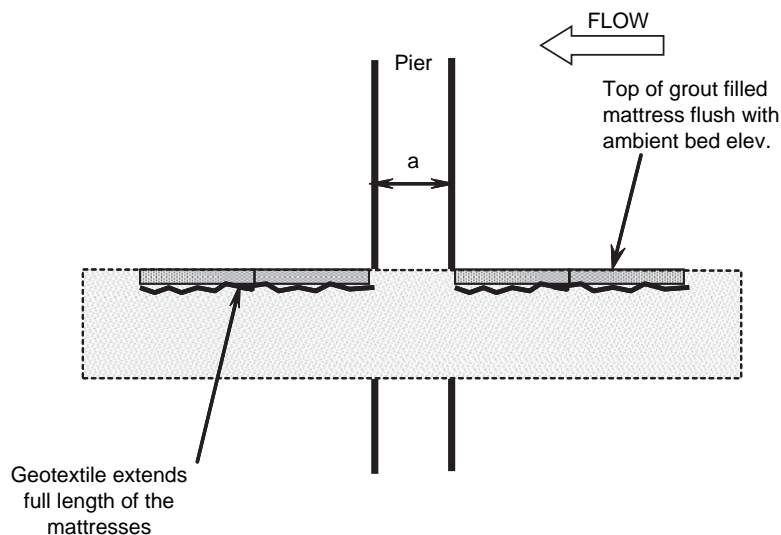
**Figure H.5. Typical grout-filled mattress coverage test layout.**

Table H.17. Rigid Grout-Filled Mattress Coverage Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
4a areal extent parallel to flow, 3a normal to flow	Full geotextile	Areal coverage	Minor undercutting after 2 hours at $1.0V_{crit}$. After 2 hours at $2.0V_{crit}$ undercutting was severe; pier exposed under mattress	C7	C
4a areal extent parallel to flow, 5a normal to flow			Undercutting around entire perimeter after 2 hours at $2.0V_{crit}$	C7	B
5.5a areal extent parallel to flow, 5a normal to flow			Undercutting on sides and behind pier after 2 hours at $2.0V_{crit}$	C7	A

Table H.18. Rigid Grout-Filled Mattress Termination Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
2:1 turndown to a depth of 0.75a, 4a parallel to flow, 5a normal to flow	Full geotextile	Termination detail	Undercutting on sides after 2 hours at $2.0V_{crit}$	C8	C
4:1 turndown to a depth of 0.38a, 4a areal extent parallel to flow, 5a normal to flow			Undercutting on sides after 2 hours at $2.0V_{crit}$	C8	B
0.75a horizontal extent from pier face, 3:1 turndown at periphery to depth of 0.25a, 4a total areal extent			Undercutting around entire perimeter; pier exposed under mattress after 2 hours at $2.0V_{crit}$	C8	A

Table H.19. Flexible Grout Mattress Tests.

Layout	Filter	Design Intent	Comments	Test	Pier
2:1 turndown to a depth of 0.75a, 4a parallel to flow, 5a normal to flow	Full geotextile	Articulating grout mattress	Undercutting severe; pier exposed under mattress after 2 hours at $2.0V_{crit}$	C18	C
4:1 turndown to a depth of 0.38a, 4a areal extent parallel to flow, 5a normal to flow			Undercutting on sides and behind pier after 2 hours at $2.0V_{crit}$	C18	B
0.75a horizontal extent from pier face, 2:1 turndown at periphery to depth of 0.25a, 4a total areal extent			Undercutting behind pier, mats had begun to gap after 2 hours at $2.0V_{crit}$	C18	A

Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI-NA	Airports Council International-North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation